

Flexural behavior of plain and reinforced concrete beams incorporating colloidal nano silica: Experimental and finite element investigation

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Abstract

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Colloidal nano silica is a highly reactive pozzolanic material that can refine the microstructure and improve the strength of concrete. Its effect has mostly been evaluated at the material level, while its influence on the flexural behavior of reinforced concrete beams and on finite element prediction of this behavior has received less attention. This paper presents an experimental and numerical investigation on plain and reinforced concrete beams made with 2.5% colloidal nano silica as a partial cement replacement. Beam specimens of $100 \times 150 \times 1360$ mm were cast for plain and reinforced cases, using a control mix and a colloidal nano silica mix, with three specimens per condition. All beams were water-cured for 28 days and tested to failure under four-point bending. Finite element models were built in Abaqus using the concrete damaged plasticity model, with inputs from cylinder test data. The reinforced colloidal nano silica beam showed about 10% higher yield load, 5.8% higher ultimate load, and 22% greater pre-yield energy absorption than the control, while the plain colloidal nano silica beam carried about 10% more load at failure. However, the reinforced colloidal nano silica beam reached its ultimate load at a smaller deflection, indicating reduced ductility. The finite element predictions showed differences of 0.44% to 5.23% in ultimate load and 3.03% to 9.31% in mid-span deflection. The results show that colloidal nano silica improves the pre-yield stiffness and delays cracking in reinforced concrete beams but reduces their ductility. The model was preliminarily verified using the results obtained from the experimental study and is limited to the tested configurations.

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1. Introduction

Concrete and steel have been used together in structural construction for more than a century. Concrete carries compression, steel carries tension, and together they form a composite that resists bending and shear in ways neither material could manage alone [1]. Plain concrete beams crack on the tension face under modest loads and fail with little warning. Once steel reinforcement is placed on the tension side, load is redistributed across the cracks, deflection develops gradually rather than suddenly, and the member gains the ductility that safe structural design requires [2].

Reinforcement improves the load behavior of beams but does not address durability. Concrete is porous, and this porosity allows chloride ions, carbon dioxide, and moisture to reach the embedded steel, which is the main cause of long-term deterioration in reinforced concrete structures [3, 4]. To refine the pore structure, the construction industry uses supplementary cementitious materials such as fly ash, ground granulated blast furnace slag, and silica fume, which react pozzolanically to consume calcium hydroxide and form additional C-S-H gel [4, 5]. These materials are effective, but their particle sizes prevent them from filling the finest pores in the cement matrix [5, 6]. Nano silica,

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with particle sizes between 5 and 50 nm, occupies pores that conventional supplementary materials cannot reach, including those in the interfacial transition zone (ITZ) between the cement paste and the aggregate [5, 6]. The ITZ is the weakest region in concrete because water films around the aggregate raise the local water-to-cement ratio and promote oriented $\text{Ca}(\text{OH})_2$ crystals, producing a zone that is weaker and more permeable than the bulk paste [4]. Nano silica densifies this zone by filling its voids and by reacting with $\text{Ca}(\text{OH})_2$ to form additional C-S-H gel [5, 7, 8]. Because the ITZ governs the bond and tensile response at the aggregate interface, a denser ITZ can raise the first cracking load, increase the post-cracking stiffness, and influence how cracks propagate through the section [7, 8].

The mechanical benefits of nano silica are consistent but dosage-dependent. Compressive and tensile strength gains are well documented at replacement levels of about 1 to 3% by weight of cement [5, 9]. Beyond this range, workability drops and the strength gains disappear, because nanoparticle agglomeration counteracts the filling and pozzolanic effects [9, 10]. Chloride penetration resistance follows a similar trend, improving at low dosages [11], and combining nano silica with micro silica produces better durability in mortar than either material alone [12]. These material-level gains, however, do not translate proportionally to the structural response of a reinforced beam. The flexural behavior of a beam is governed by the interaction between the concrete and the reinforcement, by bond, and by cracking mechanics, so an increase in matrix strength does not necessarily produce a proportional increase in load capacity or ductility. A stronger and stiffer matrix can raise the cracking load and the pre-yield stiffness, but it may also reduce the deformation capacity of the member [13]. The structural effect of colloidal nano silica therefore cannot be inferred from material tests alone and must be evaluated at the beam level.

Studies on reinforced concrete beams incorporating nano silica are fewer than the volume of material-level research would suggest, and most have notable scope limitations. Work on flexural behavior has largely focused on high-strength concrete rather than normal-strength mixes, often combined with manufactured sand, which makes it difficult to isolate the contribution of nano silica [14]. Other studies combined nano silica with steel fibers, which further complicates attributing any improvement to nano silica alone [15], and numerical work has tended to examine self-compacting concrete rather than conventionally placed structural mixes [16]. Nano silica is commercially available in two forms, dry powder and colloidal suspension, and most structural studies have used the powder form. Colloidal nano silica differs in an important way because its particles are dispersed in water before they enter the mix, which reduces the agglomeration that limits powder nano silica at higher dosages [17, 18]. It has been reported to improve workability and strength at low replacement levels, partly because of this better dispersion [17, 18]. Despite this, the flexural response of normal-strength reinforced concrete beams made with colloidal nano silica, including first cracking, post-cracking stiffness, ultimate capacity, crack pattern, and energy absorption, has received very little attention under standard four-point bending.

Because experimental testing is costly and time-consuming, a validated numerical model is useful for extending a limited dataset and for supporting further study. The concrete damage plasticity (CDP) model in Abaqus is widely used for the nonlinear analysis of reinforced concrete members. Its plastic damage formulation was established by Lubliner et al. [19] and extended by Lee and Fenves [20], and it has been applied to post-tensioned beams and to beams with different stirrup configurations [21, 22]. The accuracy of the model, however, depends on the selection of its parameters, including the mesh, the tensile softening law, the fracture energy, and the bond assumptions, so its predictions must be verified against measured data for each application. For colloidal nano silica concrete, the inputs such as the stress-strain curves, tensile strength, and elastic modulus cannot be taken from standard concrete data and must be obtained from tests on the modified material, which the present study provides.

This study investigates the flexural behavior of normal-strength reinforced concrete beams incorporating 2.5% colloidal nano silica as a partial cement replacement. The 2.5% dosage, expressed as the mass of the colloidal suspension, was adopted based on the optimum colloidal silica content established for compressive strength in earlier work [23, 24], which provides the material basis for the present beam study. For each of the four conditions, namely plain control,

plain colloidal nano silica, reinforced control, and reinforced colloidal nano silica, three specimens were cast and tested under four-point bending at 28 days, giving a total of twelve beams. Finite element models of the beams were built in Abaqus using cylinder test data and compared with the measured responses. It is hypothesized that 2.5% colloidal nano silica increases the first cracking load, the pre-yield stiffness, and the load capacity of the beams, while reducing their ductility because of the stiffer and more brittle matrix. The objectives are, first, to quantify the structural effect of colloidal nano silica on flexural capacity, stiffness, cracking behavior, and energy absorption; and second, to develop a numerical model verified against the experimental data that can provide a basis for further study of nano-modified reinforced concrete beams.

The significance of this study lies in linking the material-level behavior of colloidal nano silica to the structural flexural response of reinforced concrete beams and in providing a numerical model verified against the measured data for the beams tested. This contributes to the understanding of how colloidal nano silica affects the flexural behavior of reinforced concrete members within the range of configurations investigated in this study.

2. Experimental Program

2.1 Materials

Ordinary Portland Cement (OPC) of grade 43, conforming to IS 8112:2013 [25], was used throughout this investigation, with a specific gravity of 3.12, a standard consistency of 29%, and an initial setting time of 132 minutes. Natural river sand conforming to Zone II of IS 383:2016 [26] was used as fine aggregate, with a specific gravity of 2.6, a fineness modulus of 2.26, and a bulk density of 1598 kg/m³. Crushed granite of 10 mm maximum nominal size, conforming to IS 383:2016 [26], was used as coarse aggregate, with a specific gravity of 2.70, a fineness modulus of 6.1, and a bulk density of 1563 kg/m³. Potable water free from impurities was used for both mixing and curing.

Commercially available colloidal nano silica (CemSynXTX, Bee Chems, India) was used as a partial cement replacement. The suspension had an active nano silica solids content of 30%, a specific gravity of 1.20, a pH between 9.0 and 10.0, and a silica purity of 99.99% SiO₂. The average particle size of the colloidal nano silica ranged from 10 to 20 nm.

High-yield-strength deformed bars of 8 mm diameter, Fe550 grade, conforming to IS 1786:2008 [27], were used as longitudinal reinforcement, and 6 mm diameter mild steel plain bars of Fe250 grade, conforming to IS 432 (Part 1):1982 [28], were used for stirrups. Uniaxial tensile tests were carried out on samples of both bar diameters in accordance with IS 1608:2005 [29], and the measured mechanical properties are reported in Table 1.

