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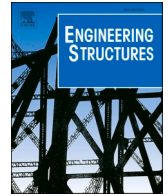
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Arch action and beam action in reinforced engineered cementitious composite (ECC) beams subjected to shear

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ABSTRACT

Shear failure in conventional reinforced concrete (R/C) beams is characterized by pronounced brittleness, limited energy dissipation capacity, and a high propensity for catastrophic collapse due to the absence of discernible warning signs. This study investigates the use of high-ductility fiber-reinforced cementitious composites (Engineered Cementitious Composites, ECC) to enhance the shear performance of concrete beams. An experimental program was conducted on both R/C and reinforced ECC (R/ECC) beams subjected to shear, complemented by a novel numerical approach to quantify the contributions of arch action (V_a) and beam action (V_b) to shear resistance. The findings reveal a strong positive correlation between the efficiency of arch action and overall shear performance, including shear carrying capacity, deformation capacity, and ductility. Building on these insights, an optimized design strategy incorporating partial ECC replacement and pre-defined voids is proposed, illustrating the potential of a mechanism-driven approach to achieve superior shear behavior in structural elements.

1. Introduction

Engineered Cementitious Composites (ECC) are an advanced class of cementitious materials characterized by tensile strain-hardening behavior and multiple cracking under uniaxial tension [1]. Through the incorporation of randomly distributed short synthetic fibers, ECC exhibits exceptional deformation capacity and ductility, with ultimate tensile strains typically ranging from 3% to 12% [2–4]. Under shear loading, ECC demonstrates superior shear strength [5], ductility [6], and energy dissipation capacity [7,8], accompanied by steady crack propagation [8,9]. Owing to these outstanding shear properties, ECC offers a promising pathway to mitigate shear failure in concrete structures, which is typically brittle and susceptible to sudden, catastrophic collapse without prior indication [10]. Previous studies have shown that partial replacement of concrete with ECC in the shear-bending zone can effectively enhance the shear performance of reinforced concrete (R/C) beams [11–13]. Such enhancement includes stable crack propagation, improved deformation capacity, and a shift in failure mode from brittle shear-dominated behavior to ductile flexural response [12,14]. These investigations enabled optimized shear design strategies for ECC applications across diverse structural contexts, including high-rise buildings,

long-span bridges, and other critical infrastructure systems [15–17]. However, theoretical frameworks and predictive models capable of elucidating the complex shear-resisting mechanisms in reinforced ECC (R/ECC) or concrete-ECC composite beams remain scarce.

For the shear behavior of reinforced concrete (R/C) beams, the classical truss model proposed by Ritter and Morsch [18] employs parallel chords to describe shear transfer in the web, as illustrated in Fig. 1 (a). In this model, bending moments are resisted by the top and bottom chords, while shear forces are transferred through concrete compression and stirrup tension, forming the inclined and vertical members of the truss. This simplified approach has long served as the basis for shear design in various design specifications and codes [19–21]. Over the following decades, shear theories for R/C beams have evolved considerably. To more accurately capture the contribution of concrete and the complex stress field in beams under shear, several advanced truss-based systems were proposed [22–24]. The strut-and-tie model, shown in Fig. 1(b), conceptualizes the shear-resisting mechanism as a combination of a tied arch and a conventional truss. This framework explicitly distinguishes different shear-resisting components (such as the concrete compression zone, aggregate interlock, and dowel action) represented by diagonal members. Experimental studies by Leonhardt [24]

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demonstrated that these diagonal members carry a substantial portion of the shear force, particularly in T-beams or short beams. Based on these findings, the refined truss model was developed, in which the compressive chords are merged into a curved arch, as shown in Fig. 1(c). In parallel, Kani [25] conducted extensive experimental investigations to establish a more rational theory of shear resistance. It was found that, after initial shear cracking, an R/C beam behaves like a comb structure, as depicted in Fig. 1(d), where the beam-like concrete “teeth” govern flexural capacity, and the concrete compression zone controls shear capacity by forming an internal tied arch. This so-called tooth model revealed that shear resistance depends on the interaction of two internal mechanisms: arch action (V_a) and beam action (V_b) [26,27]. The arch action is a shear-resisting mechanism formed by the compression zone and bottom longitudinal reinforcement, whereas beam action represents the shear contribution of the web elements, as schematically illustrated in Fig. 3. However, due to the computational complexity of statically indeterminate systems, these models have largely been employed as conceptual tools for describing beam behavior rather than as practical, quantitative analytical methods.

