

Article

Computational Insights into the Use of Polymer Cement Mortar for Negative Moment Strengthening in RC T-Beams

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Abstract

This study provides computational insights into the flexural strengthening of reinforced concrete (RC) T-beams in the negative moment region using steel-reinforced polymer cement mortar (PCM) overlays. A validated three-dimensional nonlinear finite element (FE) model was developed using the Advanced Tool for Engineering Nonlinear Analysis (ATENA) software (version 2023.0.0.22492) to simulate the behavior of beams retrofitted with 40 mm thick PCM layers embedded with 13 mm and 16 mm deformed bars. Model validation was performed against previously published experimental results reported by the authors, demonstrating excellent agreement, with normalized mean square error (NMSE) values expressed as fractions between 0.0001 and 0.0022, and experimental-to-numerical ultimate load ratios ranging from 0.99 to 1.01. Parametric analyses were then conducted to investigate the influence of key variables, concrete compressive strength, PCM overlay thickness, and longitudinal reinforcement ratio on the global flexural performance. The results revealed that increasing the overlay thickness raised the ultimate load capacity by up to 15.4% and improved energy absorption by 43%. Enhancing concrete strength led to gains of up to 12.5% in load capacity and 15.8% in stiffness. Variations in reinforcement ratio had the most significant impact, increasing peak load by up to a factor of 2.02 and improving energy absorption by up to a factor of 1.49. Despite these improvements, reductions in ductility were observed across all strengthening configurations, underscoring a strength–deformability trade-off critical for seismic applications. These findings affirm the efficacy of steel-reinforced PCM overlays and provide design-oriented insights for optimizing negative moment retrofitting strategies in RC bridge girders and continuous beam systems.

Keywords: polymer cement mortar; parametric study; finite element analysis; flexural performance



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1. Introduction

Concrete has long been the material of choice for global construction, and is valued for its cost-effectiveness, ease of casting, fire resistance, and long-term durability [1–4]. However, despite these advantages, reinforced concrete (RC) structures are not immune to deterioration; for instance, beams are prone to progressive degradation under continuous exposure to environmental and mechanical stressors. Factors such as moisture ingress, thermal cycling, chemical attack, and sustained loading gradually compromise the mechanical

integrity of concrete, leading to declines in its strength, stiffness, and serviceability [5–7]. If not properly addressed, this deterioration can result in structural deficiencies that threaten safety, accelerate maintenance demands, and shorten service life, issues that are especially pronounced for aging infrastructure. Consequently, the development of effective strengthening and retrofitting strategies has become essential to restore structural performance and ensure compliance with evolving safety standards [8–11]. This pressing demand has driven significant advancements in retrofit technologies across both traditional and modern contexts.

In response, a broad range of strengthening techniques have been developed over recent decades. Traditional methods include section enlargement using concrete or steel jacketing [12–15], integration of external frame systems [16,17], and the use of bonded or unbonded prestressing to improve load capacity [18–21]. More recently, innovations in material science have given rise to advanced retrofitting solutions such as fabric-reinforced cementitious matrices (FRCMs) [22] and ultra-high-performance concrete (UHPC) overlays [23–26]. Although these approaches have expanded the options for structural rehabilitation, their practical application is often constrained by technical and logistical challenges. For instance, cross-sectional enlargement increases dead loads and may interfere with architectural systems. External frames are labor-intensive and difficult to coordinate with existing facades, especially in urban environments. FRCMs may suffer from poor adhesion to deteriorated substrates, and UHPC requires precise execution and surface treatment, both of which raise the cost and complexity.

To overcome some of these limitations, fiber-reinforced polymer (FRP) systems have garnered widespread attention due to their high tensile strength, low weight, corrosion resistance, and ease of installation, with minimal alteration to the structural form [27–35]. These systems are particularly effective in regard to enhancing tension zones or confining RC elements to improve performance under flexural, axial, or shear loads. However, epoxy-based FRP systems present notable durability concerns. Epoxy resins are vulnerable to thermal degradation, exhibit poor fire performance, and emit toxic smoke under elevated temperatures. In addition, prolonged UV exposure and environmental moisture may degrade the adhesive bond, while thermal mismatch between epoxy and concrete can lead to interfacial debonding [36–40].

Amid these concerns, cementitious alternatives that are more compatible with concrete substrates have received growing interest. One such solution is polymer cement mortar (PCM), a hybrid composite that combines the structural benefits of cement with the performance-enhancing properties of polymer additives. PCM has demonstrated promising behavior in retrofitting applications, owing to its superior bonding capability, freeze–thaw resistance, low permeability, and improved dimensional stability [41–47]. These characteristics are largely attributed to the polymer modifiers, which improve flexibility, reduce shrinkage, and enhance adhesion, attributes not typically observed in conventional mortar systems [48–56].

Defined by its hybrid composition, PCM promotes strong adhesion to concrete substrates, a critical property for maintaining bond integrity in strengthening applications [57–60]. The inclusion of polymers fosters the formation of a dense, cohesive microstructure during curing, especially under dry or variable environmental conditions, which further improves durability and interfacial performance [61,62]. In practice, PCM is often used to strengthen RC structures such as dams, bridge decks, and pavements by embedding reinforcement and applying overlays through spraying or troweling [63–65]. Experimental studies have consistently confirmed its ability to enhance flexural capacity and stiffness [66], reduce deformation and stress concentrations [67], and improve fatigue resistance under repeated loading [63].

As a complement to experimental work, finite element (FE) modeling has become an essential tool for investigating the nonlinear behavior of RC structures, particularly in zones where direct measurements are limited or intrusive [68–72]. FE simulations enable a detailed understanding of stress–strain development, damage propagation, and system-level response under various loading conditions. For instance, Haryanto et al. [73] used the Advanced Tool for Engineering Nonlinear Analysis (ATENA) software (version 2023.0.0.22492) [74] to study RC T-beams strengthened with bonded steel wire ropes in the negative moment region, and found strong agreement between simulated and experimental outcomes. Nugroho et al. [26] conducted similar work with UHPC overlays, achieving low normalized mean square error (NMSE) values of 0.002 and 0.003 for unstrengthened and strengthened beams, respectively. Furthermore, Haryanto et al. [75] modeled T-beams retrofitted with near-surface mounted (NSM) FRP rods, producing load prediction ratios of 0.95–1.04 and NMSE values as low as 0.006. These examples underscore the reliability of FE methods in evaluating retrofit strategies for tension-critical regions.

Despite the proven potential of PCM in strengthening RC members, its application in the negative moment region of RC T-beams, especially in configurations involving steel-reinforced PCM overlays under monotonic loading, remains underexplored [76]. This region is structurally demanding, due to the simultaneous presence of top-fiber tensile stresses and high shear demands near supports. As noted by Jumaat et al. [77], the negative moment region is often the governing failure region in continuous beams where bending and shear interactions peak, as depicted in Figure 1. Yet comprehensive studies examining how design parameters such as concrete compressive strength, longitudinal reinforcement ratio, and PCM overlay thickness affect flexural performance in this zone are limited.

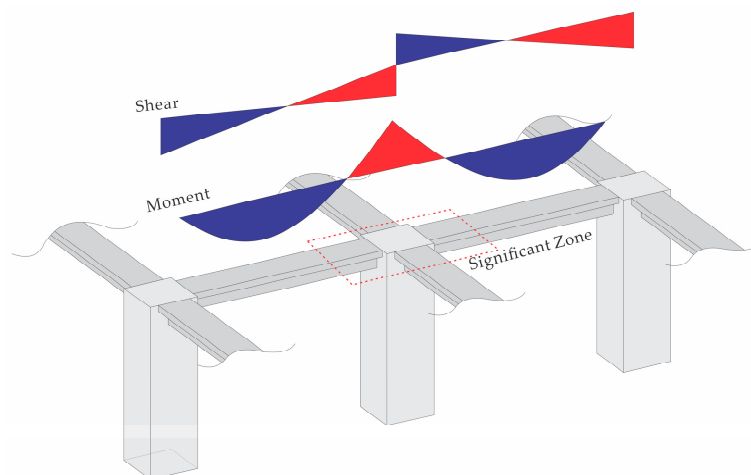


Figure 1. Idealized representation of the negative moment region in a structural element.

To address this gap, the present study involves a parametric investigation with a focus on the flexural behavior of RC T-beams retrofitted with steel-reinforced PCM overlays in the negative moment region. In view of the limitations of full-scale experimental testing, a three-dimensional FE model validated using experimental data reported by Haryanto et al. [76] is employed to simulate various design scenarios. The analysis explores the influence of the compressive strength of the concrete, PCM thickness, and reinforcement ratio, on the load capacity, stiffness, and overall structural response. Accordingly, the primary emphasis of this study is placed on structural-level performance and response characteristics rather than on material-level chemical or microstructural characterization. The insights gained are therefore intended to optimize PCM-based retrofit strategies from a structural performance perspective and to contribute to the development of practical, design-oriented guidelines for strengthening tension-critical zones in RC structures.

2. Summary of the Experimental Campaign

This section presents a summary of the experimental program reported by Haryanto et al. [76], which investigated the flexural behavior of RC T-beams strengthened with steel-reinforced polymer cement mortar overlays in the negative moment region. The experimental configuration, material properties, strengthening details, and observed structural responses are reintroduced in this study to provide a complete and self-contained reference framework for the development and validation of the finite element model.

2.1. Configuration of Test Specimens

The experiments focused on the construction and testing of three full-scale RC T-beam specimens, and were specifically designed to investigate the flexural behavior of steel-reinforced PCM overlays in the negative moment region. The dimensions of the specimen were carefully selected to reflect realistic beam proportions commonly found in building floor systems, while remaining compatible with the limitations of the laboratory testing facilities. Each beam measured 3300 mm in length and featured a cross-sectional profile comprising a 600 mm-wide flange, 120 mm flange thickness, 150 mm web width, and a total depth of 300 mm. The selection of the flange width was guided by established design provisions, which define the effective flange width as the lesser of $L/4$, the clear spacing between adjacent beams, and the actual slab width [78]. For a span length of 3300 mm, the $L/4$ criterion corresponds to 825 mm. Thus, the adopted 600 mm flange width was deemed a conservative yet realistic choice that ensured both structural representativeness and experimental feasibility. This configuration provided a reliable basis for assessing the effectiveness of PCM overlays in terms of enhancing the negative moment capacity. The overall reinforcement detailing and specimen configuration are illustrated schematically in Figure 2.

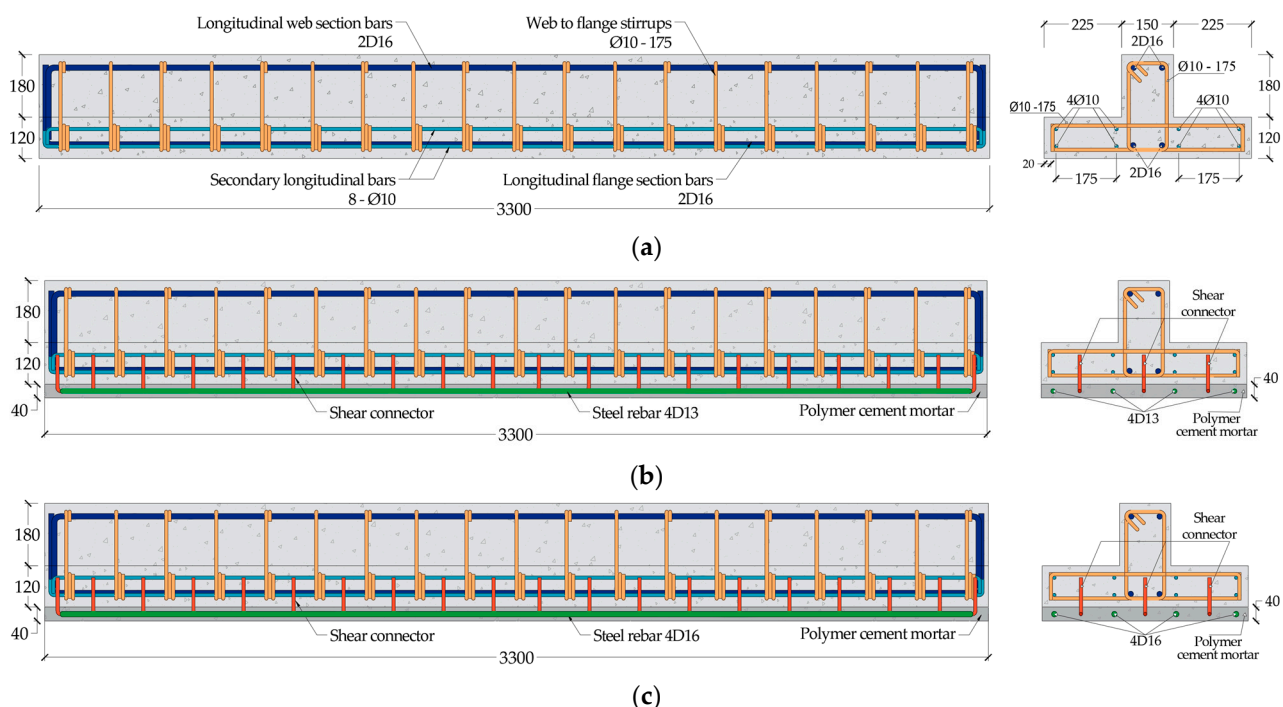


Figure 2. Test specimens in [76] (units: mm): (a) unstrengthened control specimen of UB; (b) strengthened specimen of SB-M-13; (c) strengthened specimen of SB-M-16.

The longitudinal reinforcement layout comprised two 16 mm diameter (D16) deformed bars positioned in both the tension and compression zones of the web, to ensure

flexural capacity under reversed moment conditions. The flange reinforcement consisted of eight plain round bars with a diameter of 10 mm ($\varnothing 10$), evenly spaced at 175 mm. This reinforcement level was intentionally selected to represent slab reinforcement typically present in RC T-beams used in building floor systems and continuous beams, where the slab acts integrally with the beam and contributes to negative moment resistance over supports. The adopted flange reinforcement ratio is consistent with common detailing practice for crack control and load distribution in tension-dominated slab regions, rather than being introduced as a strengthening measure. To enhance the shear resistance and prevent premature diagonal failure, $\varnothing 10$ stirrups were employed as transverse reinforcements throughout the beam length, with the same 175 mm spacing. A 20 mm concrete cover was applied over the stirrups to ensure compliance with durability and corrosion protection standards.