Table 1. Properties of reinforcing steel obtained from uniaxial tensile tests

Steel bar diameter	Yield strength (MPa)	Ultimate strength (MPa)	Modulus of elasticity (MPa)
8 mm	612	670	210000
6 mm	392.215	515	189000

2.2 Mix Proportions

Two concrete mixes of M25 grade were prepared: a control mix without colloidal nano silica and a modified mix in which colloidal nano silica replaced part of the cement. The mix design was carried out in accordance with IS 10262:2019 [30] for M25 grade. The two mixes were designated CC (control concrete) and CCCNS (concrete with colloidal nano silica). The proportions of both mixes are given in Table 2.

The colloidal nano silica was introduced at a dosage of 2.5% by mass of cement, a value within the range reported in the literature as optimum for nano silica in concrete and established as the optimum colloidal silica content for compressive strength in earlier work [23, 24]. This dosage refers to the mass of the colloidal suspension, and the associated solids and water contents were accounted for as described below. A water-to-binder ratio of 0.46 was maintained in both mixes,

and a superplasticizer (Auramix 400) was added at 0.5% by mass of cement, with the dosage determined through trial mixes to achieve the required workability.

Table 2. Mix proportions of control and colloidal nano silica concrete

Mix ID	Cement (kg/m ³)	Nano silica suspension (kg/m ³)	Fine aggregate (kg/m ³)	Coarse aggregate (kg/m ³)	W/B ratio	Total water (kg/m ³)	Superplasticizer (% by mass of cement)
CC	350	0	725	1180	0.46	161	0.5
CCCNS	347.38	8.75	725	1180	0.46	161	0.5

The colloidal nano silica was supplied as an aqueous suspension with a solid nano silica content of 30% by mass, the remaining 70% being water. At a dosage of 2.5% by mass of cement, the suspension added to the CCCNS mix was 8.75 kg/m³, comprising 2.625 kg/m³ of nano silica solids and 6.125 kg/m³ of water. The cement content was reduced by the mass of the nano silica solids, giving 347.38 kg/m³, so that the total binder content of 350 kg/m³ was the same as in the control mix. To maintain the same water-to-binder ratio of 0.46, the water carried by the suspension was deducted from the added mixing water. The added mixing water for the CCCNS mix was therefore 154.875 kg/m³, and together with the 6.125 kg/m³ contributed by the suspension, the total water content was 161.0 kg/m³, equal to that of the control mix. Thus, the total binder content, total water content, and water-to-binder ratio were kept constant between the two mixes, allowing the effect of colloidal nano silica to be evaluated without introducing differences in these overall mixture parameters.

2.3 Beam Specimens

A total of twelve scaled-down under-reinforced beams measuring 100 × 150 × 1360 mm, with an effective span of 1200 mm and a clear cover of 10 mm, were cast and water-cured for 28 days. They comprised four conditions, plain control (CC), plain colloidal nano silica (CCCNS), reinforced control (RCC), and reinforced colloidal nano silica (RCCCNS), with three specimens per condition. In the reinforced specimens, two 8 mm diameter Fe550 grade bars were placed at the bottom face as longitudinal tensile reinforcement, and two bars of the same diameter were placed at the top to serve as hanger bars for stirrup anchorage. Shear reinforcement was provided in the form of 6 mm diameter closed stirrups at a uniform spacing of 100 mm center-to-center along the entire beam length. The unreinforced specimens contained no reinforcement of any kind. The resulting tension reinforcement ratio was 0.74%, which places the reinforced sections in the under-reinforced range so that failure is initiated by yielding of the tension steel and is ductile.

The beams were tested under four-point bending over the simply supported span, with the two-point loads applied symmetrically at a spacing of 400 mm. This divides the span into three equal parts, giving a shear span of 400 mm and a shear span to effective depth ratio of 2.94, which ensures a flexure-dominated response with negligible shear influence. The maximum aggregate size was retained at 10 mm rather than scaled so that the concrete matrix and the bond and cracking behavior at the steel-concrete interface remained representative of a normal structural concrete. As all specimens shared the same geometry, reinforcement, and aggregate, any size effect applied equally to the four conditions and did not affect the comparison between the control and colloidal nano silica beams. The casting arrangement and cross-sectional details of the reinforced beams are shown in Fig. 1 and Fig. 2, respectively, and the specimen designations are summarized in Table 3.

Table 3. Beam specimen details

Specimen	Mix	Type	Dimensions (mm)
CC	CC	Plain	100 × 150 × 1360
CCCNS	CCCNS	Plain	100 × 150 × 1360
RCC	CC	Reinforced	100 × 150 × 1360
RCCCNS	CCCNS	Reinforced	100 × 150 × 1360



Fig. 1. Experimental setup for casting concrete beams

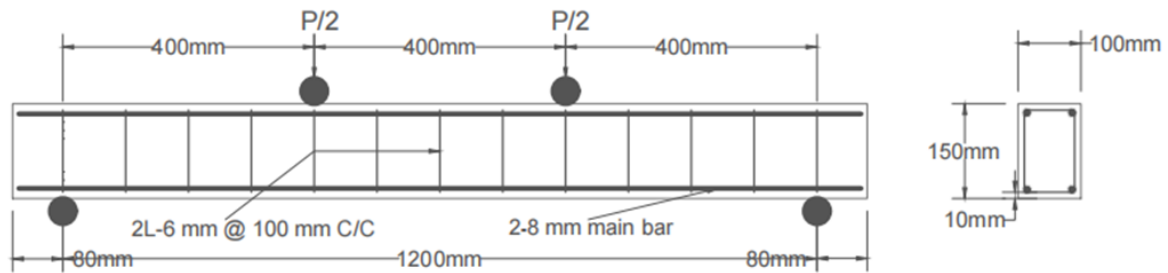


Fig. 2. Geometric dimensions of beam specimens and Reinforcement detailing

2.4 Test Setup and Instrumentation

2.4.1 Material Characterization Tests

Compressive strength and split tensile strength tests were performed on specimens cast from both mixes to obtain the mechanical properties required for numerical modelling and analytical calculations. Cylindrical specimens measuring 150 × 300 mm were used to determine the compressive stress–strain response and elastic modulus, as well as the split tensile strength, while 150 mm cube specimens were tested to determine the cube compressive strength. Three specimens were tested for each mix and each test at 28 days, and the reported values represent the mean of the three specimens tested.

2.4.1.1 Uniaxial Compressive Strength Test

The compressive test was conducted on 150 × 300 mm cylindrical specimens in accordance with IS 516 (Part 1/Sec 1):2021 [31] to obtain the stress–strain response and elastic modulus of both concrete mixes. The specimens were tested in a compression testing machine under load-controlled conditions, with the load applied continuously and without shock at a stress rate of 14 MPa/min, corresponding to approximately 247 kN/min for the 150 mm diameter specimens. Axial deformation was measured over a 200 mm gauge length using a compressometer fitted with a dial gauge having a least count of 0.01 mm. Load and corresponding displacement readings were recorded at every 10 kN increment throughout the test. The elastic modulus was determined from the slope of the initial linear portion of the stress–strain curve. The stress–strain data, together with the post-peak response defined following the CEB-FIP Model Code [32], were used as the compressive input for the concrete damaged plasticity model in Abaqus. In addition, 150 mm cube specimens were tested in compression in accordance with IS 516 (Part 1/Sec 1):2021 [31] to

determine the cube compressive strength, with three cubes tested for each mix. The test setup is shown in Fig. 3(a).

2.4.1.2 Split Tensile Strength Test

The split tensile test was conducted on 150×300 mm cylinders in accordance with IS 516 (Part 1) [31] by applying a diametral compressive load along the length of the specimen. The measured split tensile strength was reduced by a factor of 0.9 to obtain the uniaxial tensile strength input for the concrete-damaged plasticity model. The test setup is shown in Fig. 3b. The mechanical properties of both mixes at 28 days, together with their standard deviation and coefficient of variation, are summarized in Table 4.



Fig. 3. (a) Uniaxial compressive strength test setup (b) Split tensile strength test setup

Table 4. Mechanical properties of CC and CCCNS mixes at 28 days

Property	CC	CCCNS	Change (%)
Compressive strength of Cube (MPa)	40.67 ± 1.57	52.00 ± 3.80	27.9
Cylinder compressive strength, f_c (MPa)	33.0 ± 2.55	43.9 ± 3.11	33
Elastic modulus, E (GPa)	21.0	24.0	14.3
Split tensile strength, f_{sp} (MPa)	2.735 ± 0.081	2.994 ± 0.041	9.5
Tensile strength for CDP, $f_t = 0.9 f_{sp}$ (MPa)	2.46	2.69	9.5

Note: Strength values are the mean of three specimens ($n = 3$); $\pm$ values denote the standard deviation. Coefficients of variation are 7.7% (CC) and 7.1% (CCCNS) for compressive strength, and 3.0% (CC) and 1.4% (CCCNS) for split tensile strength.