To quantify V_a and V_b in R/C beams, Kim et al. [28] formulated a compatibility equation between the tied arch and the web element, resulting in an analytical model capable of decomposing these two fundamental shear-resisting components. Beyond analytical approaches, Nakamura et al. [29] employed the three-dimensional Rigid-Body-Spring Model (3D-RBSM) to assess the respective contributions of arch and beam actions after cracking, achieving an accurate simulation of the concrete fracture process. The conceptual delineation of arch and beam actions has further inspired investigations on structures with unconventional geometries. Similar to Kani’s investigation on shear-resisting mechanism of beams without transverse reinforcement, Mak et al. [30] performed experimental studies on beams with strategically introduced voids to isolate these two mechanisms, demonstrating that targeted material removal in the web region can significantly enhance shear performance and delay the onset of diagonal cracking. Collectively, these studies highlight the critical role of rigorous arch action and beam action evaluation in elucidating the shear-resisting mechanisms of R/C beams.

The nature of shear resistance in ECC differs fundamentally from that in conventional concrete. Owing to the absence of coarse aggregates, the aggregate interlock force along diagonal shear cracks in ECC is replaced by the fiber bridging effect. In the shear design of R/C beams, decomposing the total shear capacity (V) into contributions from matrix material (V_c) and stirrups (V_s) has proven effective [19,20,31]. This framework can be extended to R/ECC beams by incorporating the fiber bridging effect into V_c , as adopted in the ECC standard specification JSCE-08 [32]. Following this approach, Kanakubo et al. [33,34]

developed an analytical model based on the truss-arch shear transfer mechanism, wherein V_c comprises the contributions from the ECC compression zone and fiber bridging. Gu et al. [35–37] conducted an experimental program using a continuously distributed strain gauging system to measure strains along the full length of stirrups and longitudinal reinforcement, thereby enabling quantitative evaluation of V_c and V_s . Despite these advancements, the interactive coupling among different shear-resisting components in R/ECC members remains unclear. To date, no study has interpreted the shear-resisting mechanism of R/ECC members from the perspective of arch action (V_a) and beam action (V_b).

To better understand the shear-resisting mechanism in R/ECC beams and improve shear design methodologies, this study integrates experimental testing and numerical simulation to quantitatively examine the coupling effects of arch action and beam action (schematically illustrated in Fig. 2). The shear behavior of both R/ECC and R/C beams is investigated through four-point bending tests and finite element simulations, incorporating parametric analyses of shear performance indicators such as failure modes, reinforcement strain distribution, and load-displacement response. A novel numerical approach is developed to decouple the contributions of arch action and beam action in reinforced beams. Building on these insights, an optimized structural configuration for reinforced ECC-concrete composite beams is proposed, involving targeted removal of web material and selective replacement of concrete in the compression zone with ECC.

2. Overview of arch action and beam action model

The shear transfer mechanism of shear-critical beams can be decoupled into arch action and beam action, as illustrated in Fig. 3 [38]. Within the shear span, the shear force (V) and bending moment (M) coexist, and the moment is balanced by the compressive force (C) and tensile force (T) separated by a lever arm (z) ($M = Tz = Cz$), as shown in Fig. 3(b) and (c). Given that $V = dM/dx$, the total shear force can be expressed as the sum of two components [10]:

$$V = z \frac{dT}{dx} + T \frac{dz}{dx} \quad (1)$$

The first term represents beam action, corresponding to shear transfer within the web element (Fig. 3(b)), while the second term represents arch action, corresponding to the vertical component of the resultant compressive force in the compression zone (Fig. 3(c)). The relative contribution of arch action to total shear resistance is quantified by a dimensionless coefficient α ($0 < \alpha < 1$):