As part of the experimental program, one specimen was designated as the unstrengthened control beam (UB), which served as a baseline to represent the flexural response of an RC T-beam without retrofitting. The other two specimens were strengthened in the negative moment region using PCM overlays. In both cases, a 40 mm-thick PCM layer was applied along the full length of the top tensile flange, a value that is consistent with effective retrofit thicknesses reported in previous studies [79–86]. To investigate the influence of the embedded reinforcement within the overlay, two bar diameters were adopted: the first specimen (SB-M-13) incorporated 13 mm deformed bars, while the second (SB-M-16) used 16 mm bars. Both bar sizes were embedded with careful consideration to ensure effective force transfer between the PCM overlay and the existing flange, with specific anchorage and detailing as described in a subsequent section. This configuration enabled a comparative assessment of how the reinforcement content influences the flexural performance of PCM-strengthened beams.

2.2. Specimen Fabrication Process

Figure 3 shows the installation layout and the details of the shear connectors embedded within the retrofitted beam flange. The fabrication process began with the construction of reinforcement cages, which were accurately positioned within custom-fabricated T-section molds to ensure dimensional consistency and facilitate uniform casting. Spacer blocks were installed to achieve the specified concrete cover and to maintain alignment of the reinforcements along the length of the beam.

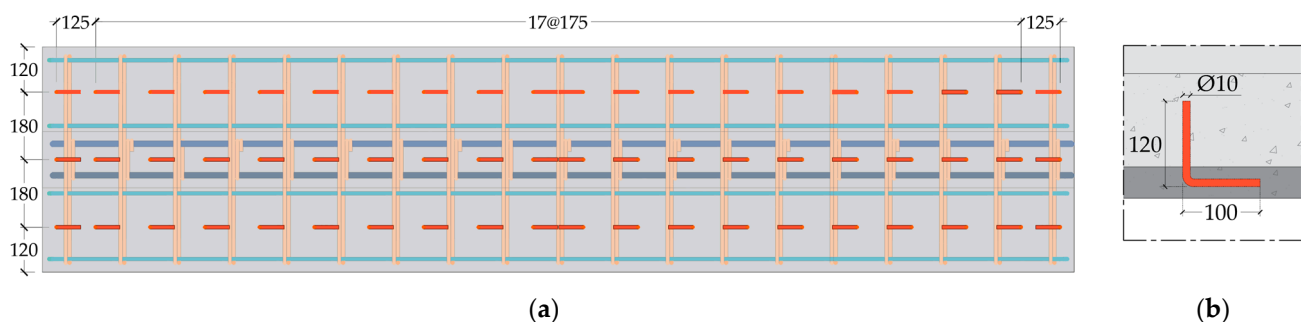


Figure 3. Application of shear connector in [76] (units: mm): (a) placement of shear connectors on strengthened beams; (b) details of the shear connector.

To replicate realistic retrofit scenarios that are commonly encountered in existing structures, the strengthening intervention was deliberately delayed until the beams had undergone a full 28-day curing cycle, thus allowing the concrete to attain its target compressive strength. Prior to overlay installation, the exposed tensile flange surface was mechanically treated using a chipping hammer to enhance the surface roughness and

improve interfacial bond performance. The aim of this preparation step was to promote mechanical interlock and ensure effective adhesion between the existing concrete substrate and the subsequently applied PCM layer.

To promote effective composite behavior between the normal-strength concrete (NSC) substrate and the PCM overlay, mechanical shear connectors were integrated along the interface to facilitate shear force transfer and ensure unified action under external loading. The nominal capacity of each connector was established following the provisions outlined in the AASHTO LRFD bridge design specifications [87]. Taking into account the expected tensile demand generated by the PCM layer and its embedded reinforcement, the required quantity of connectors was first computed for a single shear plane and then doubled to accommodate both sides of the beam flange. To ensure a uniform load distribution and to mitigate the risk of interface debonding, the connectors were installed at regular 175 mm intervals along the length of the flange. Although no direct push-off or interface shear tests were conducted in this study, the adopted design approach follows established code provisions and is consistent with previously published strengthening studies that employed similar connector configurations and reported adequate composite behavior [26,76,88].

To limit disruption to the existing internal reinforcement and minimize the risk of concrete damage during installation, 10 mm diameter steel bars were selected as shear connectors. Their modest size allowed for a reduced embedment depth, making them especially compatible with the limited thickness of the PCM overlay. To improve anchorage performance within the overlay, each connector was bent at a 90-degree angle at the embedded end, thereby enhancing mechanical interlock and improving resistance against interface slip. In order to prevent pull-out failure and ensure sufficient anchorage within the thin PCM overlay, the embedment depth of the shear connectors was designed in accordance with the anchorage provisions of ACI 318 [78], which specify a minimum embedment length equal to 10 times the bar diameter for deformed reinforcement. Accordingly, an embedment length of 100 mm was adopted, satisfying code requirements and reflecting established RC detailing practice for reliable force transfer.

To accommodate the connector bars, holes 12 mm in diameter were accurately drilled at predetermined positions along the flange surface. Prior to casting the PCM overlay, thorough surface preparation was carried out to promote optimal bonding at the interface. This included mechanical cleaning of the concrete surface and brushing of connector holes, followed by removal of residual dust and debris using compressed air. Shear connectors were installed using HIT-RE 500 V3 (Hilti), a high-performance injectable epoxy adhesive for post-installed reinforcement in concrete, with a working time of approximately 30 min and a curing period of 7 h at +20 °C in accordance with the manufacturer's specifications. The exposed steel connectors and reinforcement bars were cleaned to remove surface contaminants, and no additional chemical priming of the steel was applied. After the shear connectors were secured and the required steel bars were positioned according to the retrofit design, a cement-based bonding agent (SikaCem® Concentrate) was uniformly applied to the cleaned concrete substrate immediately prior to PCM placement. The PCM overlay was then cast over the top flange, ensuring continuous contact and effective integration with the existing concrete surface.

The high-strength PCM used for the strengthening overlays was a commercially manufactured product supplied by UBE Industries, Tokyo, Japan [89,90]. Compared with normal strength concrete, this material was selected for its enhanced tensile capacity, improved adhesion to existing substrates, and favorable cracking and shrinkage control characteristics, all of which contribute to efficient load transfer in thin overlay applications. The product carried a nominal design compressive strength of 60 MPa. However, detailed mixed

proportions, water-to-cement ratio, polymer dosage, aggregate gradation, or chemical composition, and material properties such as bonding strength and elastic modulus were not disclosed by the manufacturer. The available information was limited to the measured compressive strength, as the primary aim to investigate the comparative structural response of beams strengthened with PCM overlays; therefore, the analysis follows the material information as reported in the experimental reference [76].

Immediately after casting, the overlay was subjected to wet curing conditions to control moisture loss and facilitate proper hydration. A curing period of 28 days was applied to allow the material to develop its intended strength and durability characteristics. The overall sequence of the strengthening process is visually summarized in Figure 4.

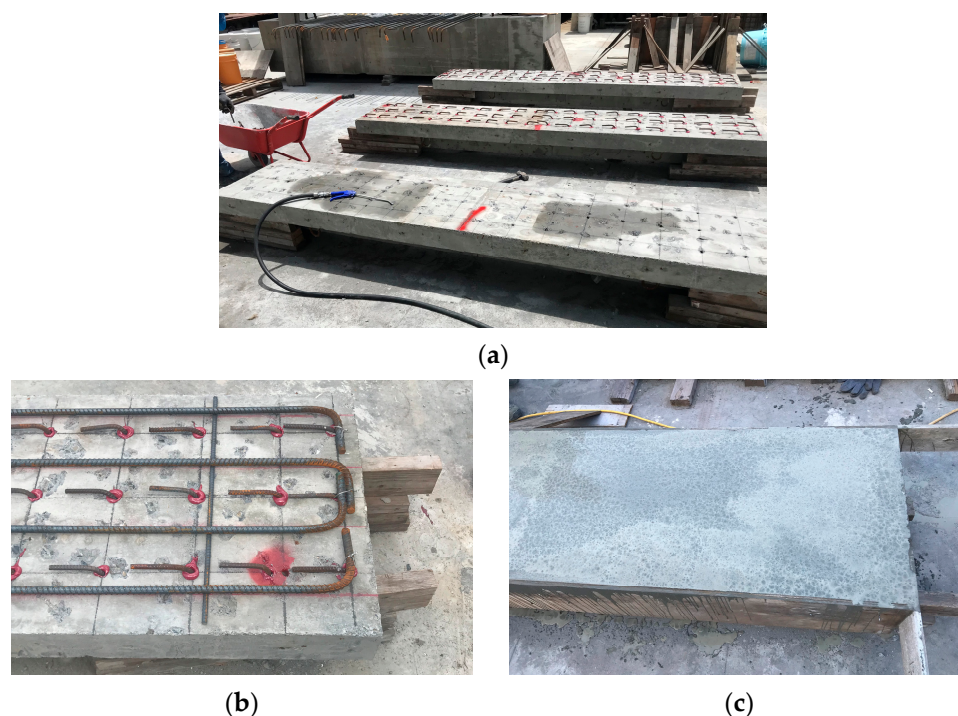


Figure 4. Strengthening procedure [76]: (a) interface preparation and shear connector installation; (b) reinforcement layer arrangement; (c) PCM casting.

2.3. Experimental Setup and Instrumentation Scheme

To replicate the structural demands experienced by RC T-beams in negative moment regions, where the flange is subjected to tension and the web experiences compression, a reverse bending test configuration was adopted. This was achieved by inverting the beams and applying a three-point bending load across a clear span of 2300 mm, as illustrated in Figure 5. Although four-point bending setups are commonly favored for producing constant moment regions, the three-point method was selected here to concentrate both the bending and shear effects at midspan. This loading condition produces a nonuniform stress field with high moment and shear gradients, which is representative of the localized stress concentration typically developed in negative moment regions near supports of continuous RC beams, particularly under service and ultimate limit states. While the stress distribution differs from the idealized, more uniformly distributed negative moment region observed in actual continuous beams, the selected configuration enables a conservative assessment of flexural–shear interaction and crack development under combined actions, thereby providing meaningful insights into the strengthening performance of the PCM system rather than a direct replication of in situ stress conditions.

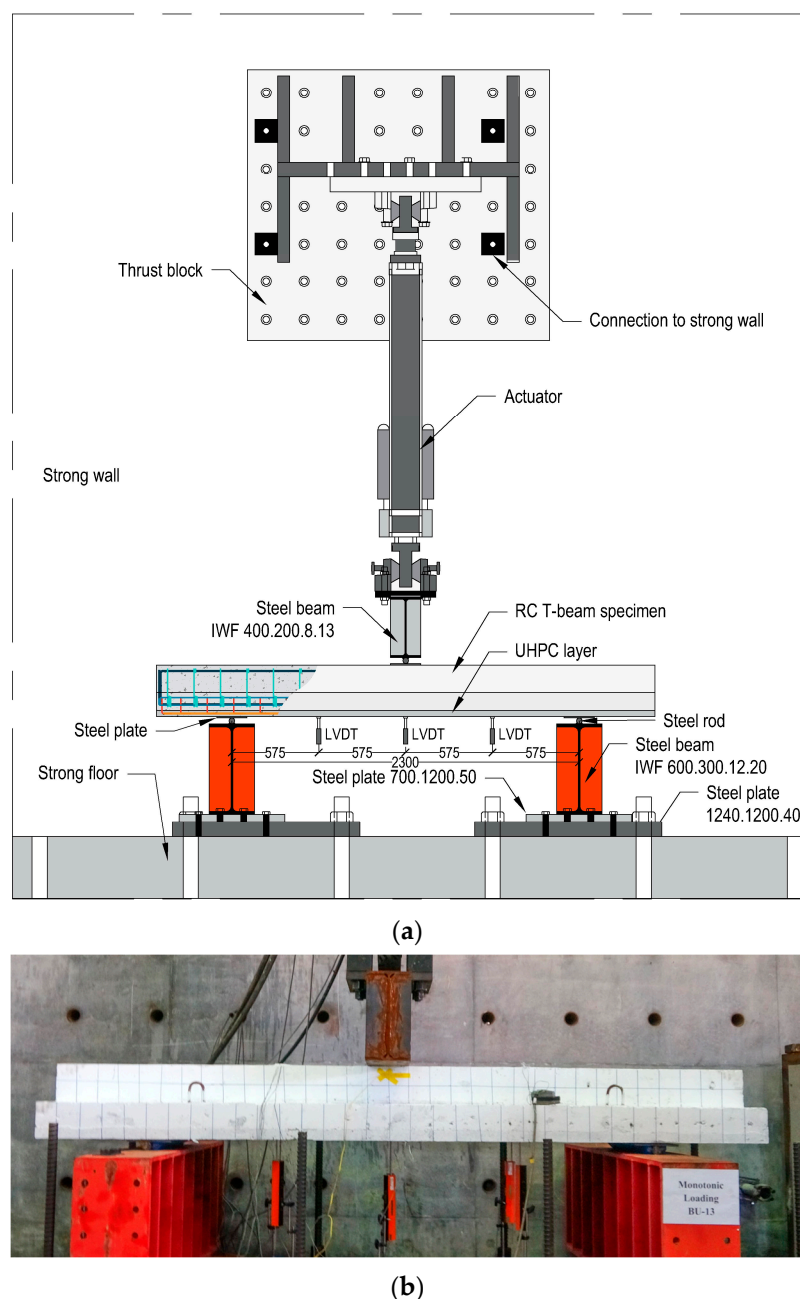


Figure 5. Experimental setup of the tested beam [76]: (a) detailed components and instrumentation (units: mm); (b) photograph of the specimen undergoing testing.

In addition, despite the fact that the PCM overlay extended beyond the nominal support zone, creating an idealized scenario that was not fully representative of in situ applications, this approach was necessary to provide a sufficient development length for the embedded reinforcement, thereby ensuring full mobilization of the tensile capacity within the overlay. Experimental loading was performed using a 500 kN servo-hydraulic actuator operating under displacement control at a constant rate of 0.4 mm/s throughout the test. As illustrated in Figure 5, beam deflections were monitored using three linear variable differential transformers (LVDTs), with one located beneath the mid-span loading point and two positioned symmetrically at distances of 575 mm from the mid-span. Load and displacement responses were recorded using a calibrated load cell and the LVDTs at an effective sampling frequency of approximately 3.5 Hz. Structural failure was defined by a 20% reduction in load-carrying capacity.