2.4.2 Flexural Strength Test

All beam specimens were tested under four-point bending in a simply supported configuration on two roller supports, in accordance with IS 516 (Part 1) [31], using a 100-tonne universal testing machine. The load was applied through a spreader beam and measured with a 500 kN load cell having a resolution of 1.69 kN per division. Midspan deflection was measured with a dial gauge of 0.01 mm least count positioned at the soffit of the beam. The test was conducted under load control, with the load applied monotonically at a rate of 4 kN/min, and the load and corresponding deflection were recorded at each increment of 1.69 kN until failure. Crack initiation was observed visually, and the first crack load was recorded. The four-point bending test setup is shown in Fig. 4.

3. Numerical Analysis

3.1 Finite Element Model Setup

Finite element models were developed in Abaqus/Standard to simulate the behavior of the experimentally tested beams. The concrete beam, measuring $100 \times 150 \times 1360$ mm, was modeled as a three-dimensional solid, while the steel reinforcement bars were created as wire elements and

positioned within the concrete cross-section in accordance with the experimental reinforcement layout. The steel loading and support rollers were also modeled as three-dimensional solids. Material properties for concrete and steel were assigned based on the experimental test data described in Section 2.4. The reinforcement was embedded within the concrete using the embedded region constraint, which assumes a perfect bond between the reinforcing steel and the surrounding concrete. This assumption neglects bond slip and may influence the predicted post-yield deflection, crack distribution, and ductility of the reinforced beams and is therefore acknowledged as a limitation of the model.



Fig. 4. Flexure strength test setup for four-point bending condition

Surface-to-surface contact was defined between the beam and the rollers, with a tangential friction coefficient of 0.3 and hard contact in the normal direction. The rollers were modeled as rigid bodies and positioned at locations corresponding to the four-point bending configuration, ensuring that they maintained their original shape throughout the loading process. The support rollers were restrained to reproduce a simply supported condition, and the load was applied through the loading rollers as an imposed displacement at their reference points. The steel reinforcement cage and the assembled model with the reference points are shown in Fig. 5 and Fig. 6. The concrete was discretized using eight-node linear brick elements with reduced integration (C3D8R), and the steel reinforcement using two-node linear truss elements (T3D2). The nonlinear analysis was carried out using the Static, General procedure with the full Newton-Raphson iterative solver, and geometric nonlinearity was accounted for by activating the NLGEOM option. Automatic incrementation was used, with an initial increment of 0.01, a minimum of 1×10^{-9} , and a maximum of 0.1 of the total step time, and a maximum of 10000 increments.