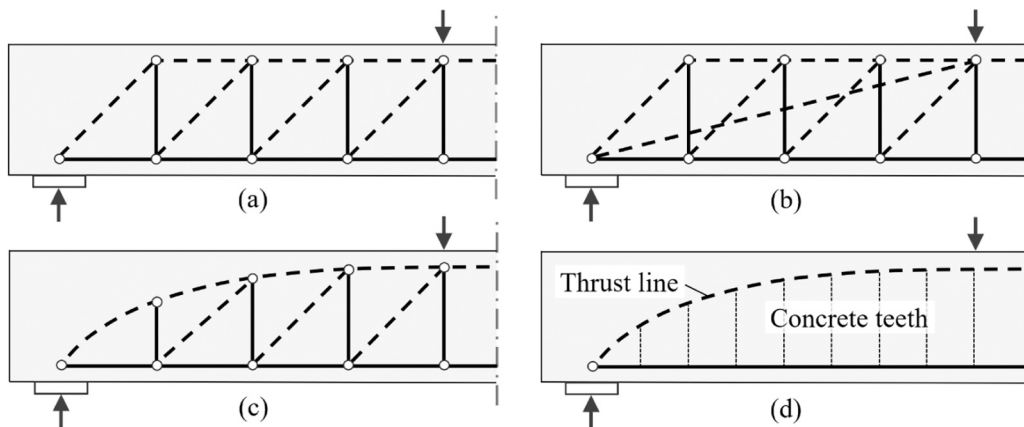


Fig. 1. Analytical models of R/C beam subjected to shear: (a) classical truss model [18]; (b) strut and tie model [22]; (c) refined truss model [24]; (d) tooth model [25].