3. FE Modeling Framework

3.1. Definition of Model Geometry

The three-dimensional FE model developed in this study was implemented using ATENA, a well-established platform tailored to simulations of the nonlinear behavior of RC elements [91–93]. The numerical model was configured to closely replicate the experimental setup, including accurate geometric dimensions, material properties, and boundary conditions. To enhance computational efficiency without sacrificing predictive accuracy, geometric and loading symmetry were leveraged by simulating only one-quarter of the beam, as illustrated in Figure 6a for the unstrengthened control specimen and Figure 6b for the strengthened beams incorporating the PCM overlay. This symmetry assumption was adopted based on the symmetric geometry, reinforcement layout, loading configuration, and boundary conditions of the tested specimens, which promote predominantly symmetric global response under monotonic loading. It is acknowledged, however, that nonlinear cracking, damage localization, and interface-related phenomena may introduce localized asymmetry that cannot be fully captured by a reduced-symmetry model. Accordingly, the quarter-model formulation is intended to capture the dominant structural response and overall load–deformation behavior rather than potential asymmetric crack patterns, which represent a limitation of the present numerical approach. Future studies incorporating half- or full-beam models are therefore recommended to further assess the influence of asymmetric cracking and interface effects on the predicted response.

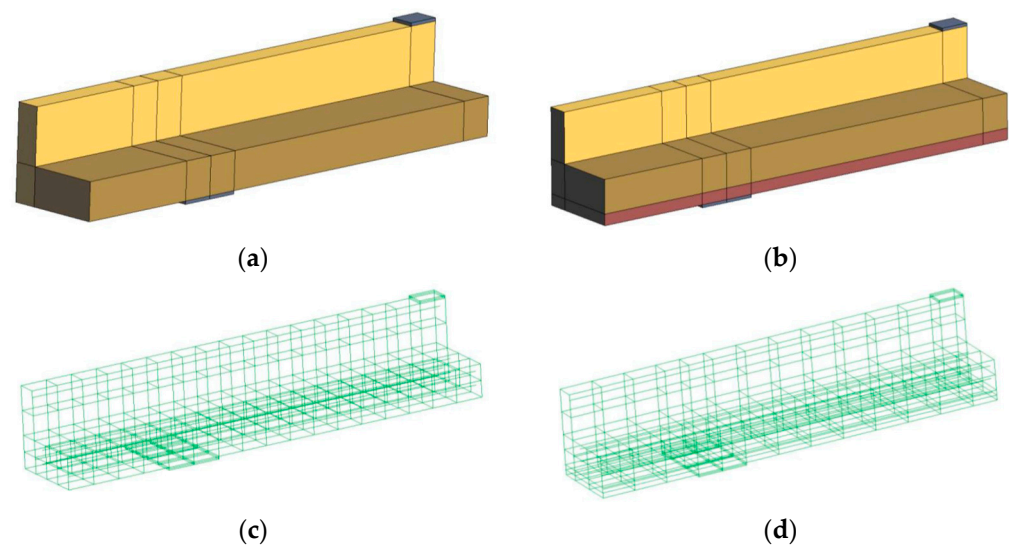


Figure 6. Finite element model configurations of beam specimens: (a) geometry of the unstrengthened control beam; (b) geometry of the strengthened beams with PCM overlay; (c) mesh distribution for the unstrengthened beam; (d) mesh distribution for the strengthened beams.

A structured mesh strategy was adopted, with element sizes determined through controlled segmentation of the beam geometry to achieve an optimal balance between computational cost and solution fidelity. This meshing approach was informed by the work of Zheng et al. [94], who noted that excessive mesh refinement offers minimal improvements in accuracy while significantly increasing the processing time. The resulting mesh provided sufficient resolution to capture localized stress concentrations and crack propagation behavior within the critical regions of interest. The meshing scheme was developed using a structured partitioning of the beam geometry along the x, y, and z axes, with the aim of optimizing the trade-off between computational cost and solution accuracy.

As illustrated in Figure 6c, the control beam model, representing the unstrengthened specimen, included six embedded linear reinforcement elements and was discretized into

182 hexahedral solid elements. For the strengthened configuration, shown in Figure 6d, two additional reinforcement elements were introduced, increasing the total to eight, and the model was discretized into 188 hexahedral elements. The adopted mesh density was sufficient to capture the global flexural response and failure mechanism of the beams under negative bending, which constitutes the primary focus of this study. Owing to the use of a smeared-crack concrete model with fracture-energy-based regularization in ATENA, key global response parameters are less sensitive to local mesh refinement than detailed crack spacing or individual crack widths. Although a formal mesh convergence study targeting localized crack development and interface stresses was not conducted, the applied discretization reliably reproduced the dominant structural behavior required for model validation and subsequent parametric analyses. Mesh generation followed a directionally refined approach, wherein segmentation along each axis governed the distribution and total number of elements.

To streamline the simulation while maintaining a reasonable representation of global structural behavior, a perfect bond was assumed at all material interfaces. This assumption was adopted based on experimental observations reported in previous studies [26,76], in which shear connectors and a bonding agent were employed and where observed local debonding or interface slip did not govern the ultimate flexural capacity or the primary failure mechanism under monotonic loading. In the present numerical investigation, neither explicit interface modeling using cohesive or contact elements nor a dedicated bond–slip sensitivity analysis was conducted.

Accordingly, the perfect-bond assumption should be regarded as a modeling idealization adopted to capture the dominant load–deformation behavior rather than to resolve interface-level failure phenomena. While neglecting interfacial debonding may result in a modest overestimation of stiffness and peak load and a smoother post-peak response, its influence on the predicted ultimate capacity is expected to be limited for strengthening configurations with adequate mechanical anchorage and confinement provided by shear connectors. The results should therefore be interpreted within the context of structural-level performance assessment, and future studies incorporating explicit interface modeling or parametric degradation of interface shear transfer are recommended to further quantify the robustness of the response under potential debonding scenarios.

3.2. Constitutive Material Modeling

3.2.1. Concrete Behavior Representation

Due to its brittle and nonlinear behavior under load, concrete is typically represented in FE analysis using damage-plasticity-based constitutive models. This approach enables accurate simulation of both compressive failure and tensile cracking by combining plasticity theory with continuum damage mechanics. Compressive degradation is modeled through inelastic strain accumulation, while tensile failure is captured using a smeared cracking formulation in which element stiffness is progressively reduced. In general, two primary modeling strategies are employed to represent cracking in concrete: discrete and smeared crack models. In discrete crack models, cracks are introduced explicitly along predefined element boundaries once a failure criterion is met. However, this technique is often sensitive to mesh alignment, which limits its flexibility in simulating arbitrary crack paths. In contrast, smeared crack models distribute the cracking behavior over a finite region by modifying the stiffness of the elements themselves, thus allowing for evolution of the crack within the continuum and offering a more mesh-independent representation of fracture processes.

In smeared crack modeling, the orientation of cracking can be governed by either a fixed or rotating crack approach, each of which is based on distinct assumptions about

post-cracking behavior. In the fixed crack method, as depicted in Figure 7a, the crack direction is assigned based on the principal stress at the moment of initial cracking, and this orientation is maintained throughout the remainder of the loading process [95]. Prior to cracking, the isotropic nature of concrete ensures that the principal stress and strain directions coincide. However, once cracking is initiated, the material response becomes anisotropic, with the damaged axis oriented perpendicular to the crack plane.

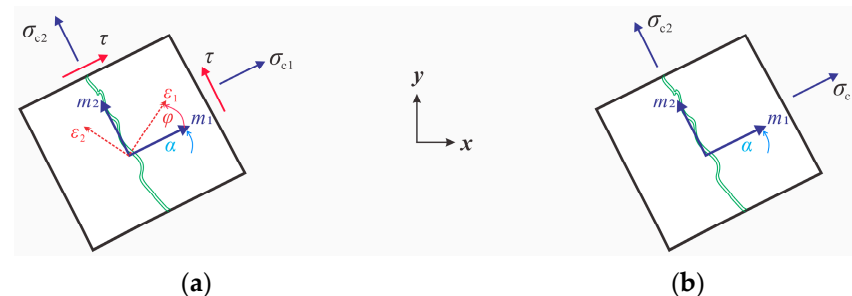


Figure 7. Diagrams illustrating smeared crack modeling approaches [74]: (a) fixed crack orientation model; (b) rotating crack orientation model.

In contrast, the rotating crack model, illustrated in Figure 7b, allows the orientation of the crack to evolve dynamically in response to changes in the principal stress direction during continued loading. In this model, ongoing alignment is maintained between the stress and strain principal axes, thereby eliminating shear transfer across the fracture surface. As a result, the model offers improved fidelity in capturing the response of concrete under complex and variable stress fields, particularly in simulations involving load reversals or multi-axial loading conditions [96]. Figure 8a presents the complete uniaxial stress–strain profile for concrete, which is divided into four distinct phases that serve as the basis for the modeling of damage progression and material softening.

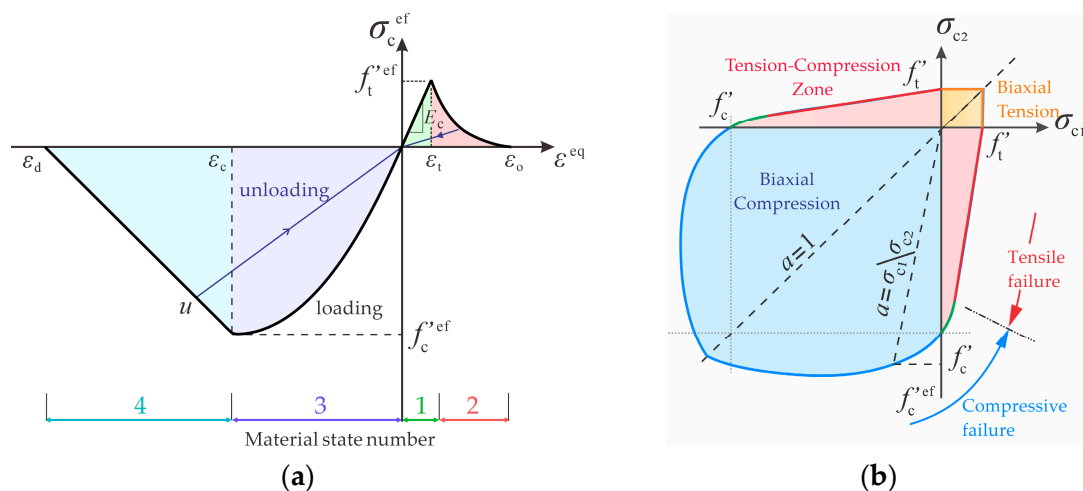


Figure 8. Constitutive models for concrete [74]: (a) uniaxial stress–strain relationship; (b) biaxial failure surface.

The compressive response was formulated following the CEB-FIP Model Code [97], which provides empirical expressions suitable for both normal- and high-strength concretes. To simulate post-peak degradation, a linear descending branch was adopted to represent the strain localization and energy dissipation characteristics that are typical of quasi-brittle behavior [95]. When subjected to biaxial stress conditions, the nonlinear material response was expressed as a relationship between effective stress and equivalent strain, denoted as

σ_c^{ef} and ε^{eq} , respectively. The equivalent strain ε^{eq} was derived from the corresponding uniaxial stress data σ_{ci} , scaled by the elastic modulus in the principal direction E_{ci} , as defined in Equation (1). This formulation is path-dependent and evolves with the loading history, particularly relative to the unloading reference point U.

$$\varepsilon^{eq} = \frac{\sigma_{ci}}{E_{ci}} \quad (1)$$

As a complement to this graph, Figure 8b, adapted from Kupfer et al. [98], depicts the variation in the peak tensile and compressive strengths under biaxial loading (f_c^{tef} and f_t^{tef}), highlighting the anisotropic and interactive nature of multiaxial stress states. These relationships are critical for capturing the directional sensitivity of the failure envelope of concrete under complex loading regimes.

A variable shear retention factor was introduced to represent the progressive reduction in shear capacity with increasing crack width [99], as illustrated in Figure 9, in order to better replicate the behavior of concrete observed in experimental tests. The model captures the degradation in the concrete's shear stiffness associated with tensile cracking. The shear retention factor, denoted as r_g , is used to modify the initial shear modulus, denoted as G_c , to create a reduced shear modulus, denoted as G . The relationships are presented in Equations (2) and (3), where G_c is derived from the elastic modulus (E_c) and Poisson's ratio (ν).

$$G = r_g G_c \quad (2)$$

$$G_c = \frac{E_c}{2(1 + \nu)} \quad (3)$$

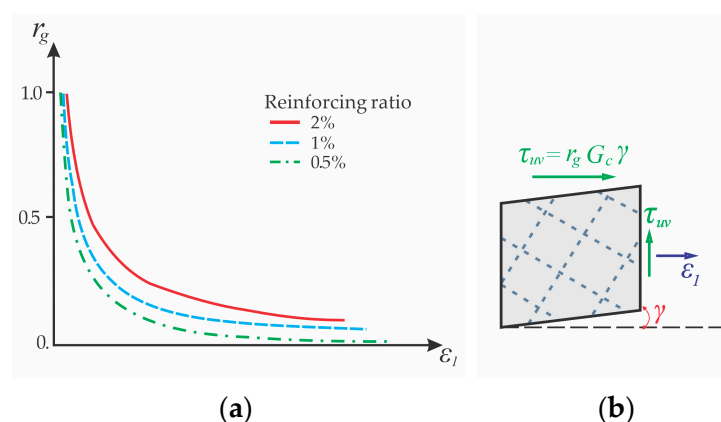


Figure 9. (a) Shear retention factor in cracked concrete with respect to tensile strain [74]; (b) Schematic representation of the cracked concrete interface [74].

The shear stress along the crack interface, denoted as τ_{uv} , is limited by the concrete's tensile strength f_t' and is computed using Equation (4), where γ denotes the imposed shear strain. As described by Vos [100], the fracture energy (G_F) captures the amount of energy dissipated during crack propagation and is formulated in Equation (5). This provides a reliable basis for characterizing the energy required to advance a tensile crack through the concrete matrix.

$$\tau_{uv} = G\gamma \quad (4)$$

$$G_F = 25f_t' \text{ (N/m)} \quad (5)$$

In this study, the response of the concrete was modeled using the CC3DnonLinCementitious2 material model, which was applied to solid elements with eight nodes. This constitutive model is particularly suitable for nonlinear, time-dependent analyses, as it accounts for the progressive

evolution of stiffness during the pre-peak hardening phase and enables dynamic updates to material properties throughout loading. The required input parameters included the compressive and tensile strengths, elastic modulus, Poisson's ratio, fracture energy, and critical compressive displacement, as summarized in Table 1. The compressive strength was obtained experimentally [76] in accordance with ASTM C39/C39M-21 [101], while the remaining parameters were determined using empirical correlations and established formulations from design guidelines [74]. It is acknowledged that ATENA's smeared-crack approach is inherently mesh-sensitive; however, in the present study, fracture energy-based crack-band regularization was employed through the material model to mitigate mesh dependency. No dedicated mesh-refinement or mesh-objectivity study was conducted, and the adopted mesh density was selected to provide a balance between numerical stability, computational efficiency, and accurate reproduction of the global load–deformation response.