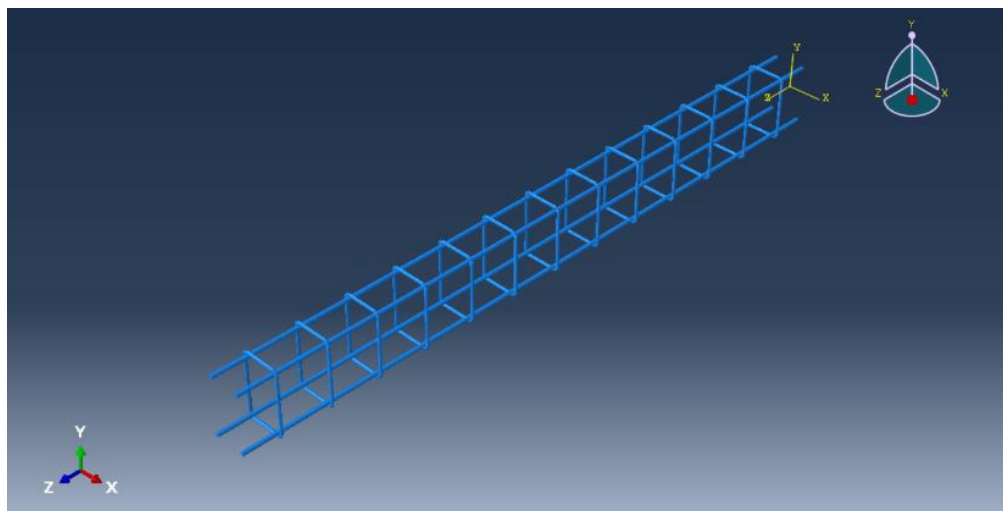


Fig. 5. Steel reinforcement cage modelled in Abaqus

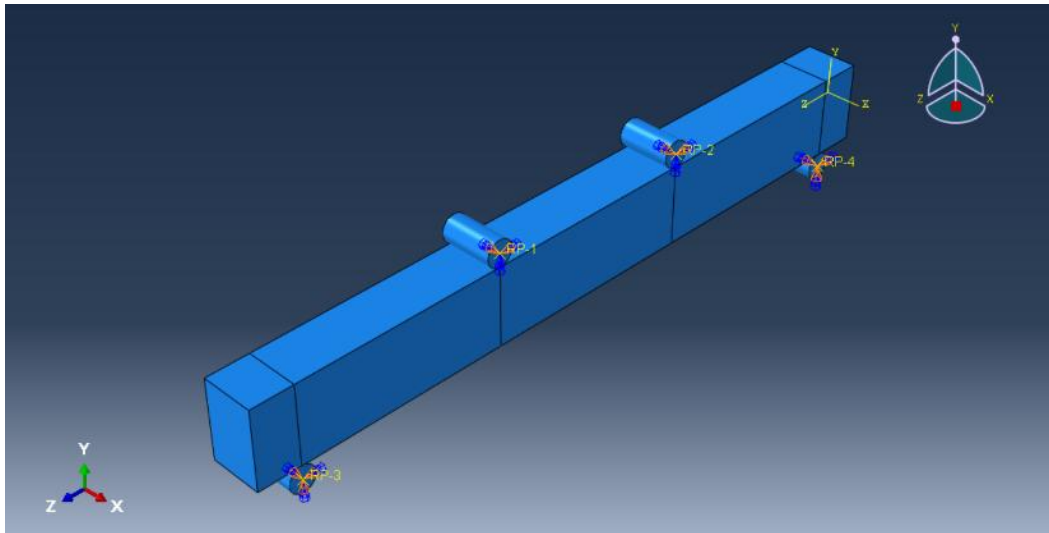


Fig. 6. Assembled finite element model with rollers and reference points

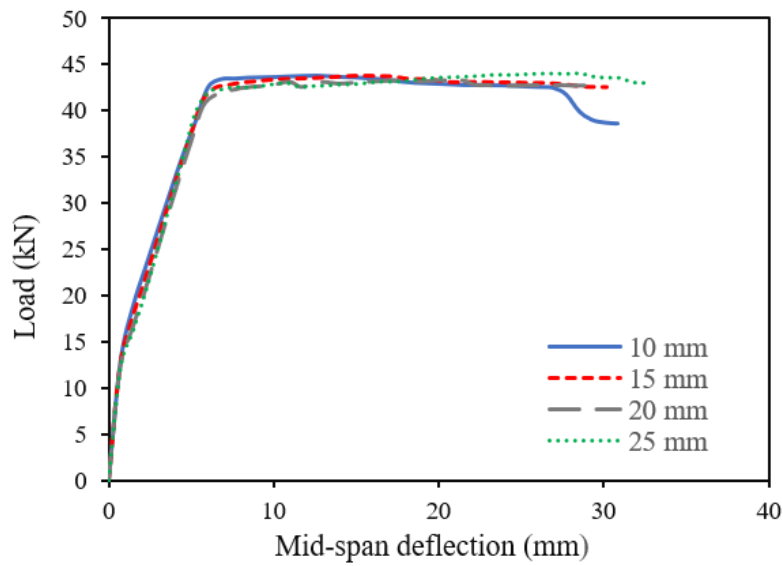


Fig. 7. Load-deflection responses for different mesh sizes

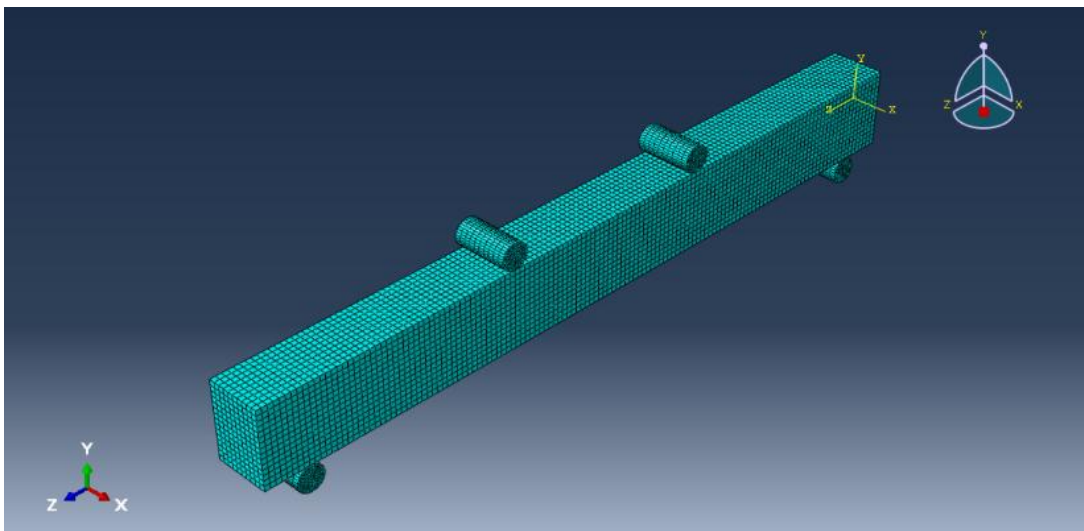


Fig. 8. Finite element mesh of a representative reinforced concrete beam

Convergence at each increment was checked using the default force and displacement tolerances of Abaqus/Standard. The element size was selected through a convergence study in which sizes of 10, 15, 20, and 25 mm were evaluated by comparing the load-deflection responses obtained from the finite element model for each mesh, with the tensile softening regularized by the characteristic length of each mesh. The responses are compared in Fig. 7. The peak load varied by less than about 3.2% across the four meshes, confirming that the global response was essentially independent of element size. Although the meshes gave similar loads and deflections, the 10 mm mesh was adopted for the final analysis because the finer mesh resolved the localization of cracking and the damage distribution more distinctly than the coarser meshes at an acceptable computational cost. The finite element mesh of a representative reinforced concrete beam is shown in Fig. 8.

3.2 Material Constitutive Models

3.2.1 Concrete — Concrete Damage Plasticity Model

The behavior of concrete in the CDP model is governed by two distinct failure mechanisms: tensile cracking and compressive crushing. Each mechanism is tracked through a scalar damage variable, d_t and d_c , respectively, that progressively degrades the elastic stiffness as loading continues [19, 20]. The model couples both mechanisms through a unified plastic-damage framework that captures the stiffness recovery observed when a cracked concrete specimen is reloaded in compression [20]. The yield surface and plastic flow potential are controlled by five parameters, which are listed together with the values adopted in this study and their sources in Table 5.

Table 5. CDP plasticity parameters adopted in the FEM model

Parameter	Symbol	Value	Source
Dilation angle	ψ	36°	[33]
Eccentricity	ε	0.1	[33]
Biaxial/uniaxial strength ratio	f^b_0/fc_0	1.16	[33, 34]
Second stress invariant ratio	Kc	0.667	[33]
Viscosity parameter	μ	0.0005	[33]

The compressive behavior was defined using the experimental uniaxial stress-strain curves obtained from the cylinder tests described in Section 2.4.1.1. The measured data defined the ascending branch up to the peak stress; the initial part of the descending branch was defined following the CEB-FIP Model Code [32], and the extended descending branch followed the relationship adopted by Pavlović et al. [35] from the Abaqus documentation [36]. The measured stress values were then converted to inelastic strain, as required by the CDP model, from the total strain as

$$\varepsilon_{cin} = \varepsilon_c - \sigma_c / E_0 \quad (1)$$

where ε_c is the total compressive strain, σ_c the corresponding stress, and E_0 the elastic modulus from the cylinder tests reported in Table 4. The compression damage variable was defined as [37].

$$d_c = 1 - \sigma_c / f_{cm} \quad (2)$$

Where; f_{cm} is the mean compressive strength. The elastic modulus for each mix was taken from the cylinder test results in Table 4, and a Poisson's ratio of 0.2 was adopted for both mixes. The compressive stress-strain curves and the compression damage evolution are shown in Fig. 9(a) and Fig. 9(b).

The tensile behavior was defined using an exponential tension-softening model. The uniaxial tensile strength f_t was derived by multiplying the measured split tensile strength by a reduction factor of 0.9, following the CEB-FIP Model Code [32], and the fracture energy G_f was estimated from the CEB-FIP Model Code [32] based on the compressive strength and the maximum aggregate size of 10 mm. The post-peak softening followed the exponential relationship of Cornelissen, Hordijk, and Reinhardt [38].

$$\sigma_t/f_t = [1 + (c_1 w/w_c)^3] \exp(-c_2 w/w_c) - (w/w_c) [1 + c_1^3] \exp(-c_2) \quad (3)$$

where w is the crack opening, $c_1 = 3$ and $c_2 = 6.93$ are material constants, and $w_c = 5.14 G_f/f_t$ is the crack opening at which the stress reduces to zero. The crack opening was converted to cracking strain through the characteristic length l_c of the element.

$$\varepsilon_t^{ck} = w/l_c \quad (4)$$

This crack-band regularization was applied separately for each mesh, using the characteristic length corresponding to that mesh, so that the same fracture energy was dissipated irrespective of element size. The tension damage variable was defined consistently with the compressive case [37] as $d_t = 1 - \sigma_t/f_t$. The tension softening response and the tension damage evolution are shown in Fig. 9(c) and Fig. 9(d)

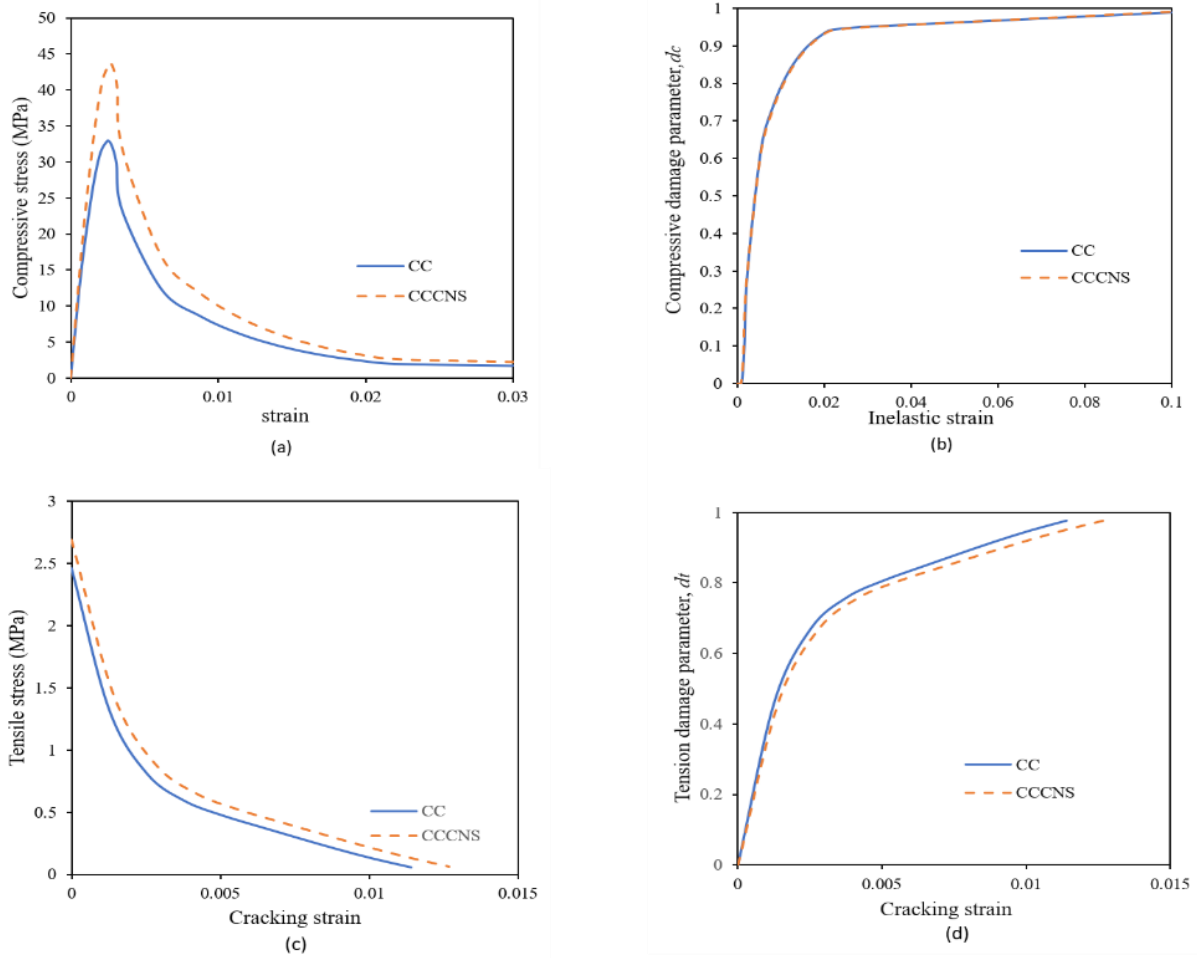


Fig. 9. Concrete constitutive input curves for the CC and CCCNS mixes: (a) compressive stress-strain, (b) compression damage, (c) tension softening, (d) tension damage

3.2.2 Reinforcing Steel

The reinforcing steel was modeled using a bilinear elastic-plastic constitutive law with strain hardening, following the idealization of Eurocode 2 [39]. The elastic modulus, yield strength, and ultimate tensile strength were taken directly from the experimentally measured values reported in Table 1, and a Poisson's ratio of 0.3 was assigned. The plastic response was defined from the measured yield point to the measured ultimate strength, with the hardening branch entered in terms of true stress and true plastic strain, and the measured elongation at fracture of 14% representing the ductility of the reinforcement.