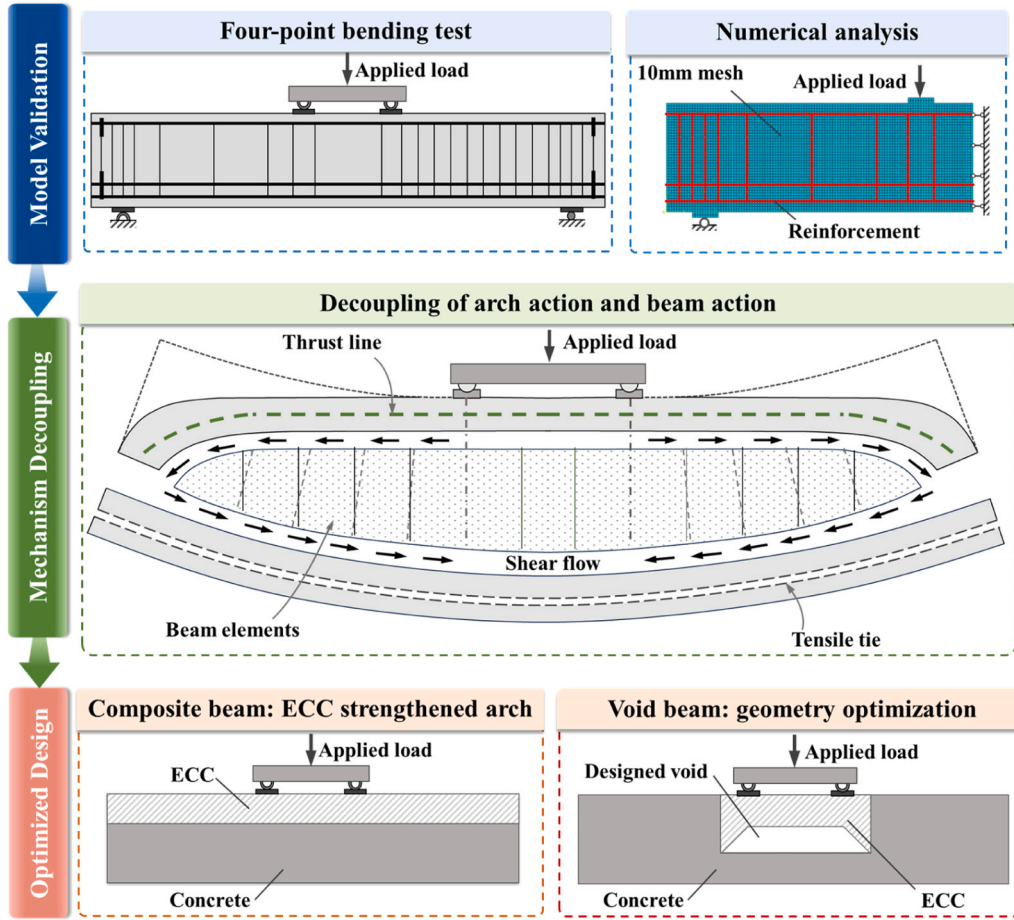


Fig. 2. Schematic diagram of optimizing shear performance of reinforced ECC/concrete composite beam by strengthening the arch action.

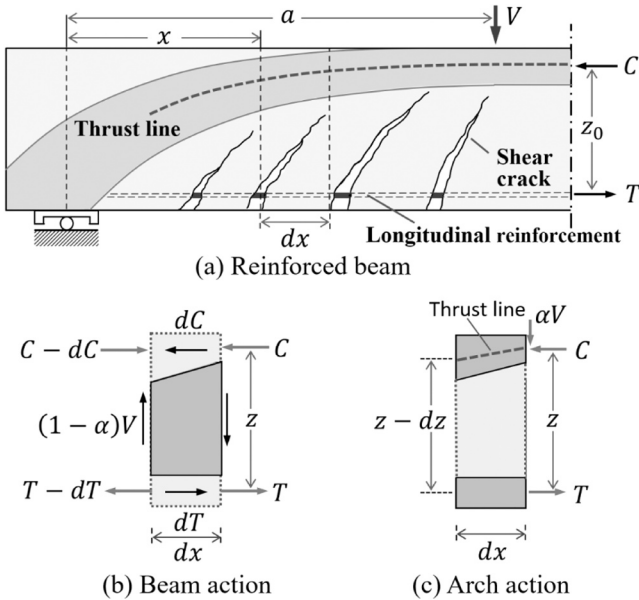


Fig. 3. Mechanism interpretation of shear resistance in R/C beam [38]: (a) reinforced beam after cracking; (b) beam action; (c) arch action.

$$\alpha = \frac{V_a}{V} = \frac{T}{V} \frac{dz}{dx} \quad (2)$$

where V_a is the shear force resisted by arch action. Combining $\alpha V = Tdz/dx$ with $Tz = Cz = Vx$ for simply supported beams under concentrated loads, the arch action coefficient (α) can be expressed as:

$$\alpha = \frac{x dz}{z dx} \quad (3)$$

where x is the distance from support. By integrating Eq. (3) with the boundary condition $z = z_0$ at $x = a$ (where a is the shear span length), an expression for the compressive resultant force path can be derived as follows [39]:

$$z(x) = z_0 \left(\frac{x}{a} \right)^\alpha \quad (4)$$

Eq. (4) indicates that the thrust line follows a power function, providing a quantitative description of compressive force transfer. The arch action coefficient (α) thus serves as a direct measure for numerically evaluating the relative contributions of arch action and beam action to shear resistance.

3. Finite element modeling

To further elucidate the shear-resisting mechanism, nonlinear two-dimensional finite element (FE) analyses are conducted using the commercial software package ABAQUS [40]. The explicit dynamic analysis procedure is adopted to simulate crack initiation and propagation in the nonlinear regime. Owing to the geometric and loading symmetry, only half of the structure is modeled, as illustrated in Fig. 4.

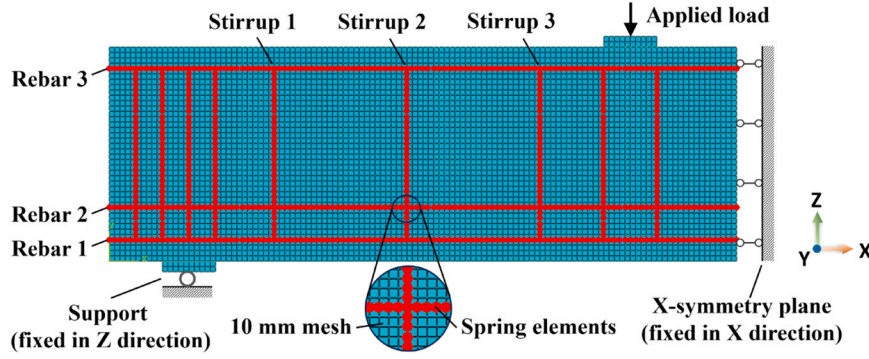


Fig. 4. The two-dimensional FE model of shear test beams.