Table 1. Input parameters for concrete used in the FE model.

Property	Formula	Value
Poisson's ratio [74], ν	N/A	0.2
Critical compressive displacement [74], W_d (m)	N/A	−0.0005
Shear retention factor	N/A	Variable
NSC		
Compressive strength [76], f'_c (MPa)	Test result	28.69
Elastic modulus [74], E_c (MPa)	$3320\sqrt{f'_c} + 6900$	24,679.83
Tensile strength [74], f'_t (MPa)	$0.5\sqrt{f'_c}$	2.67
Specific fracture energy [74], G_F (N/m)	$25f'_t$	66.94
PCM		
Compressive strength [76], f'_c (MPa)	Test result	60.57
Elastic modulus [74], E_c (MPa)	$3320\sqrt{f'_c} + 6900$	32,710.73
Tensile strength [74], f'_t (MPa)	$0.5\sqrt{f'_c}$	3.88
Specific fracture energy [74], G_F (N/m)	$25f'_t$	97.17

3.2.2. Modeling of Steel Reinforcement

The mechanical properties of the steel reinforcement were determined through tensile tests following ASTM A370-18 [102], as presented in Table 2, with a value for the elastic modulus of steel assumed as 200 GPa.

Table 2. Tensile test results for steel reinforcement bars.

Bar Size	Tensile Strength (MPa)	Yield Strength (MPa)
10	445.92	311.82
13	532.45	430.25
16	658.53	464.49

Within the ATENA FE framework, the reinforcement can be represented using either a discrete or a smeared modeling strategy. The discrete approach explicitly defines the individual steel bars as truss elements, whereas the smeared approach distributes the reinforcement across composite zones. In the smeared representation, reinforcement may be incorporated either as a mesh-aligned layer or as an embedded phase within concrete elements. Both approaches assume uniaxial stress conditions and use consistent stress–strain relationships to maintain numerical compatibility and reliability.

To model the axial response of the longitudinal steel bars, a two-node CCRReinforcement element was utilized. In this configuration, the compressive strength was disregarded, in view of the limited capacity arising from its anisotropic properties and localized failure

mechanisms, such as matrix cracking and fiber micro-buckling. The longitudinal reinforcement was characterized using a multi-linear constitutive law, as illustrated in Figure 10a, which consisted of four distinct stages: elastic, yielding, hardening, and fracture. The contribution of the 10 mm diameter shear connectors with bent ends employed in the experimental program to enhance interface shear transfer was represented implicitly in the FE model through a perfect-bond assumption at the PCM–concrete interface. This modeling approach was adopted to represent composite action at the structural level, without explicitly resolving local interface or connector-level phenomena.

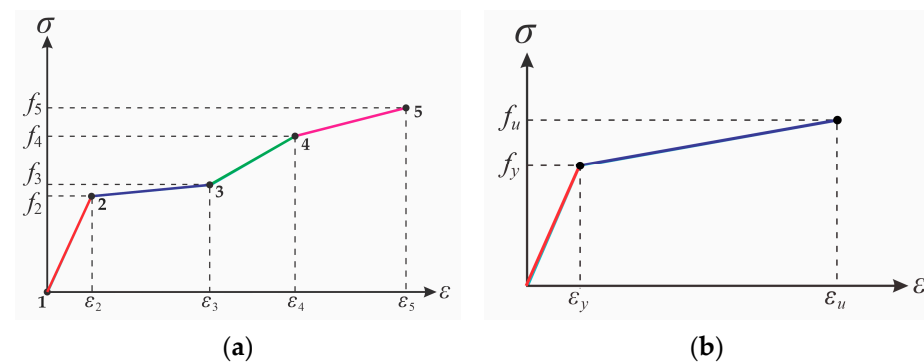


Figure 10. Constitutive stress–strain models for steel [74]: (a) multi-linear model for longitudinal steel reinforcement; (b) bilinear response with strain hardening for stirrups.

For concrete with integrated transverse reinforcement, the CCMaterial element was adopted, in which the CC3DNonLinCementitious2 concrete model was combined with the CCSmearedReinfComp stirrup formulation to give a unified material representation. This approach enables interaction between concrete cracking and smeared transverse reinforcement within the same continuum framework, allowing shear retention and post-cracking stiffness contributions to be captured in an averaged sense. The same fracture energy and crack-band regularization principles were applied consistently to these regions to ensure uniform treatment of cracking behavior across the model. Nevertheless, potential local effects related to mesh size, crack localization, and shear-retention behavior were not explicitly quantified and are recognized as a limitation of the present numerical study, warranting further investigation through systematic mesh-dependency analyses in future work.

3.3. Boundary Conditions and Response Monitoring

To accurately replicate the experimental setup, boundary conditions were carefully applied to the inverted beam model to restrain rigid-body motion while preserving the flexural response. Vertical restraint was imposed using the Constraint for Line function along the central axis of the supporting steel plate, as illustrated in Figure 11a, which prevented vertical translation at the support without restraining rotational degrees of freedom of the beam section. Negative flexural behavior was simulated by applying displacement-controlled loading at the top flange, thereby inducing tensile stresses in the slab region consistent with hogging moment conditions, as shown in Figure 11b. To improve computational efficiency, a quarter-span symmetry was adopted. This was implemented using the Constraint for Surface function, which enforced displacement compatibility normal to the symmetry planes while allowing in-plane rotations and deformations, ensuring that the reduced model accurately reproduced the global flexural behavior of the full-span beam. Additional symmetry at the mid-span cross-section was maintained by constraining translational degrees of freedom in the x- and z-directions only, as illustrated in Figure 11c,

thereby avoiding artificial rotational restraint and preventing unintended stiffening of the numerical response.

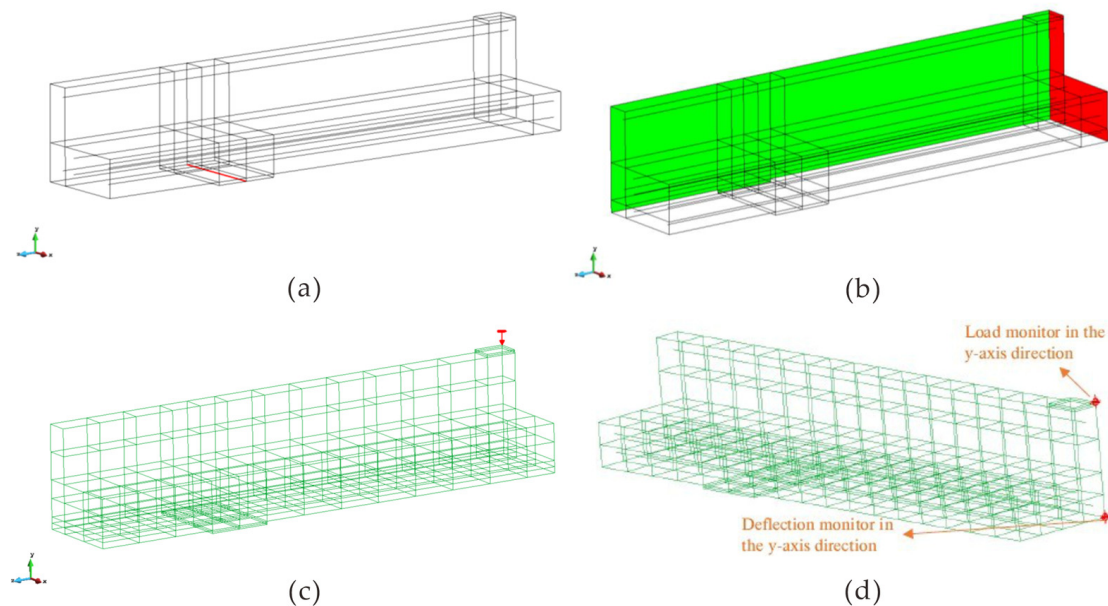


Figure 11. Implementation of boundary conditions: (a) line constraint applied along the support axis; (b) surface constraint enforcing specimen symmetry; (c) load application using the displacement-control method; (d) reaction force measured at the loading plate and vertical displacement monitored at the bottom beam mid-span. Reprinted with permission from Ref. [26], Elsevier, 2025.

3.4. Solution Strategy

To capture the nonlinear flexural behavior of the beam models with high precision, a displacement-controlled incremental analysis was adopted in conjunction with a Newton–Raphson iterative framework. This combined approach facilitated stable equilibrium iterations by minimizing the residual force imbalance between external and internal nodal reactions, as schematically represented in Figure 12.

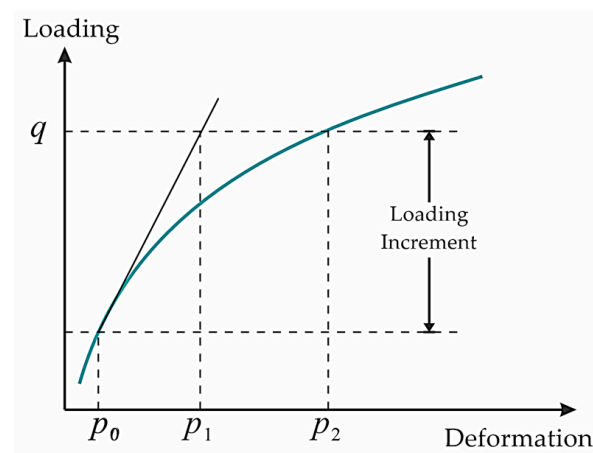


Figure 12. Schematic representation of the Newton–Raphson iteration procedure implemented within a displacement-controlled loading approach [74].

Following a series of sensitivity studies, a displacement step size of 0.05 mm was selected as the most effective, as it offered the refined resolution necessary to trace softening behavior beyond peak load while preserving the stability of convergence. Larger step sizes tended to trigger non-convergent responses or numerical oscillations, whereas smaller

increments, although more robust, imposed an unnecessary computational burden. To ensure the consistency of the solution, convergence tolerances for both force and displacement residuals were uniformly set to 0.01, representing a rational compromise between computational demand and predictive accuracy.

4. Results and Discussion

This section presents a validation-driven numerical investigation that builds upon previously published experimental results, in which experimentally validated FE models are first used to reproduce the observed structural behavior of CFRP- and PCM-strengthened RC T-beams and are then extended through a systematic, design-oriented parametric study to provide new structural-level insights into the influence of key strengthening parameters beyond the scope of the original experiments.

4.1. Summary of Experimental Findings

As described in earlier works by Nugroho et al. [26] and Haryanto et al. [76], the UB developed its first flexural crack at 34.42 kN, which was formed at mid-span. Additional cracks gradually propagated through the flange as loading progressed, and yielding was reached at 168.52 kN. As illustrated in Figure 13, the load–deflection response of the UB displayed a distinct yielding plateau, and the beam ultimately reached a peak load of 176.66 kN. Beyond this point, the load-carrying capacity began to decline as the dominant flexural crack widened and penetrated into the compression zone. Final failure occurred through crushing of the top concrete fiber, confirming a ductile flexural mode of failure but highlighting the limited strength of the unstrengthened section.

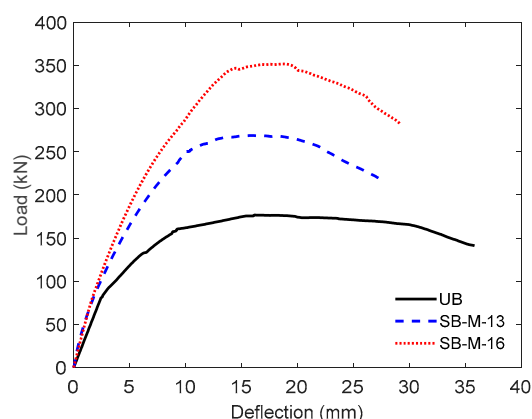


Figure 13. Load–deflection responses of the tested beam specimens [26,76]. Reprinted with permission from Ref. [26], Elsevier, 2025.

In contrast, the strengthened specimens investigated by Haryanto et al. [76] exhibited a higher load capacity. The SB-M-13 specimen resisted initial cracking until 86.04 kN, more than double that of the control beam, while SB-M-16 cracked even later, at 91.51 kN. Both specimens sustained significantly higher loads before yielding at values of 253.99 kN for SB-M-13 and 328.23 kN for SB-M-16. In terms of peak loading, SB-M-13 achieved 268.91 kN, while SB-M-16 reached 351.81 kN, representing strength gains of approximately 52.22% and 99.14%, respectively, compared to the UB. These pronounced increases reflect the combined contribution of the PCM overlay and the higher longitudinal reinforcement ratio; however, their effective mobilization is contingent on sufficient anchorage length and composite action between the overlay and the existing concrete. In the present configuration, the PCM layer was extended beyond the immediate negative moment region near the support, providing sufficient anchorage length for the embedded reinforcement and promoting

effective stress transfer between the overlay and the existing concrete. This configuration was intentionally adopted to prevent premature debonding or anchorage failure and to isolate the flexural contribution of the PCM system under negative bending.

The failure patterns of both retrofitted beams were similar. Minor cracking within the high-strength PCM layer initiated localized separation near peak load, followed by crushing of the concrete in the compression zone at mid-span, as illustrated in Figure 14. While the extended PCM length represents a somewhat idealized condition compared with typical field applications, where the overlay length may be constrained by constructability or detailing requirements, it provides an upper-bound assessment of the strengthening effectiveness when adequate development length is ensured. The strengthened specimens also exhibited improved stiffness and toughness, with marked increases in both initial and effective stiffness and energy absorption throughout loading. Although a slight reduction in ductility was observed, toughness increased by 12% for SB-M-13 and 45% for SB-M-16 at failure relative to the UB, indicating that, even under idealized anchorage conditions, the PCM overlay significantly enhanced the beams' ability to resist damage prior to collapse.