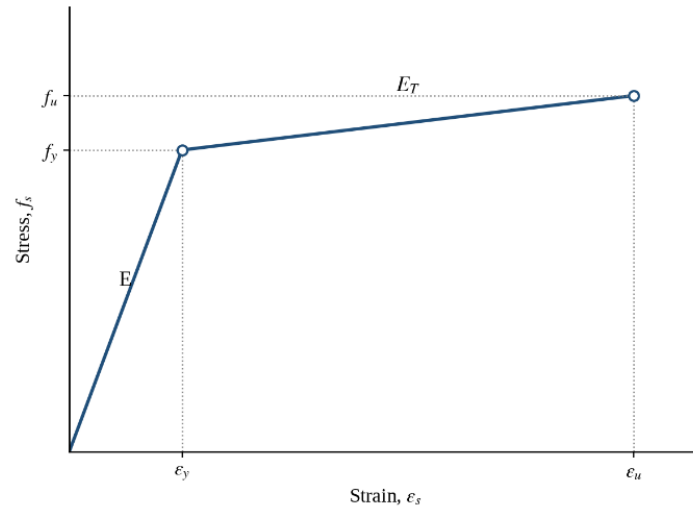


Fig. 10. Bilinear elastic-plastic stress-strain idealization of the reinforcing steel with strain hardening

4. Experimental Results and Discussion

4.1 Flexural Response of Beams

The flexural behavior of all four beam types was assessed under four-point bending at 28 days. Three specimens were tested for each type, and the ultimate loads of the individual specimens are compared in Fig. 11, in which the error bars denote the standard deviation. The first-crack, yield, and ultimate loads are reported in Table 6 as the mean of the three specimens with the standard deviation. The load-deflection responses of the specimens are shown in Fig. 12.

The two reinforced beams, RCC and RCCCNS, followed nearly identical load-deflection paths up to about 33 kN, beyond which their responses diverged. The RCC beam yielded at 33.85 kN at a mid-span deflection of 3.90 mm, whereas the RCCCNS beam yielded at 37.23 kN at 4.47 mm, an increase of about 10% in yield load. This delay in yielding is consistent with the higher tensile strength measured for the colloidal nano silica mix (Table 4). Colloidal nano silica has been reported to refine the microstructure and increase the tensile strength of concrete at the material level [5, 7, 8], which in the present beams corresponded to a higher first-crack load and delayed yielding.

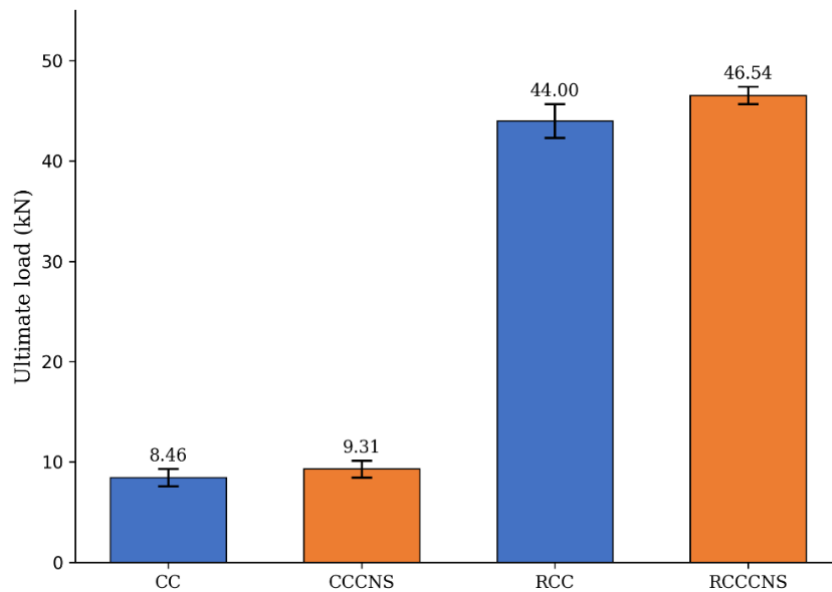


Fig. 11. Mean ultimate load of the tested beams

Table 6. Load and deflection at first crack, yield, and ultimate for the specimens

Specimen	First crack load (kN)	Deflection at first crack (mm)	Yield load (kN)	Deflection at yield (mm)	Ultimate load (kN)	Deflection at ultimate (mm)	Improvement in ultimate load (%)
CC	–	–	–	–	8.46 ± 0.85	0.58 ± 0.08	–
CCCNS	–	–	–	–	9.31 ± 0.85	0.63 ± 0.05	10.0
RCC	27.08 ± 3.38	2.65 ± 0.62	33.85 ± 1.69	3.90 ± 0.60	44.00 ± 1.69	17.90 ± 1.40	–
RCCCNS	30.46 ± 2.93	3.04 ± 0.61	37.23 ± 1.69	4.47 ± 0.36	46.54 ± 0.85	15.90 ± 0.14	5.8

Note: Plain beams (CC, CCCNS) exhibit no distinct yield point.

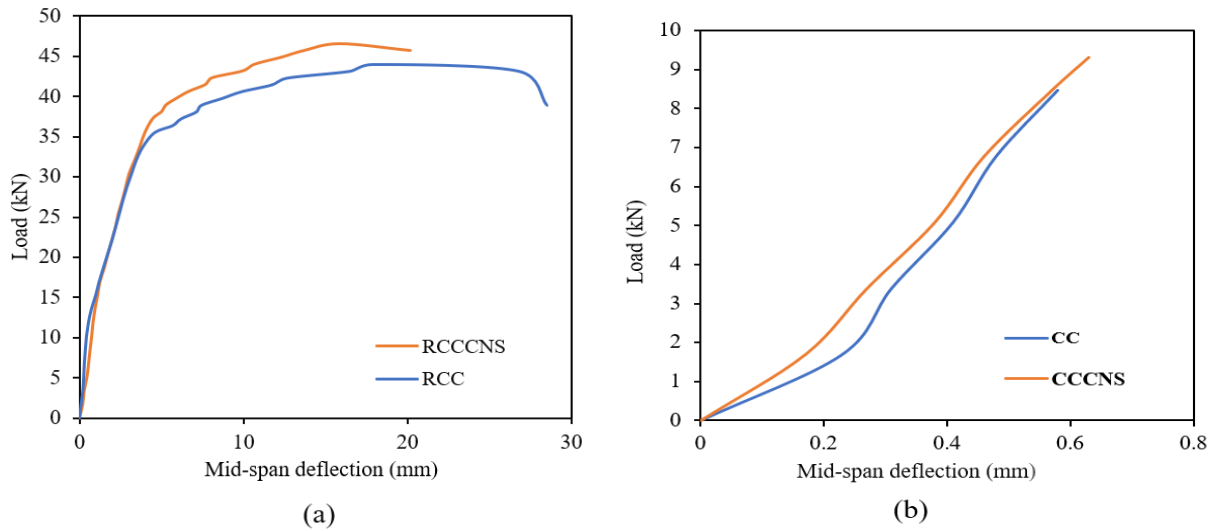


Fig. 12. Load-deflection responses: (a) reinforced beams (RCC and RCCCNS); (b) plain beams (CC and CCCNS)

Beyond yielding, the RCCCNS beam reached an ultimate load of 46.54 kN at a deflection of 15.90 mm before softening, while the RCC beam reached 44.00 kN at 17.90 mm. The corresponding increase in ultimate load was 5.8%, which is modest relative to the material-level strength gains reported in Table 4. This is consistent with the behavior of under-reinforced sections, in which the ultimate flexural capacity is governed largely by the yielding of the tension steel rather than by the compressive strength of the concrete. The contribution of colloidal nano silica was therefore more pronounced at the serviceability stage, through the delay in cracking and the reduction in deflection under working loads, than in the enhancement of ultimate capacity.

For the plain concrete beams, the flexural capacity depended entirely on the tensile strength of the concrete, as no reinforcement was present. The CCCNS beam reached a peak load of 9.31 kN at a deflection of 0.63 mm, compared with 8.46 kN at 0.58 mm for the CC beam, an increase of about 10%. The improvement imparted by colloidal nano silica therefore translated directly into the peak flexural load (Table 4). Both plain beams failed abruptly at peak load with no residual capacity, which is characteristic of unreinforced concrete in flexure.

The improvement in strength associated with colloidal nano silica has been attributed in the literature to its pozzolanic reactivity and pore-refining action, in which the nano-scale silica reacts to form additional calcium silicate hydrate and fills the capillary pores of the paste, densifying the matrix and the interfacial transition zone [5, 7, 8]. As no microstructural characterization was carried out in the present study, this mechanism is cited as a literature-based interpretation and is consistent with the higher compressive and tensile strengths measured for the colloidal nano silica

mix in Table 4. As these increases are based on three specimens per condition, they should be interpreted in light of the limited sample size and the associated experimental variability.