3.1. Materials

The four nodes bilinear plane stress quadrilateral (CPS4R) element is adopted to model the mechanical behavior of both concrete and ECC. The Concrete Damage Plasticity (CDP) model is employed, which can reflect the elastic-plastic behavior of cementitious composites.

(1) Concrete

The stress-strain relationship for concrete specified in GB 50010–2010 [20] is used (Fig. 5). For both tension and compression, stiffness degradation and material failure are represented by the tensile and compressive damage parameters d_t and d_c , respectively. For tension, the constitutive relation is expressed as:

$$\sigma_{tc} = (1 - d_t)E_c \varepsilon_{tc} \quad (5)$$

$$d_t = \begin{cases} 1 - \rho_t [1.2 - 0.2x_{tc}^5], & x_{tc} \leq 1 \\ 1 - \frac{\rho_t}{\alpha_t(x_{tc} - 1)^{1.7} + x_{tc}}, & x_{tc} > 1 \end{cases} \quad (6)$$

where σ_{tc} is the tensile stress of concrete, ε_{tc} is tensile strain, f_{tc} is the tensile strength, ε_{tuc} is the tensile strain corresponding to f_{tc} , E_c is the elastic modulus of concrete, $x_{tc} = \varepsilon_{tc}/\varepsilon_{tuc}$, $\rho_t = f_{tc}/E_c \varepsilon_{tuc}$, and α_t is the softening parameter for concrete in tension, given by $0.312f_{tc}^2$. For compression, the constitutive relation is expressed as:

$$\sigma_{cc} = (1 - d_c)E_c \varepsilon_{cc} \quad (7)$$

$$d_c = \begin{cases} 1 - \frac{\rho_c n}{n - 1 + x_{cc}^n}, & x_{cc} \leq 1 \\ 1 - \frac{\rho_c}{\alpha_c(x_{cc} - 1)^2 + x_{cc}}, & x_{cc} > 1 \end{cases} \quad (8)$$

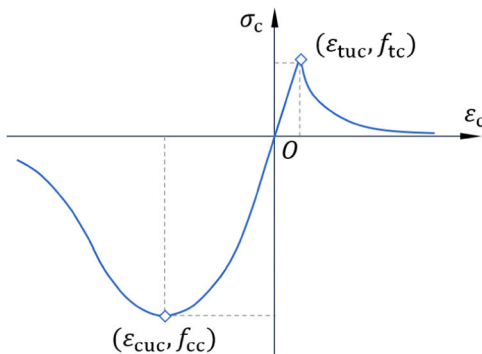


Fig. 5. Constitutive law of concrete under uniaxial compression and tension.

$$n = \frac{E_c \varepsilon_{cuc}}{E_c \varepsilon_{cuc} - f_{cc}} \quad (9)$$

where σ_{cc} is compressive stress of concrete, ε_{cc} is the compressive strain, f_{cc} is the compressive strength, ε_{cuc} is the compressive strain corresponding to f_{cc} , $x_{cc} = \varepsilon_{cc}/\varepsilon_{cuc}$, $\rho_c = f_{cc}/E_c \varepsilon_{cuc}$, and α_c is the softening parameter for concrete in compression, given by $0.157f_{cc}^{0.785} - 0.905$.