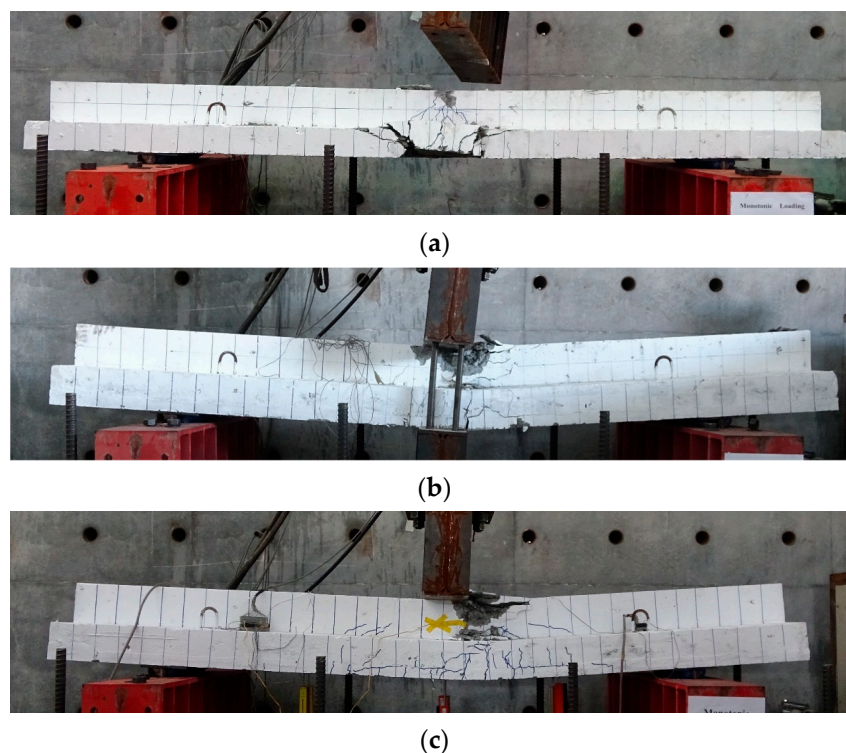


Figure 14. Observed failure modes of all tested RC T-beams; (a) specimen UB [26,76]; (b) specimen SB-M-13 [76]; (c) specimen SB-M-16 [76]. Reprinted with permission from Ref. [26], Elsevier, 2025.

Although the oxide and phase compositions of the cementitious binder used in the PCM were not explicitly characterized in this study, a prior investigation has demonstrated that the performance of polymer-modified cementitious composites is strongly governed by the relative contents of major oxides such as CaO, SiO₂, Al₂O₃, and Fe₂O₃ [103]. High CaO and SiO₂ contents promote the formation of calcium silicate hydrate (C–S–H), which governs matrix stiffness and strength, while Al₂O₃ is closely associated with ettringite formation during early hydration. The reported findings indicate that polymer incorporation can alter hydration pathways by suppressing excessive ettringite formation and stabilizing C–S–H development, thereby reducing microcracking potential and improving dimensional stability. These chemical effects contribute to enhanced durability of the interfacial transition zone between the polymer-modified overlay and the existing con-

crete substrate. Accordingly, the improved crack resistance, delayed crack initiation, and enhanced stiffness observed in the PCM-strengthened beams are consistent with oxide-controlled hydration mechanisms reported in the literature, even in the absence of direct chemical characterization in the present study.

Furthermore, scanning electron microscopy (SEM) observations and associated microstructural analyses reported in a previous study provide valuable insights into the mechanisms underlying the macroscopic behavior observed in this work. Yu et al. [104] demonstrated that polymer-modified mortars exhibit not only continuous or reticular polymer films coating hydration products, but also a refined interfacial transition zone (ITZ) between the repair layer and the concrete substrate. Their SEM-based observations revealed reduced pore connectivity, suppression of large capillary pores, and improved matrix densification within both the bulk mortar and the ITZ. In addition, microcrack analysis showed that polymer films effectively bridge microcracks and redistribute stresses, limiting crack initiation and restraining crack propagation across the interface. Compared with unmodified cement matrices, polymer-modified systems exhibited lower macroporosity, fewer microcracks, and enhanced cohesion between hydration products, particularly when polymers with strong film-forming capability were employed. These combined effects of pore refinement, ITZ densification, and crack-bridging action contribute to improved stress transfer and crack control at the structural scale. Consequently, the increased stiffness, higher load capacity, and enhanced energy absorption observed in the PCM-strengthened RC T-beams in the present study are in good agreement with the SEM-based microstructural, interfacial, and porosity-related evidence reported by Yu et al. [104], supporting the proposed strengthening mechanism despite the absence of direct SEM or microstructural characterization in this experimental program.

4.2. Validation Model

Model validation is a critical step in establishing the credibility of FE simulations, as it ensures that the numerical framework can reliably replicate the physical behavior observed in experimental settings. In addition to verifying accuracy, the process enables refinement of the input parameters, identification of the limitations of the model, and alignment with established design standards. In this study, validation was performed through a two-stage approach using independent experimental datasets. In the first phase, the numerical model was benchmarked against the global flexural response of CFRP-strengthened RC T-beams described by Haryanto et al. [29,35], thus confirming the model's ability to capture load–deflection behavior with high fidelity. The second phase involved a comparison with experimental results reported by Haryanto et al. [76], further demonstrating the model's robustness and providing a reliable foundation for the subsequent parametric analyses.

4.2.1. CFRP-Strengthened RC T-Beams Reported by Haryanto et al. [29,35,75]

In a study focusing on the negative moment region of RC T-beams, Haryanto et al. [29,35,75] examined the effectiveness of near-surface mounted carbon-fiber-reinforced polymer (NSM-CFRP) rods for flexural strengthening. Their experimental program involved three full-scale beams, each with standardized dimensions: length 2600 mm, flange width 600 mm, web width 150 mm, flange thickness 120 mm, and an overall depth of 300 mm. To simulate realistic reinforcement detailing, the longitudinal system comprised two D16 deformed bars symmetrically placed in both the top (compression) and bottom (tension) zones of the web. In addition, eight plain round bars (D10) were embedded along the flange to replicate the slab reinforcement commonly found in composite T-beam constructions. Transverse shear capacity was addressed by installing D10 closed stirrups at a uniform 175 mm

spacing, extending through both the web and flange to provide continuous confinement and effective shear resistance across critical regions.

To implement the NSM strengthening approach, two of the beam specimens were retrofitted with a pair of 8 mm diameter CFRP rods (Sika® CarboDur® BC8) embedded longitudinally within pre-cut grooves along the top flange, to replicate the tensile reinforcement that is typically active in negative moment regions. Following a standard 28-day curing period, shallow slots were precisely carved into the concrete surface to accommodate the CFRP rods, thus ensuring proper alignment and embedment. To enhance the shear resistance of the retrofitted specimens, externally bonded U-shaped CFRP strips (SikaWrap®-231 C), each 100 mm in width, were applied to both web faces using a compatible epoxy adhesive. These transverse wraps were uniformly spaced at 130 mm intervals along the span to provide additional confinement and reduce the likelihood of shear-induced failure mechanisms. All CFRP materials and adhesive systems used in the strengthening process were supplied by PT. Sika Indonesia. An overview of the retrofitting configurations and geometric layout is presented in Figure 15.

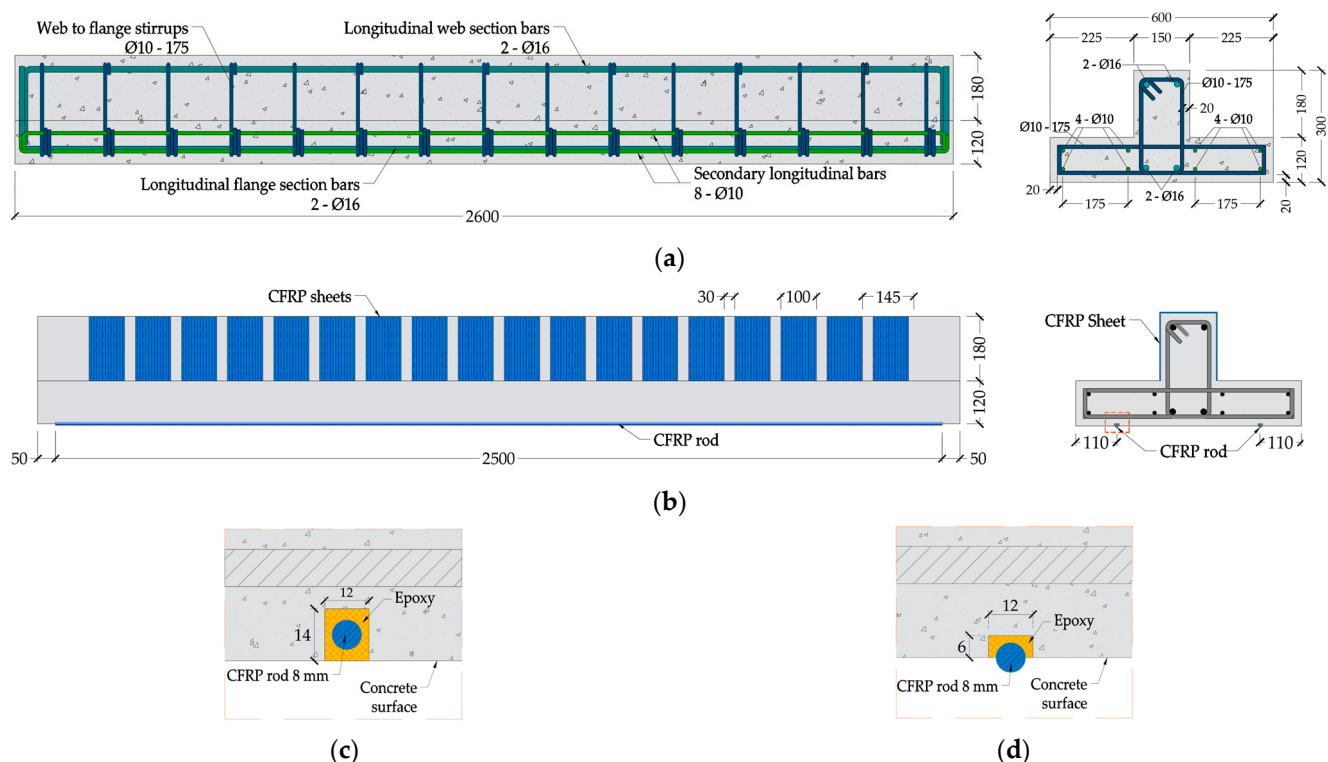


Figure 15. Configuration of RC T-beam specimens tested by Haryanto et al. [29,35,75] (units: mm): (a) control beam; (b) NSM-CFRP-strengthened beam; (c) beam with fully embedded CFRP rods; (d) beam with half-embedded CFRP rods. Reprinted with permission from Ref. [29], Elsevier, 2022.

To enable a comparative evaluation, two strengthened beam configurations were prepared. The first specimen (SF) was retrofitted with CFRP rods embedded in full compliance with standard detailing provisions, ensuring complete coverage and proper anchorage. The second specimen (SH) was intentionally designed with a reduced embedment, in which the rods were inserted to only half of their diameter and left without a protective cover layer, to replicate the practical limitations often encountered during field retrofits. Both strengthened beams were tested alongside an unstrengthened control specimen (CB) to examine the structural consequences of insufficient embedment on the overall flexural performance.

To optimize the computational performance without compromising the fidelity of the results, the FE model was constructed to represent only one-quarter of the beam, thus taking advantage of both the geometrical and loading symmetry. This strategic simplification reduced the total number of elements, and significantly minimized the processing time. The nonlinear behavior of plain concrete was simulated using the CC3DnonLinCementitious2 material model, which was applied through eight-node solid elements capable of capturing complex failure mechanisms. In regions that included transverse reinforcement, the CCCombinedMaterial element was adopted, an advanced eight-node formulation that integrates concrete and smeared stirrup effects into a single composite response, thereby enabling multidirectional interaction. The axial responses of both the embedded longitudinal steel reinforcement and the CFRP rods were modeled using the CCReinforcement element, a two-node truss-type element that was specifically developed to capture uniaxial behavior in embedded reinforcements. The externally bonded CFRP sheets were simulated with CCIsoShellBrick elements, to represent thin orthotropic composite layers with shell-like behavior. Finally, the loading and support plates were modeled using the CC3DElastIsotropic element, a four-node brick formulation with multiple integration points that is suitable for replicating the linear elastic responses of isotropic materials under mechanical loads.

Figure 16 presents the comparative load–deflection responses obtained from experimental testing and FE simulations for the three CFRP-strengthened RC T-beam configurations. For the control specimen (CB), the numerical model effectively captured the overall load trajectory, including the initial stiffness, yield point, and subsequent hardening, with only minor deviations in the post-yield plateau. This consistency highlights the model’s ability to replicate the flexural performance of unstrengthened T-beams under negative moment conditions. In the case of the strengthened beams, there was strong alignment between the numerical and experimental curves for both configurations, SH and SF. The SH specimen, representing a constrained retrofit scenario with reduced embedment of CFRP rods, demonstrated response trends that were accurately captured by the model throughout all loading phases. Similarly, the results for the SF specimen, strengthened using fully embedded CFRP rods in accordance with design guidelines, showed excellent agreement between the simulation and test data. This consistency across both the limited and ideal retrofit configurations validates the robustness of the model in terms of simulating diverse strengthening strategies. Across all cases, the FE approach consistently reproduced the key flexural characteristics, thus reinforcing its applicability for parametric investigations and performance-driven evaluations of CFRP-strengthened T-beams in negative moment regions.

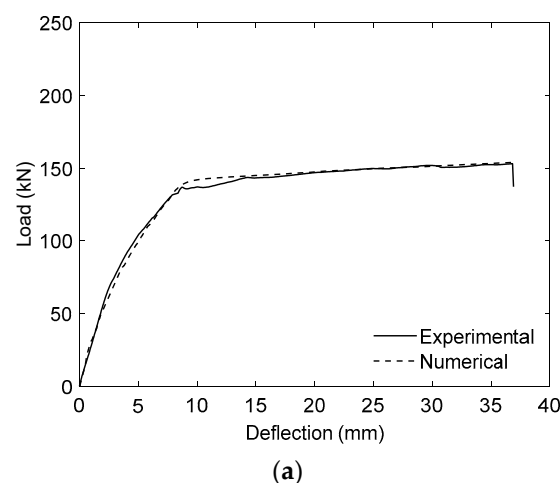


Figure 16. Cont.