4.2 Energy Absorption Index

The energy absorption was evaluated as the area under the load-deflection curve up to the ultimate load, computed using the trapezoidal rule. For the reinforced beams, it was separated into a pre-yield component, from zero to yield, and a post-yield component, from yield to ultimate. Ductility was defined as the ratio of the deflection at ultimate load to the deflection at yield load. The energy values and ductility indices are given in Table 7, and the energy absorption responses of the specimens are shown in Fig. 13.

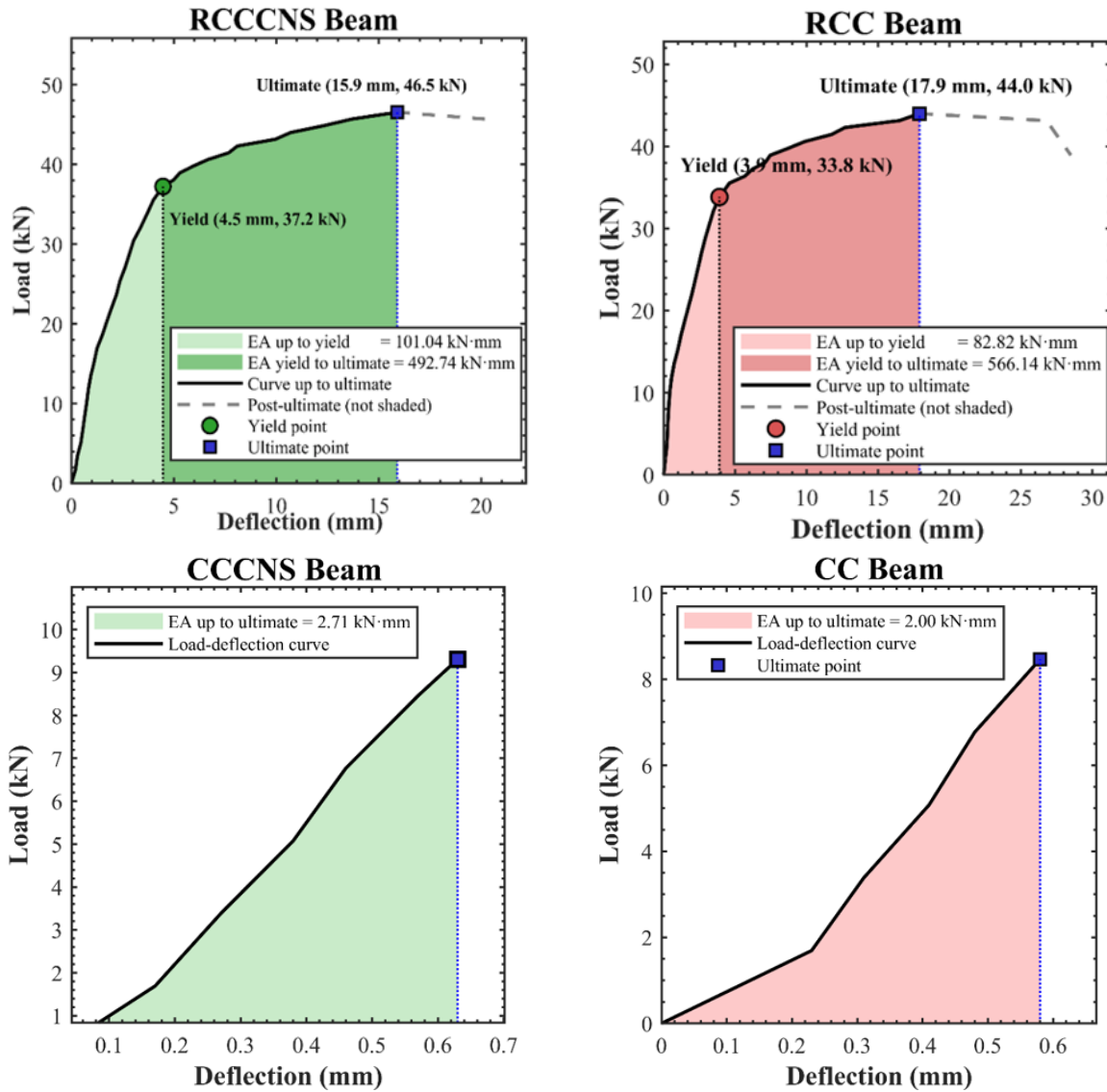


Fig. 13. Energy absorption responses of the specimens obtained from the load-deflection curves

Table 7. Energy absorption and ductility index of the specimens

Specimen	EA pre-yield (kN·mm)	EA post-yield (kN·mm)	EA total (kN·mm)	Ductility index
CC	–	–	2.00	–
CCCNS	–	–	2.71	–
RCC	82.82	566.14	648.96	4.59
RCCNS	101.04	492.74	593.78	3.55

Note: Plain beams (CC, CCCNS) exhibit no distinct yield point, so only the total energy absorption up to ultimate load is reported.

The pre-yield energy absorption of the RCCCNS beam was about 22% higher than that of the RCC beam, reflecting the higher stiffness and the delayed crack initiation of the CNS-modified concrete. Beyond the yield point, however, the trend reversed. The RCC beam absorbed about 15% more energy than the RCCCNS beam in the post-yield stage, and its total energy absorption up to ultimate load, 648.96 kN·mm, exceeded that of the RCCCNS beam, 593.78 kN·mm. Consistently, the ductility index of the RCCCNS beam, 3.55, was lower than that of the RCC beam, 4.59. These results indicate that, although colloidal nano silica increased the yield and ultimate loads and improved the pre-yield energy absorption, it reduced the post-yield deformation capacity. The RCCCNS beam reached its peak load more abruptly and deformed less beyond yield than the RCC beam. For the plain beams, the CCCNS beam absorbed about 35% more energy than the CC beam.

5. Analytical Results

5.1 Theoretical Flexural Capacity

The nominal flexural capacity of the reinforced beams was determined analytically using the limit-state method of IS 456, with the measured material strengths and without partial safety factors. The depth of the neutral axis was obtained from force equilibrium as

$$x_u = \frac{f_y A_s}{0.536 \cdot f_{ck} b} \quad (5)$$

and the nominal moment of resistance as

$$M_n = f_y A_s (d - 0.42 x_u) \quad (6)$$

The nominal flexural load under the four-point configuration is

$$P_n = \frac{2M_n}{a_v} \quad (7)$$

where f_{ck} is the measured cube compressive strength of the concrete, f_y is the yield strength of the steel, A_s is the area of tension reinforcement, b is the beam width, x_u is the depth of the neutral axis, d is the effective depth, M_n is the nominal moment of resistance, P_n is the nominal flexural load, and a_v is the shear span. The nominal capacities of the reinforced beams are compared with the measured ultimate loads in Table 8.

Table 8. Nominal and experimental ultimate loads of the reinforced beams

Beam	Nominal capacity, P_n (kN)	Experimental ultimate load (kN)	P_{exp} / P_n
RCC	38.2	44.00	1.15
RCCCNS	39.0	46.54	1.19

The nominal capacities of the two reinforced beams were close, at 38.2 kN and 39.0 kN, as the flexural capacity of an under-reinforced section is governed by yielding of the tension steel, so the higher concrete strength of the colloidal nano silica mix had only a small effect. The measured ultimate loads were higher than the nominal capacities, which is expected for a ductile under-reinforced section in which the load increases beyond yielding as the reinforcement strain-hardens from its yield strength toward its measured ultimate strength. The nominal capacity therefore represents the strength at yielding of the tension steel, and the higher measured loads are consistent with the post-yield behavior of the beams.

5.2 Load-Deflection Response and Ultimate Capacity of Beams Using FEM

The predicted and measured load-deflection curves are compared in Fig. 14. The experimental and FEM ultimate loads and deflections are compared in Table 9.