(2) ECC

The stress-strain relationship for ECC is shown in Fig. 6. The constitutive model proposed by Zhou [41] is applied for ECC under compression:

$$\sigma_{ce} = \begin{cases} E_e \varepsilon_{ce}, & 0 \leq \varepsilon_{ce} < \varepsilon_{ce1} \\ E_e \varepsilon_{ce} (1.124 - 0.308 \varepsilon_{ce} E_e \varepsilon_{ce} / E_e), & \varepsilon_{ce1} \leq \varepsilon_{ce} < \varepsilon_{cue} \\ 14190(\varepsilon_{ce} - \varepsilon_{cue}) + f_{ce}, & \varepsilon_{cue} \leq \varepsilon_{ce} < \varepsilon_{ce2} \\ 390(\varepsilon_{ce} - \varepsilon_{ce2}) + \sigma_{ce2}, & \varepsilon_{ce2} \leq \varepsilon_{ce} < \varepsilon_{ce3} \end{cases} \quad (10)$$

where E_e is the elastic modulus of ECC, σ_{ce1} and ε_{ce1} are the stress and strain corresponding to 40% of the ECC compressive strength, f_{ce} and ε_{cue} are the compressive strength of ECC and the corresponding strain, σ_{ce2} and ε_{ce2} are the stress and strain at the inflection point of the softening branch, σ_{ce3} and ε_{ce3} are the ultimate compressive stress and strain of ECC.

The tensile behavior of ECC is modeled using the bilinear model specified in JSCE-08 [32]. In this model, f_{tce} and ε_{tce} represent the cracking strength of ECC and the corresponding tensile strain, while f_{te} and ε_{tue} represent the tensile strength and the corresponding strain. The plasticity parameters used in the CDP model are summarized in Table 1.

The two-node, two-dimensional truss element (T2D2) is used to model the reinforcement. Based on the experimentally obtained tensile stress-strain relationship, a bilinear model is adopted to capture the hardening behavior, as expressed in Eq. (11).

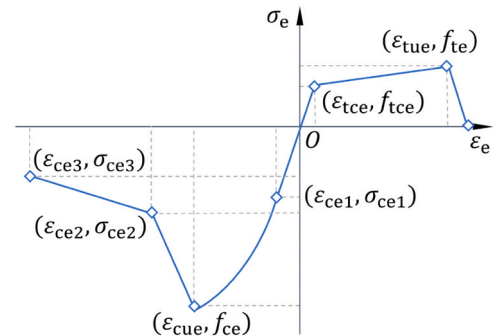


Fig. 6. Constitutive law of ECC under uniaxial compression and tension.

Table 1
Plastic parameters in CDP model.

Eccentricity	f_{b0}/f_{c0}	K	D_c (°)	D_e (°)
0.1	1.16	0.6667	30	10

Notes: D_c is the dilation angle of concrete, D_e is the dilation angle of ECC, K is the yield behavior parameter, f_{b0} is the biaxial compressive strength, and f_{c0} is the uniaxial compressive strength.

(3) Steel

$$\sigma_s = \begin{cases} E_s \varepsilon_s, & 0 \leq \varepsilon_s \leq \varepsilon_y \\ f_y + E_{sh}(\varepsilon_s - \varepsilon_y), & \varepsilon_s > \varepsilon_y \end{cases} \quad \# \quad (11)$$

where E_s is the elastic modulus of steel, σ_s is the reinforcement stress, ε_s is the reinforcement strain, ε_y is the yield strain, f_y is the yield stress, and E_{sh} is the strain-hardening modulus.

The elastic modulus of concrete, ECC, and steel are 36.1 GPa, 21.5 GPa, and 200 GPa, respectively, as determined from material tests conducted by Gu [37].

3.2. Reinforcement-concrete and reinforcement-ECC bond

The bond stress between the steel reinforcement and the surrounding concrete or ECC matrix arises from rib interlock and friction. In the numerical model, the stress-slip behavior is represented by spring elements that connect corresponding nodes.