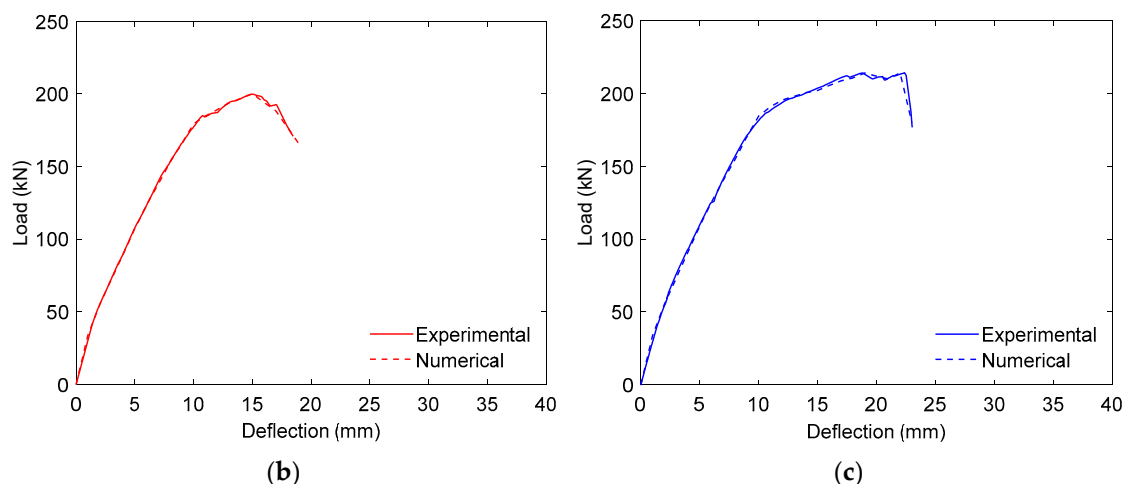


Figure 16. Comparison between experimental and FE results for CFRP-strengthened RC T-beams [29,35,75]: (a) specimen CB; (b) specimen SH; (c) specimen SF. Reprinted with permission from Ref. [29], Elsevier, 2022.

Table 3 provides a statistical evaluation of the ultimate load predictions derived from both the experimental tests and finite element simulations. The peak load ratios between the numerical and experimental results exhibit exceptional consistency, with an average value of precisely 1.00. This near-perfect alignment is further underscored by a remarkably low standard deviation of 0.01 and a coefficient of variation of just 0.58%, reflecting minimal dispersion across the dataset. These findings confirm the high fidelity of the FE model in terms of replicating the structural capacity under negative moment conditions, and reinforce the validity of the numerical framework for advanced parametric investigations. As a complementary measure of the fidelity of the model, the NMSE was employed to quantify the statistical agreement between the simulated and experimental load responses [105–107].

Table 3. Comparison between experimental and numerical results for CFRP-strengthened T-beams.

Specimen	Ultimate Load (kN)		Ratio	NMSE
	Experimental [29,35,75]	Numerical		
CB	153.14	150.00	1.01	0.0003
SH	199.80	200.12	1.00	0.0001
SF	214.13	214.14	1.00	0.0001
Average			1.00	
Standard deviation			0.01	
Coefficient of variation (%)			0.58	

The NMSE was evaluated using load values sampled at uniform 2 mm intervals of mid-span deflection, enabling a point-by-point comparison between the numerical and experimental load–deflection responses over the entire loading history. The NMSE is expressed as a dimensionless fraction and was calculated using the formulation given in Equation (6), where lower values indicate closer agreement between numerical predictions and experimental measurements. For each sampling point i , F_i denotes the experimental load and f_i represents the corresponding numerical load.

$$NMSE = \frac{\sum_i F_i^2 (1 - f_i/F_i)^2}{\sum_i F_i f_i} \quad (6)$$

As can be seen from Table 3, the NMSE values remain consistently low, indicating minimal deviation between the two datasets. It is important to note that this level of agreement was achieved without calibration of material parameters to match the test results, as all material properties were adopted a priori based on experimental measurements and established design formulations described in Section 3.2. The consistently low dispersion therefore reflects the robustness of the adopted modeling framework in capturing the dominant structural response under negative moment conditions, supporting its suitability for subsequent design-oriented parametric investigations.

Figure 17 presents a qualitative comparison of crack development in the CFRP-strengthened RC T-beams, illustrating the influence of different embedment configurations on the overall flexural damage behavior. It should be noted that, given the use of a smeared-crack formulation, the numerical results are intended to capture the dominant cracking mechanisms and structural response rather than exact geometric correspondence of individual cracks. In the control beam (CB), the model predicts relatively distributed cracking in the flange and web regions, reflecting a more ductile flexural response and gradual stiffness degradation in the absence of external strengthening.

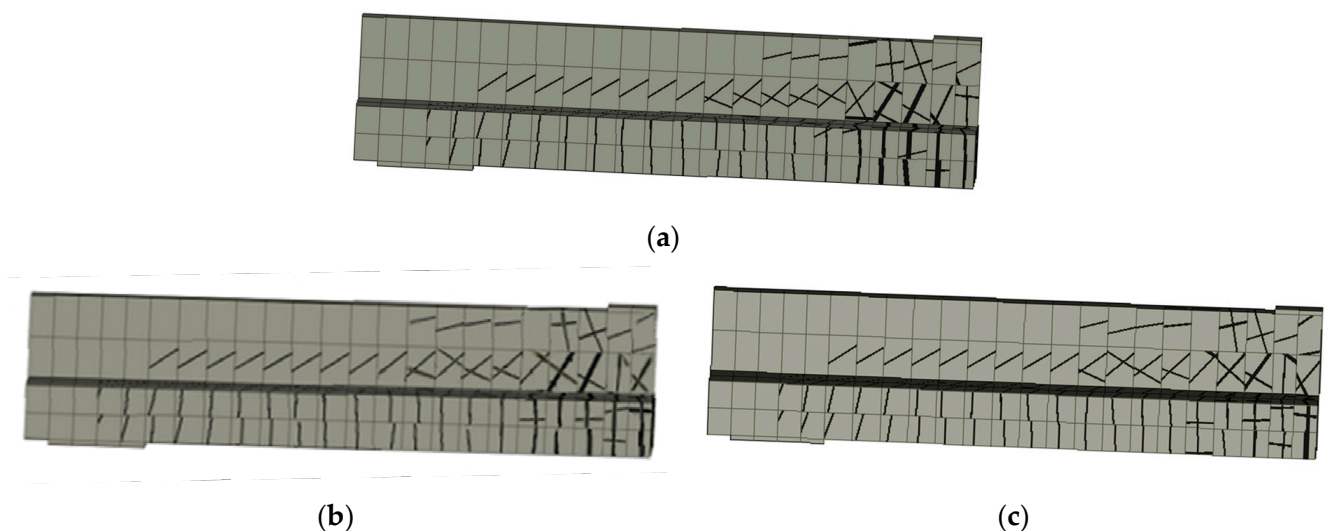


Figure 17. Visualization of crack propagation in CFRP-strengthened RC T-beams [75]. Gray lines represent the FE mesh layout, while black lines indicate simulated crack patterns: (a) specimen CB; (b) specimen SH; (c) specimen SF.

For specimen SH, where the CFRP rods were partially embedded, cracking is more concentrated near mid-span, with the development of diagonal cracking indicative of increased stiffness and reduced ductility. Specimen SF, with fully embedded CFRP rods, exhibits a more refined and evenly distributed crack pattern, consistent with enhanced tensile force transfer and improved composite action. Although the smeared-crack approach tends to produce more diffuse cracking compared to discrete experimental cracks, the predicted crack locations, orientations, and relative damage concentration agree well with experimental observations, confirming that the model adequately captures the distinct failure characteristics associated with each strengthening configuration at the structural level.

4.2.2. PCM-Strengthened RC T-Beams Reported by Haryanto et al. [76]

In the second phase of model validation, the FE predictions were compared with the experimental results of Haryanto et al. [76], with a focus on RC T-beams strengthened with steel-reinforced PCM overlays. As illustrated in Figure 18, the numerical load–deflection

curves closely followed the experimental responses for all tested specimens, reflecting the model's ability to reproduce key structural behaviors under negative moment conditions. The simulation effectively captured the major stages of the response, including the initial elastic phase, yield onset, and post-yield softening. For the control specimen (UB), the plateau region was well represented, and the transition into post-peak softening aligned with the experimental behavior [26].

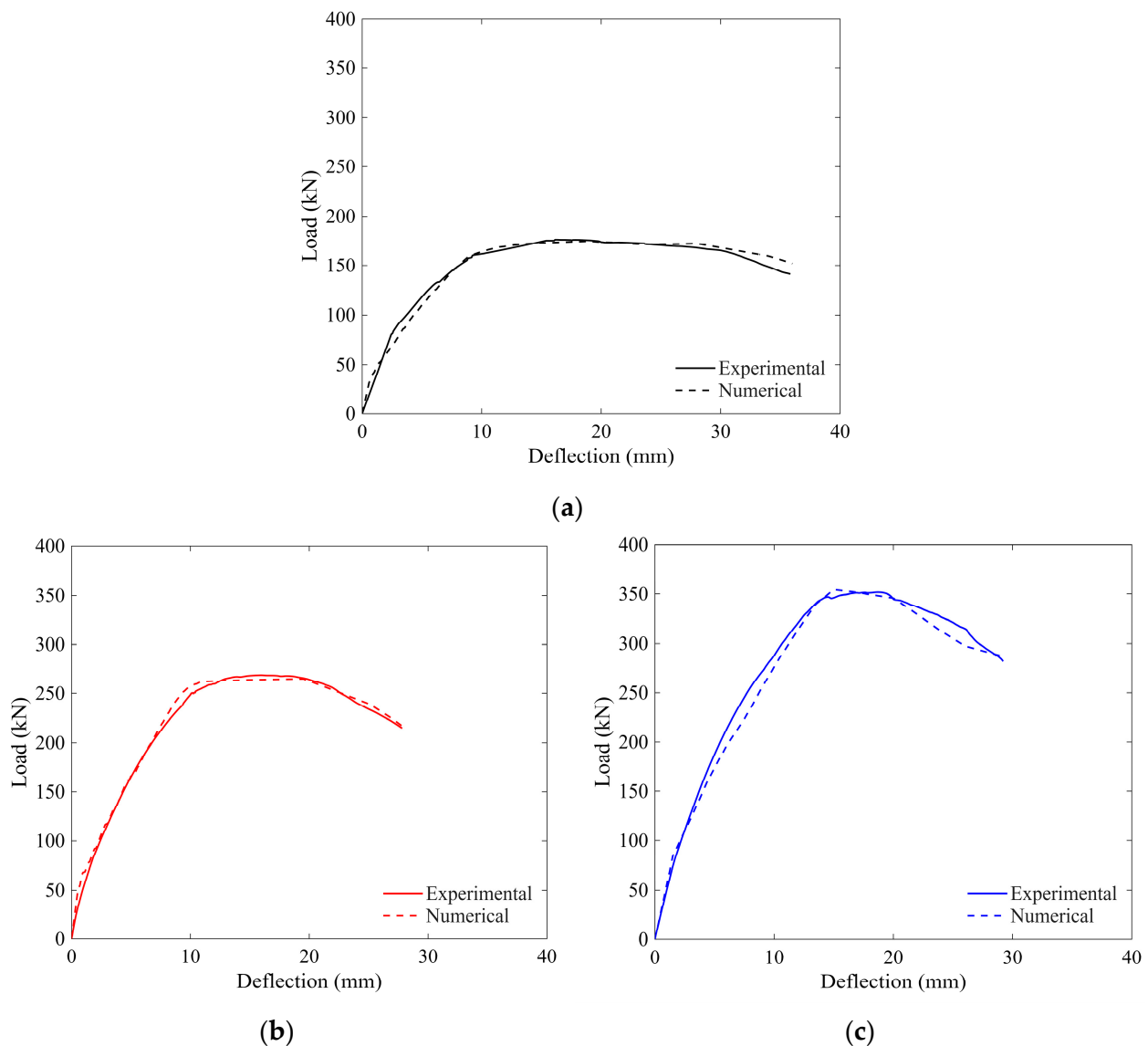


Figure 18. Experimental and numerical load–deflection responses for the tested beams: (a) specimen UB [26,76]; (b) specimen SB-M-13 [76]; (c) specimen SB-M-16 [76]. Reprinted with permission from Ref. [26], Elsevier, 2025.

Similarly, for SB-M-13 and SB-M-16, the numerical load–deflection curves closely followed the progression observed in the experimental results. A key observation is that the FE models were able to capture the influence of the adopted strengthening strategy, including variations in the diameter of the longitudinal reinforcement. Minor discrepancies were observed at specific response stages, namely during the elastic phase and post-peak region for UB and SB-M-16, and around the elastic–plastic transition for SB-M-13. Despite these localized differences, the overall correspondence between the numerical and experimental responses remained strong. The consistent trends observed across all specimens indicate that the developed FE model is capable of reproducing the global

flexural behavior of RC T-beams subjected to different strengthening configurations in negative moment regions.

Table 4 further quantifies this agreement by comparing the experimentally measured and numerically predicted ultimate loads. The numerical-to-experimental peak load ratios exhibit an average value of 0.99, with a standard deviation of 0.01 and a coefficient of variation of 1.14%, confirming the close correspondence observed in the load–deflection responses. In addition, the NMSE values, computed in accordance with Equation (6) using load data sampled at 2 mm intervals of mid-span deflection and summarized in Table 4, indicate that deviations between the simulated and experimental responses remain within acceptable limits. It should also be emphasized that the close agreement observed between the numerical predictions and experimental results was obtained without any post hoc adjustment of material parameters. All constitutive inputs were prescribed a priori based on experimentally measured values and established formulations outlined in Section 3.2. Consequently, the low statistical dispersion reflects the intrinsic capability of the adopted modeling strategy to capture the governing structural response under negative moment conditions, lending confidence to its application in subsequent structural-level, design-oriented parametric analyses.

Table 4. Comparison between experimental and numerical results for PCM-strengthened T-beams.

Specimen	Ultimate Load (kN)		Ratio	NMSE
	Experimental [26,76]	Numerical		
UB	176.66	175.12	0.99	0.0022
SB-M-13	268.91	265.00	0.99	0.0015
SB-M-16	351.81	354.41	1.01	0.0018
Average			0.99	
Standard deviation			0.01	
Coefficient of variation (%)			1.14	

Figure 19 illustrates the numerically predicted crack distribution and crack-width contours for the unstrengthened beam (UB) and the beams strengthened with steel-reinforced PCM overlays. For all specimens, cracking is predominantly concentrated in the mid-span region, corresponding to the maximum flexural tension zone, which is consistent with the expected structural behavior under negative bending. The UB specimen exhibits wider and more localized crack zones, indicating limited post-cracking tensile resistance.

In contrast, both strengthened beams show narrower crack widths. The contour gradients further reveal a broader transition zone of the crack width in the strengthened beams, indicating a more uniform crack distribution and suggesting that the PCM layer effectively distributes the tensile stresses and delays localized crack propagation. It is acknowledged that the smeared-crack formulation and mesh discretization may promote a more distributed cracking pattern than that observed experimentally; therefore, the agreement should be interpreted in terms of crack severity, extent, and localization trends rather than one-to-one geometric matching. Within this context, the numerical results reproduce the experimentally observed reduction in crack penetration into the original concrete substrate and the delayed localization of damage, supporting the reliability of the FE model in capturing the fracture behavior of PCM-strengthened RC T-beams from a structural-response perspective.