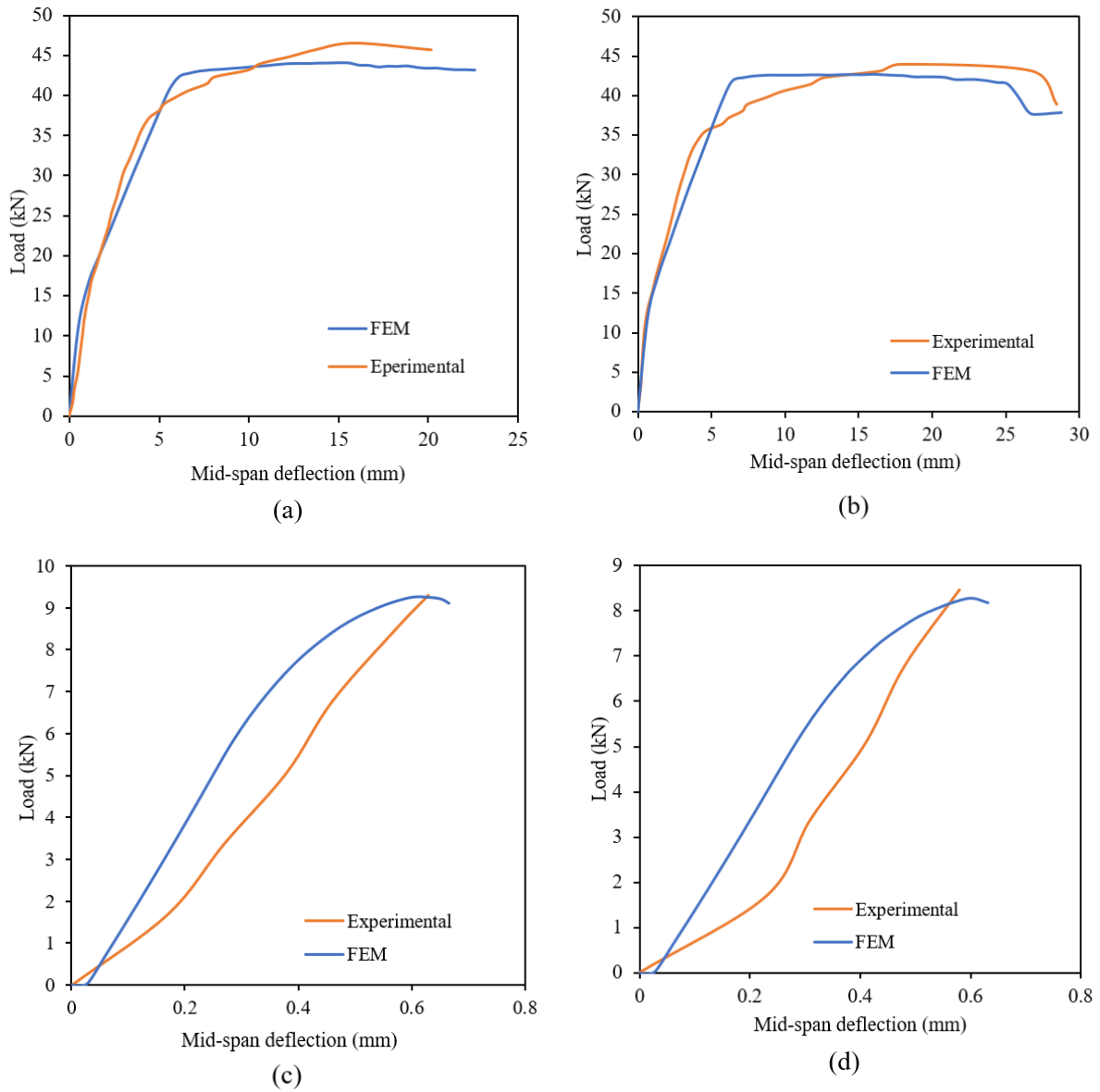


Fig. 14. Comparison of experimental and FEM load-deflection responses: (a) RCCCNS; (b) RCC; (c) CCCNS, and (d) CC beams

Table 9. Comparison of experimental and FEM ultimate load and deflection

Specimen	Exp. ultimate load (kN)	FEM ultimate load (kN)	Difference (%)	Exp. deflection (mm)	FEM deflection (mm)	Difference (%)
CC	8.46	8.27	2.23	0.58	0.60	3.45
CCCNS	9.31	9.27	0.44	0.63	0.61	3.90
RCC	44.00	42.62	3.14	17.90	16.23	9.31
RCCCNS	46.54	44.10	5.23	15.90	15.42	3.03

The finite element model predicted the ultimate load of all four beams, the differences from the measured values being 2.23%, 0.44%, 3.14%, and 5.23% for the CC, CCCNS, RCC, and RCCCNS beams. The differences in mid-span deflection at ultimate load were 3.45%, 3.90%, 9.31%, and 3.03%, respectively. The larger deflection difference in the RCC beam is consistent with the perfect bond idealization of the embedded reinforcement, which overestimates the flexural stiffness at large deformations. The deflection difference also arises from the estimated tensile softening law and fracture energy used in the concrete model, the bilinear idealization of the post-yield steel

response, and the approximate representation of tension stiffening in the smeared-crack formulation. The measured values also carry experimental uncertainty from the instrument resolution and the specimen-to-specimen variability, and the reported differences should be interpreted with this in mind.

To evaluate the model across the loading history in addition to the ultimate values, the predicted and measured loads were compared at fixed mid-span deflections of 3, 6, 9, 12, and 15 mm, as given in Table 10. Whereas Table 9 compares the ultimate load and the corresponding deflection at the peak of each response, Table 10 compares the load at fixed deflections along the loading path so that the model is assessed over the full response and not only at the ultimate point.

Table 10. Comparison of experimental and predicted load at selected mid-span deflections

Deflection (mm)	RCC Exp. load (kN)	RCC FEM load (kN)	RCC Diff. (%)	RCCCNS Exp. load (kN)	RCCCNS FEM load (kN)	RCCCNS Diff. (%)
3	29.35	25.89	11.8	30.14	27.18	9.8
6	36.91	40.61	10	39.85	42.17	5.8
9	39.98	42.56	6.5	42.72	43.48	1.8
12	41.69	42.60	2.2	44.69	43.96	1.6
15	42.82	42.61	0.5	46.19	44.1	4.5

For the RCC beam, the difference between the predicted and measured loads decreased from 11.8% at 3 mm to 0.5% at 15 mm. For the RCCCNS beam, the difference decreased from 9.8% at 3 mm to 1.6% at 12 mm and was 4.5% at 15 mm. In both beams, the differences were larger at the smallest deflection and generally smaller at the intermediate and later stages of loading. The small differences in the loads at the intermediate deflection stages, together with the difference in the deflection at ultimate load in Table 9, indicate that the model reasonably captures the measured load-deflection response but reaches its peak at a slightly smaller deflection than the experiment.

The model thus reproduced the ultimate load and the overall load-deflection behavior of both the plain and reinforced beams. On this basis, the model is considered preliminarily verified against the experimental data and limited to the tested configurations.

5.3 Crack Pattern and Failure Mode

The experimental and numerical crack patterns of the four beams are compared in Figs. 15 to 18. In both the reinforced beams, flexural cracks initiated at the soffit within the constant-moment region and propagated vertically toward the compression zone, and no diagonal cracking developed near the supports, confirming a flexural mode of failure. Failure occurred through yielding of the tension reinforcement followed by crushing of the concrete in the compression zone.

The measurable crack parameters, summarized in Table 11, were measured from the representative specimen of each reinforced beam condition and correspond to individual specimens rather than aggregated values. The control beam (RCC) developed twelve flexural cracks with a mean crack spacing of 61.6 mm and a maximum crack height of 109.6 mm, whereas the colloidal nano silica beam (RCCCNS) developed ten cracks with a larger mean spacing of 73.0 mm and a maximum crack height of 116.8 mm. The smaller number of more widely spaced cracks observed in the RCCCNS beam coincided with its higher first-crack load, indicating a difference in crack development between the two reinforced beam conditions.

Table 11. Measured crack parameters of the reinforced beams

Beam	Number of cracks	Mean crack spacing (mm)	Maximum crack height (mm)
RCC	12	61.6	109.6
RCCCNS	10	73.0	116.8

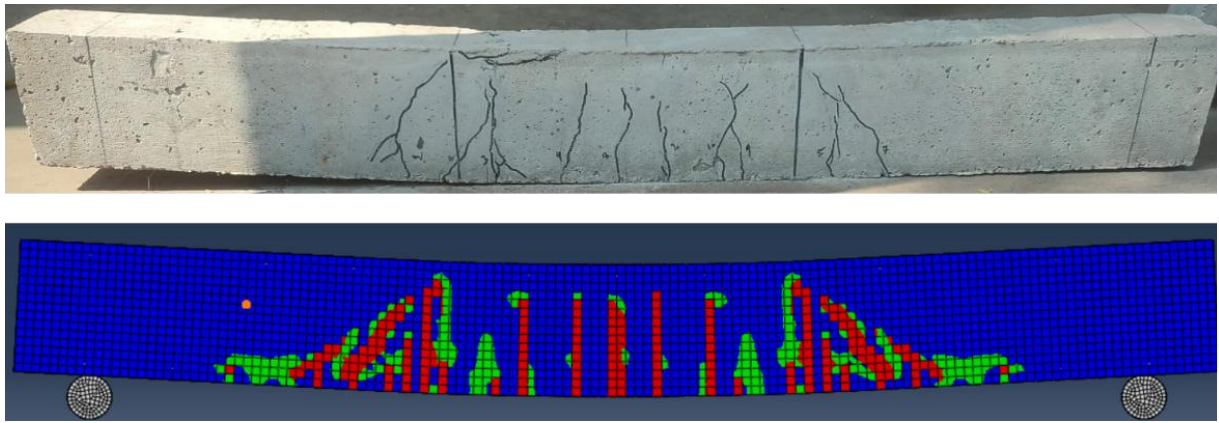


Fig. 15. Comparison of the experimental crack pattern and the numerical tension-damage field for the RCCNS beam