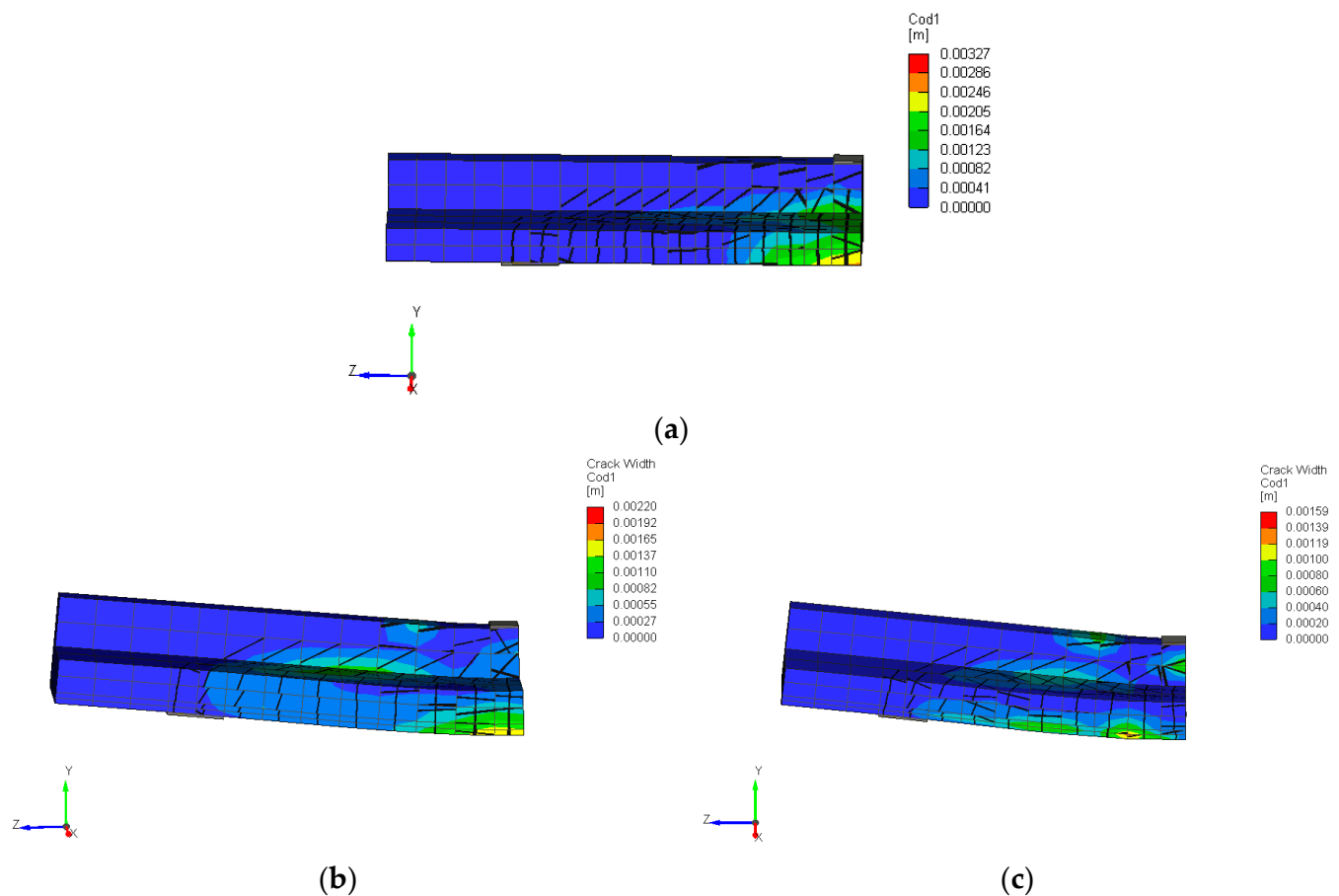


Figure 19. Comparison of compressive damage contours. Color gradients represent the crack width, with red indicating the maximum crack width and blue indicating minimal width or no crack. Black lines show the crack distribution, while gray lines show the finite element mesh. Specimens shown: (a) UB; (b) SB-M-13; (c) SB-M-16.

4.3. Parametric Study

Building on the validated FE models, the parametric study is deliberately directed toward identifying response characteristics that are specific to steel-reinforced PCM strengthening systems rather than reiterating generic RC trends. The analysis examines how variations in concrete strength, PCM overlay thickness, and embedded steel reinforcement ratio alter the interaction between the original RC section and the PCM layer, thereby modifying ultimate load capacity, stiffness evolution, ductility, and energy absorption in negative moment regions. By isolating the combined and competing effects of overlay thickness and steel reinforcement within the PCM, the study provides new, system-specific insights into how steel-reinforced PCM overlays redistribute tensile demand, enhance load transfer, and influence damage tolerance, outcomes that cannot be inferred directly from conventional RC behavior alone. These findings position the parametric results as design-oriented guidance tailored to PCM-based strengthening applications rather than as a confirmatory extension of established RC principles.

4.3.1. Influence of Concrete Compressive Strength

This part of the study examines the influence of varying the compressive strength of concrete (f'_c) on the global flexural behavior of the beams. Three strength levels were considered: 17.50 MPa, 28.68 MPa (baseline value from the experiment), and 50.00 MPa, representing low, medium, and high grades of concrete, respectively. The variation was applied to each beam configuration (UB, SB-M-13, and SB-M-16), producing a total of nine

FE models. To isolate the effect of the concrete strength, all other parameters, including the PCM overlay thickness (40 mm) and longitudinal reinforcement ratio (1.00%), were kept identical to the experimental setup.

Figure 20 presents the predicted load–deflection responses, while Table 5 summarizes the corresponding key parameters, highlighting the influence of the compressive strength on structural performance. Increasing the compressive strength leads to a consistent rise in flexural capacity for both unstrengthened and strengthened beams. This improvement arises from the higher values for the modulus of elasticity and tensile capacity of the stronger concrete, which elevate the neutral-axis depth and increase the tensile strain demand in the reinforcement, allowing the section to sustain greater loads. These tendencies correspond well with findings established in prior studies [108,109], confirming that higher concrete strength enhances the flexural resistance of both control and retrofitted members.

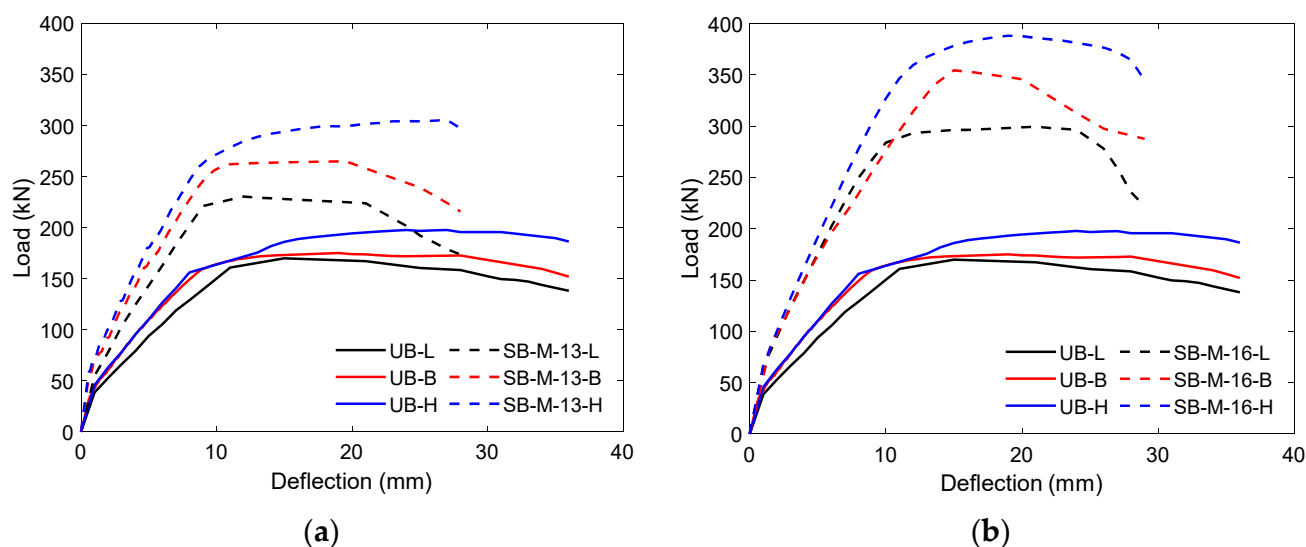


Figure 20. Predicted load–deflection response with varying concrete compressive strengths: (a) strengthened beam with 13 mm deformed rebars and PCM (SB-M-13); (b) strengthened beam with 16 mm deformed rebars and PCM (SB-M-16). All simulations were performed using a longitudinal reinforcement ratio of 1.00% and a PCM overlay thickness of 40 mm.

Table 5. Structural performance parameters of beams with varying concrete compressive strengths.

Beam Model	Concrete Compressive Strength (MPa)	Structural Performance Parameters							
		Ultimate Load (kN)		Stiffness (kN/mm)		Ductility Index		Energy Absorption (kN·mm)	
		Value	Ratio	Value	Ratio	Value	Ratio	Value	Ratio
UB-L	17.50	169.90	-	16.24	-	3.44	-	5048.37	-
UB-B	28.68	175.12	-	19.93	-	4.10	-	5455.52	-
UB-H	50.00	197.81	-	19.80	-	3.60	-	6003.54	-
SB-M-13-L	17.50	230.48	1.36	26.67	1.64	3.24	0.94	5268.52	1.04
SB-M-13-B	28.68	265.00	1.51	29.99	1.50	3.17	0.77	6163.41	1.13
SB-M-13-H	50.00	305.56	1.54	31.72	1.60	2.91	0.81	7063.51	1.18
SB-M-16-L	17.50	299.44	1.76	32.51	2.00	3.15	0.92	7125.83	1.41
SB-M-16-B	28.68	354.42	2.02	29.53	1.48	2.42	0.59	7800.24	1.43
SB-M-16-H	50.00	388.13	1.96	34.20	1.73	2.56	0.71	8988.45	1.50

The stiffness and ductility indices obtained from the FE simulations further elucidate the influence of compressive strength. For the SB-M-13 series, the low-strength beam (SB-M-13-L) exhibited an 11.07% reduction in stiffness relative to the baseline (SB-M-13-B), while the high-strength model (SB-M-13-H) showed a 5.76% increase. When compared with their respective unstrengthened counterparts, the increases in stiffness were 64.28%, 50.48%, and 60.24% for SB-M-13-L, SB-M-13-B, and SB-M-13-H, respectively, indicating that PCM strengthening was effective across all grades of concrete. In contrast, the SB-M-16 group showed a slightly different pattern: SB-M-16-L and SB-M-16-H exhibited stiffness gains of 10.09% and 15.83% over SB-M-16-B, and increases of 100.18%, 48.17%, and 71.60% relative to the corresponding unstrengthened beams, respectively.

The ductility response displayed a declining trend with increasing concrete strength. For SB-M-13, the ductility was higher for SB-M-13-L but showed a reduction of 8.19% for SB-M-13-H compared with the baseline, while the corresponding UB models showed reductions with range or ratio from 0.77 to 0.94. In contrast, the SB-M-16 configuration exhibited a 30.17% gain in ductility at low strength and a 5.79% rise at high strength relative to SB-M-16-B, but when compared with the unstrengthened beams, the overall ductility was lower by 8.43%, 40.98%, and 37.56% for SB-M-16-L, SB-M-16-B, and SB-M-16-H, respectively. Taken together, these results indicate that while a higher compressive strength contributes to greater stiffness and load resistance, it concurrently reduces the beam's deformation capacity.

The energy absorption capacity, representing the cumulative work performed before failure, showed a consistent improvement with increasing concrete strength. For the SB-M-13 series, the total absorbed energy rose progressively with the compressive strength, with an overall increase of approximately 12.74% from the low- to high-strength condition. Compared with the corresponding unstrengthened beams, the strengthened models demonstrated energy absorption ratios ranging from 1.04 to 1.18, confirming a noticeable enhancement in the beam's capacity to dissipate flexural energy. This improvement reflects the combined effect of the PCM overlay and the steel reinforcement in delaying crack localization and promoting more distributed damage before failure. A similar pattern was identified for the SB-M-16 group, where the energy absorption capacity increased by roughly approximately 6.07% across the examined strength range. Relative to the unstrengthened beams, the energy absorption ratios remained consistently above 1.40, indicating a substantially greater post-yield deformation energy compared with the control models.

4.3.2. Influence of PCM Overlay Thickness

This segment of the parametric study investigates the effect of varying the PCM overlay thickness on the flexural behavior of strengthened beams. Three thickness levels of 25, 30, and 40 mm were analyzed for both SB-M-13 and SB-M-16, representing practical values commonly adopted in field strengthening applications. This approach offers a deeper understanding of how performance varies under different combinations of the PCM layer thickness and reinforcing bar diameter, providing useful insights for optimizing retrofitting strategies. To isolate the effect of the overlay thickness, all other parameters, including the compressive strength of the concrete (28.68 MPa) and the longitudinal reinforcement ratio (1.00%), were kept constant across the simulations.

Figure 21 shows the predicted load–deflection curves, and Table 6 summarizes the structural performance. The results indicate that increasing the PCM overlay thickness consistently raises the ultimate capacity in both strengthened series. Relative to the unstrengthened baseline (UB), the ultimate load ratios are in the ranges 1.42–1.51 for SB-M-13

and 1.85–2.02 for SB-M-16, underscoring the strong contribution of the overlay over a range of thicknesses and for both bar diameters.

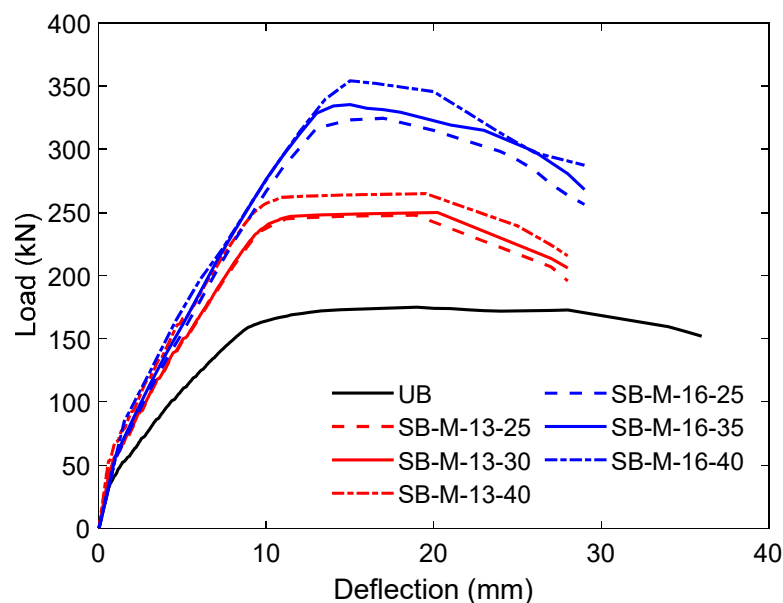


Figure 21. Predicted load–deflection response with varying PCM overlay thickness:(SB-M-13 and SB-M-16). All simulations were conducted with a concrete compressive strength of 28.68 MPa and a longitudinal reinforcement ratio of 1.00%.

Table 6. Structural performance parameters of beams with varying PCM overlay thickness.

Beam Model	PCM Overlay Thickness (mm)	Structural Performance Parameters							
		Ultimate Load (kN)		Stiffness (kN/mm)		Ductility Index		Energy Absorption (kN·mm)	
		Value	Ratio	Value	Ratio	Value	Ratio	Value	Ratio
UB	-	175.12	-	19.93	-	4.10	-	5455.52	-
SB-M-13-25	25	248.00	1.42	26.77	1.34	3.02	0.74	5660.89	1.04
SB-M-13-30	30	250.00	1.43	26.94	1.35	3.02	0.74	5756.86	1.06
SB-M-13-40	40	265.00	1.51	29.99	1.50	3.17	0.77	6163.41	1.13
SB-M-16-25	25	324.70	1.85	27.47	1.38	2.45	0.60	7265.53	1.33
SB-M-16-30	30	335.64	1.92	28.47	1.43	2.46	0.60	7533.84	1.38
SB-M-16-40	40	354.42	2.02	29.53	1.48	2.42	0.59	7800.24	1.43

This gain in strength is accompanied by a slight reduction in deformability: the ductility ratios with respect to UB remain below unity, with values in the range 0.74–0.77 for SB-M-13 and 0.59–0.60 for SB-M-16. In contrast, the stiffness ratios range from 1.34 to 1.50 for SB-M-13 and 1.38 to 1.48 for SB-M-16, showing that any of the three thicknesses improves the elastic rigidity over the control. For both series, the changes were moderate between 25 and 30 mm (0.60% for SB-M-13 and 3.64% for SB-M-16), and became more evident between 25 and 40 mm (11.95% and 7.50%, respectively). Energy absorption exhibited a similar trend; relative to the unstrengthened beam, energy ratios were 1.04–1.13 for SB-M-13 and 1.33–1.43 for SB-M-16.

4.3.3. Influence of Steel Reinforcement Ratio

In this parametric analysis, both unstrengthened and PCM-strengthened beams were modeled with varying longitudinal reinforcement ratios (ρ) of 0.6%, 1.0%, and 2.9%, where

the 1.0% ratio corresponds to the experimental configuration. This variation in ρ also represents conditions of the RC beam where the amount of longitudinal steel reinforcement may differ due to the original design. To maintain a consistent basis for comparison, the compressive strength of the concrete (28.68 MPa) and the PCM overlay thickness (40 mm) were fixed to the experimental values, thus ensuring that the observed variations in flexural response could be attributed solely to changes in reinforcement ratio. The predicted load–deflection curves for all configurations are presented in Figure 22, while Table 7 summarizes the key structural indicators, including the ultimate load, stiffness, ductility, and energy absorption. These results provide a clear depiction of how the reinforcement ratio influences the overall behavior of both the control and strengthened beams under flexural loading.

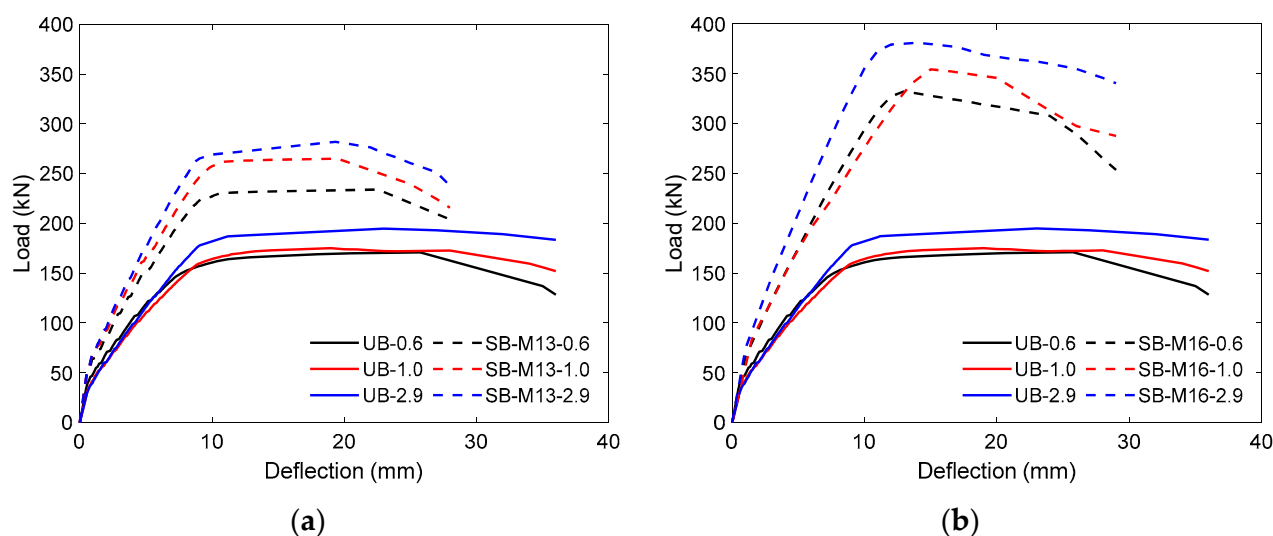


Figure 22. Predicted load–deflection response with varying steel reinforcement ratios: (a) strengthened beam with 13 mm deformed rebars and PCM (SB-M-13); (b) strengthened beam with 16 mm deformed rebars and PCM (SB-M-16). All simulations were performed with a concrete compressive strength of 28.68 MPa and a PCM overlay thickness of 40 mm.

Table 7. Structural performance parameters of beams with varying longitudinal reinforcement ratio.

Beam Model	Steel Reinforcement Ratio (%)	Structural Performance Parameters							
		Ultimate Load (kN)		Stiffness (kN/mm)		Ductility Index		Energy Absorption (kN·mm)	
		Value	Ratio	Value	Ratio	Value	Ratio	Value	Ratio
UB-0.6	0.6	171.00	-	22.34	-	4.70	-	5302.59	-
UB-1.0	1.0	175.12	-	19.93	-	4.10	-	5455.52	-
UB-2.9	2.9	194.75	-	23.84	-	3.95	-	6065.94	-
SB-M-13-0.6	0.6	234.00	1.37	27.89	1.25	3.34	0.71	5562.45	1.05
SB-M-13-1.0	1.0	265.00	1.51	29.99	1.50	3.17	0.77	6163.41	1.13
SB-M-13-2.9	2.9	282.00	1.45	34.46	1.45	3.27	0.83	6578.04	1.08
SB-M-16-0.6	0.6	332.41	1.94	31.23	1.40	2.72	0.58	7592.27	1.43
SB-M-16-1.0	1.0	354.42	2.02	29.53	1.48	2.42	0.59	7800.24	1.43
SB-M-16-2.9	2.9	381.08	1.96	38.39	1.61	2.92	0.74	9008.35	1.49

As the longitudinal reinforcement ratio increased, the beams showed a consistent rise in ultimate load capacity. In the unstrengthened specimens, the flexural strength improved by approximately 13.89% when ρ was increased from 0.6% to 2.9%, while the corresponding gains for the strengthened configurations SB-M-13 and SB-M-16 were about

20.51% and 14.64%, respectively. Compared with their unstrengthened counterparts, the strengthened models achieved load ratios of between 1.37 and 2.02, confirming that the PCM strengthening strategy remained effective in enhancing the load-carrying capacity across a wide range of reinforcement conditions, from under-reinforced to over-reinforced beams. However, these improvements in strength were accompanied by a moderate reduction in ductility. Relative to their respective control beams, the ductility ratios ranged from 0.58 to 0.83, indicating that while the PCM overlay and increased reinforcement enhance strength, they simultaneously reduce the overall deformability due to the stiffer composite action between the concrete, steel, and PCM materials.

Variation in the reinforcement ratio had only a minor effect on the stiffness of the unstrengthened beams, indicating that changes in ρ contributed little to the elastic behavior of plain RC sections. In contrast, the strengthened specimens demonstrated a clearer dependency on reinforcement content. For SB-M-13, the stiffness increased by approximately 23.55% as the ratio rose from 0.6% to 1.0%, whereas for SB-M-16, the change became more pronounced between 1.0% and 2.9%, showing an additional increase of around 22.92%. Compared with their respective unstrengthened counterparts, the stiffness ratios ranged from 1.25 to 1.45 for SB-M-13, with the largest stiffness recorded for SB-M-16-2.9, thus highlighting the combined contribution of the PCM layer and larger reinforcement bars in terms of enhancing flexural rigidity.

The total energy absorption, represented by the area under the load–deflection curve, followed a similar trend. All of the strengthened beams exhibited higher energy dissipation than their unstrengthened counterparts, with energy ratios ranging between 1.05 and 1.49, thereby confirming the synergistic effect of PCM confinement and increased reinforcement area in resisting progressive cracking. The SB-M-13 series showed only moderate improvement, with values of up to 12.98%, while the SB-M-16 series achieved greater but more consistent values across all reinforcement ratios, indicating that once a sufficient area of steel was provided, the additional contribution of the PCM overlay dominated the energy absorption mechanism.

5. Broader Contributions and Design-Oriented Implications

To synthesize the key outcomes of the numerical investigation and clarify their contribution beyond the detailed results presented earlier, this section highlights the broader mechanistic insights and practical implications derived from the study. Rather than reiterating established RC response trends, the present work advances understanding of how steel-reinforced PCM overlays function as a composite strengthening system in negative moment regions. The numerical investigation reveals that the effectiveness of PCM strengthening is governed not only by individual parameters such as overlay thickness or reinforcement ratio, but by their interaction in controlling stiffness evolution, tensile force redistribution, and damage localization. In particular, the results indicate that the PCM layer acts as an intermediary load-transfer medium whose contribution becomes significant only when sufficient reinforcement engagement and anchorage are achieved, highlighting a system-level mechanism that is not captured by conventional RC strengthening interpretations.

From an application-oriented perspective, the parametric results help delineate performance boundaries relevant to practical PCM retrofitting. The analysis suggests that increasing overlay thickness or reinforcement content does not lead to proportional performance gains beyond certain ranges, indicating diminishing returns and potential trade-offs between stiffness, ductility, and energy dissipation. These findings imply that effective PCM strengthening relies on balanced detailing rather than maximization of individual parameters, and that constructability constraints, such as available development length,

interface preparation, and reinforcement placement, play a decisive role in realizing the intended structural benefits. While the results are numerical and therefore indicative, they provide a framework for informed decision-making in preliminary design and help identify critical aspects requiring targeted experimental verification in future studies.

6. Conclusions

This study has presented a comprehensive computational investigation into the use of steel-reinforced PCM overlays for flexural strengthening of RC T-beams in the negative moment region. Two different reinforcement configurations were introduced to a 40 mm-thick PCM layer applied along the full length of the top tensile flange, based on 13 mm and 16 mm bars. Experimentally validated FE models were employed to examine the influence of three key parameters, the compressive strength of the concrete, the thickness of the PCM overlay, and longitudinal reinforcement ratio, on the structural performance of retrofitted beams. Based on the simulation results and a comparative analysis, the following conclusions can be drawn:

- The experimental results demonstrated that applying a 40 mm PCM overlay reinforced with deformed steel bars substantially improved the load capacity, initial and effective stiffness, and energy absorption compared to unstrengthened beams. Despite a slight reduction in ductility due to the increased reinforcement ratios, the strengthened beams exhibited enhanced overall energy absorption.
- The developed FE model was successfully validated against two independent experimental datasets involving CFRP- and PCM-strengthened RC T-beams, showing close agreement with experimental load–deflection responses, ultimate load capacities, and overall cracking trends, with peak load ratios close to unity and low NMSE values. These results confirm the model’s capability to capture the dominant structural response under negative moment conditions.
- The predictive accuracy of the numerical model should be interpreted in light of the adopted modeling assumptions, including perfect bond at material interfaces, a smeared-crack formulation for concrete, and the use of geometric symmetry. While these simplifications enable efficient and stable simulations suitable for design-oriented parametric studies, they may limit the direct representation of localized cracking, interface behavior, and asymmetric damage patterns.
- Increasing the compressive strength of the concrete significantly enhanced the flexural performance of both the control and strengthened beams. The load-carrying capacity increased by up to 12.5%, while the stiffness improvements reached 15.8%, reflecting the beneficial effect of the higher modulus and cracking resistance of stronger concrete mixes.
- The PCM overlay thickness had a marked impact on flexural performance, with the ultimate load increasing by up to 15.4%, the stiffness improving by a factor of up to 1.50, and energy absorption rising by as much as 43% compared to the control beam.
- Varying the longitudinal reinforcement ratio influenced the structural response significantly. Higher ratios resulted in greater load capacity and stiffness, with the ultimate load reaching up to 2.02 times the value for the control beam and the energy absorption improving by as much as 49%. These results highlight the effective synergy between embedded steel bars and the PCM overlay in terms of enhancing flexural resistance.
- The observed improvements in strength and stiffness were consistently accompanied by reductions in ductility, indicating a trade-off between load-carrying capacity and deformation capability that must be carefully considered in seismic strengthening applications.

- While the results demonstrate the effectiveness of PCM overlays in improving the negative moment performance of RC T-beams under monotonic loading, experimental validation is limited to a subset of the investigated parameter ranges. Trends within this validated domain may therefore be considered reliable, whereas predictions outside it should be interpreted as indicative numerical extrapolations. Further studies incorporating material-level characterization, durability, bond–slip behavior, and cyclic and seismic loading are suggested to support broader application in practice.

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Abbreviations

The following abbreviations are used in this manuscript:

RC	Reinforced concrete
ATENA	Advanced Tool for Engineering Nonlinear Analysis
PCM	Polymer cement mortar
FE	Finite element
NMSE	Normalized mean square error
FRCMs	Fabric-reinforced cementitious matrices
UHPC	Ultra-high-performance concrete
FRP	Fiber-reinforced polymer
NSM	Near-surface mounted
NSC	Normal-strength concrete
LVDTs	Linear variable differential transformers
CoV	Coefficient of Variation
CFRP	Carbon-fiber-reinforced polymer

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