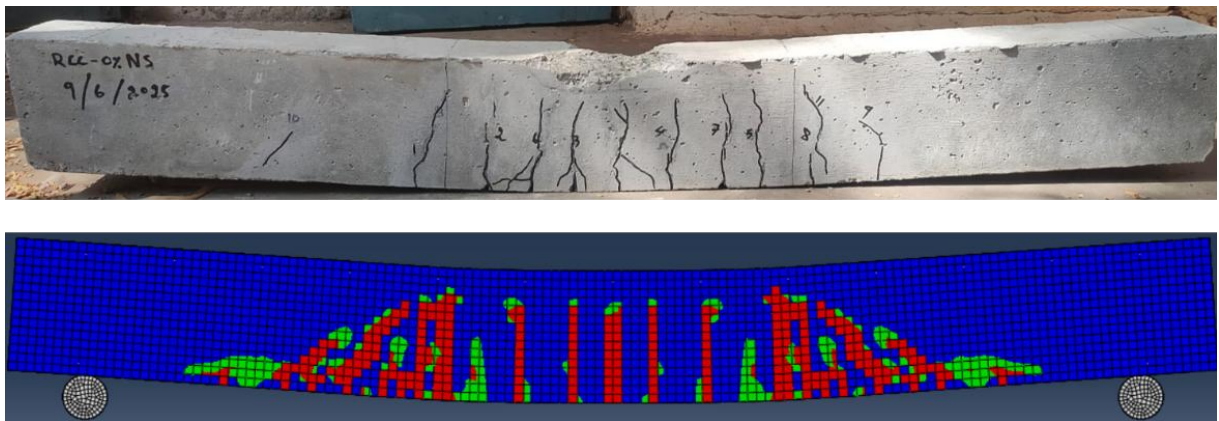


Fig. 16. Comparison of the experimental crack pattern and the numerical tension-damage field for the RCC beam

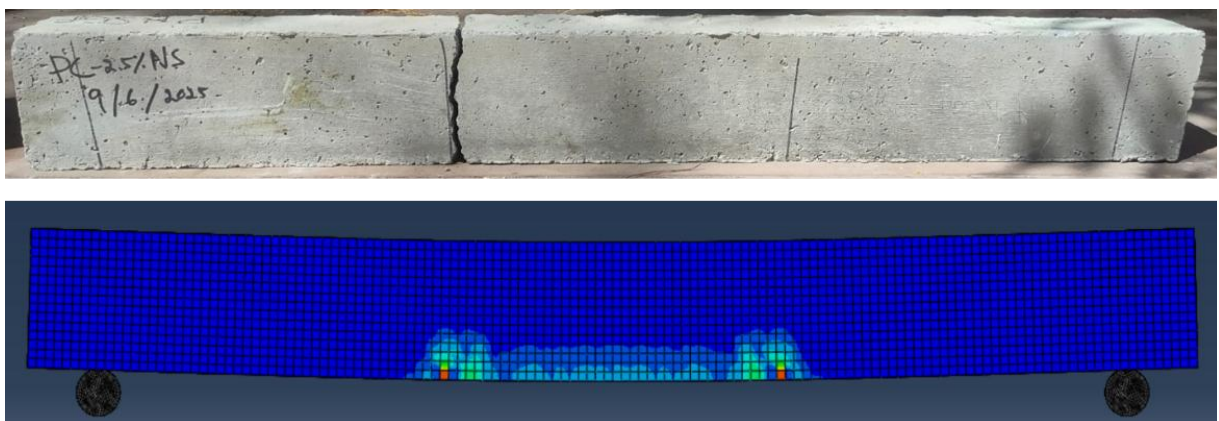


Fig. 17. Comparison of the experimental crack pattern and the numerical tension-damage field for the CCCNS beam

In the finite element model, cracking is represented by the smeared tension-damage field of the concrete damaged plasticity model, which distributes damage over the mesh as a continuous field rather than forming discrete cracks [36]. The experimental crack measurements and the numerical damage field are therefore different in nature and not directly comparable as discrete crack quantities. The crack number, spacing, and height were accordingly evaluated only from the experimental observations, while the model was compared with the experiment in terms of the location, orientation, and extent of the tension-damaged region. On this basis, the tension-damage contours were concentrated in the constant-moment region and oriented vertically from the soffit, in qualitative agreement with the observed flexural crack pattern. The comparison is therefore presented as a qualitative agreement in the damage pattern rather than as a validation of the discrete crack characteristics. Crack width was not measured during the test program.

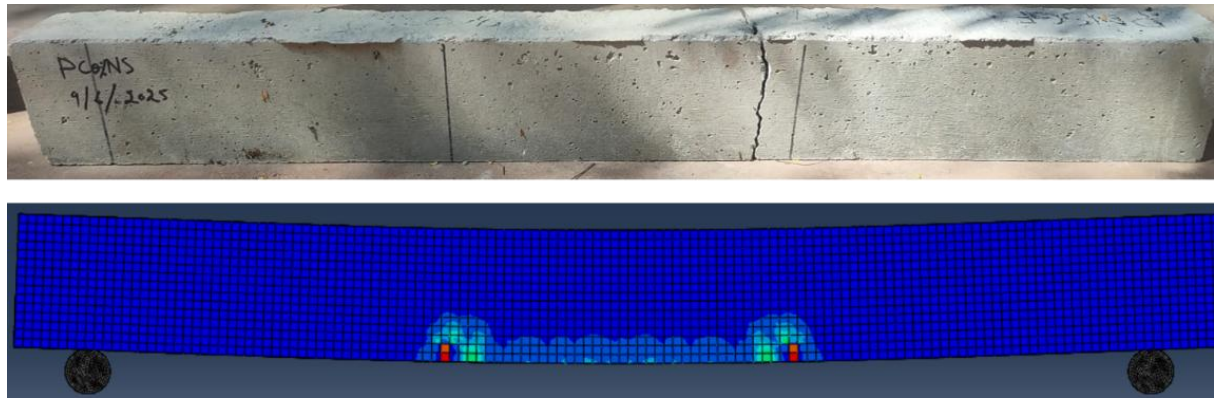


Fig. 18. Comparison of the experimental crack pattern and the numerical tension-damage field for the CC beam

For the plain concrete beams (CC and CCCNS), a single dominant flexural crack developed near mid-span and produced sudden brittle failure. In the numerical models, the corresponding tension-damage field was distributed across the constant-moment region rather than localized into a single discrete crack, which reflects the smeared representation of cracking in the concrete damaged plasticity model and is most pronounced in unreinforced concrete, where a single dominant crack governs failure [36]. The predicted location and orientation of the tension-damaged region were nevertheless consistent with the experimentally observed cracking.

6. Conclusion

Based on the experimental and finite element results from plain and reinforced concrete beams containing 2.5% CemSynXTX colloidal nano silica as partial cement replacement, the following conclusions can be drawn:

- The addition of 2.5% colloidal nano silica improved the mechanical properties of the concrete, increasing the cube compressive strength, cylinder compressive strength, elastic modulus, and split tensile strength by 27.9%, 33.0%, 14.3%, and 9.5%, respectively, at 28 days.
- Colloidal nano silica increased the flexural strength of both plain and reinforced beams. The plain colloidal nano silica beam carried about 10% more load at failure than the plain control, while the reinforced colloidal nano silica beam carried 5.8% more than the reinforced control, the smaller gain reflecting the governing role of the tension steel in the reinforced sections.
- Colloidal nano silica improved the serviceability performance of the reinforced beams, which cracked at a higher first-crack load and yielded at about 10% higher load than the control, corresponding to delayed cracking and reduced deflection under service loading.
- The addition of colloidal nano silica did not change the failure mode. Both reinforced beams failed in a ductile flexural manner through progressive cracking, yielding of the reinforcement, and concrete crushing, whereas both plain beams failed in a brittle manner at peak load.
- Colloidal nano silica improved the pre-yield performance of the reinforced beams at the expense of their post-yield ductility. The reinforced colloidal nano silica beam absorbed about 22% more energy up to yield than the control but exhibited a lower ductility index, indicating a trade-off between stiffness and deformation capacity.
- The finite element model reasonably reproduced the flexural behavior of the tested beams. The differences in predicted ultimate load were 2.23%, 0.44%, 3.14%, and 5.23%, while the differences in mid-span deflection were 3.45%, 3.90%, 9.31%, and 3.03% for the CC, CCCNS, RCC, and RCCCNS beams, respectively. The load–deflection responses showed comparable trends with the experimental observations, and the crack patterns showed qualitative agreement in location and orientation. The finite element model was therefore preliminarily verified for the beam configurations investigated in this study.

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