(1) Reinforcement-concrete bond

Based on systematic analyses and regression of an existing database, Wu and Zhao [42] proposed a nonlinear stress-slip relationship for steel reinforcement embedded in R/C structures, which is suitable for numerical simulation. In this model, the bond stress and its peak value are denoted by τ_c and τ_{cp} , respectively:

$$\tau_c = \frac{\tau_{cp}(e^{Bs} - e^{Ds})}{e^{\frac{-B \ln(\frac{B}{D})}{(B-D)}} - e^{\frac{-D \ln(\frac{B}{D})}{(B-D)}}} \quad \# \quad (12)$$

$$\tau_{cp} = \frac{2.5\sqrt{f_{cc}}}{1 + 3.1e^{-0.47(K_{co} + 33K_{st})}} \quad \# \quad (13)$$

$$B = \frac{0.0254 + K_{st}}{-0.0232 - 8.34K_{st}} \quad \# \quad (14)$$

$$D = 3 \ln \left[\frac{0.7315 + K_{co} + 33K_{st}}{5.176 + 0.3333(K_{co} + 33K_{st})} - 0.13 \right] - 3.375 \quad \# \quad (15)$$

$$K_{st} = \frac{A_{st}}{n_b S_{st} d_t} \quad \# \quad (16)$$

where s is the slip distance, K_{co} is the ratio of concrete cover to rebar diameter, K_{st} is the confinement parameter of stirrups, n_b is the number of the longitudinal rebars, d_t is the diameter of the longitudinal rebars, A_{st} and S_{st} are the cross-sectional area and spacing of stirrups, and B and D are regression coefficients.

(2) Reinforcement-ECC bond

The bond-slip relationship between ECC and steel reinforcement is modeled using the formulation proposed by Billington and Chao [43, 44], who summarized and refined existing models based on beam specimens with lap splices. In this model, the bond stress between ECC and steel is denoted by τ_e , and the slip distance is denoted by s .

The bond-slip relationship is idealized as four branches, as illustrated in Fig. 7: a power-law ascending stage (Branch I), a plateau stage before softening (Branch II), a linear descending stage (Branch III), and a

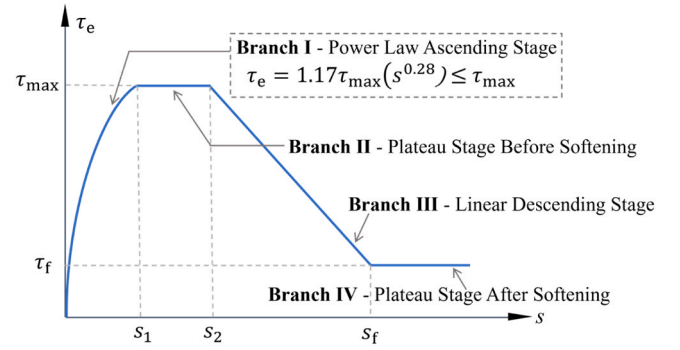


Fig. 7. The relationship between bond stress and reinforcement slip for ECC.

plateau stage after softening (Branch IV). The ascending stage can be expressed as follows:

$$\tau_e = 1.17\tau_{max}(s^{0.28}) \leq \tau_{max} \quad s < s_2 \quad (17)$$

where $\tau_{max} = 1.2\sqrt{f_{ce}}$ denotes the maximum bond stress, s_1 is the corresponding slip distance when the bond stress reaches τ_{max} , s_2 is the slip distance corresponding to the beginning of the softening stage, and f_{ce} is the compressive strength of ECC. The descending branch is defined by a linear relationship from (s_2, τ_{max}) to (s_f, τ_f) , as shown in Fig. 7, $\tau_f = 0.3\tau_{max}$ is the ultimate bond stress of the softening stage, and s_f is the corresponding slip distance of τ_f .

For Eqs. (5) - (17), the core input parameters are summarized in Table 2. Based on these parameters, the constitutive models and bond-slip relationships adopted in the numerical model can be determined. The elastic modulus of concrete, ECC, and steel are denoted by E_c , E_e , and E_s , respectively, and E_{sh} represents the hardening modulus of the steel. The value of s_{max} and s_f are obtained from experimental-based regression analyses reported by Bandelt [43].

It should be noted that, the tensile behavior of ECC exhibits a pronounced size and geometry effect. Experimental studies by Kesner et al. [52] demonstrated substantial variations in tensile strength and ultimate strain between cylindrical specimens (diameter 50 mm) and dumbbell-shaped specimens (cross-section 25 mm×25 mm), with cylindrical specimens showing lower peak tensile stress and reduced ductility. Similar observations were reported by Mechetcherine et al. [53,54], who found that cylindrical specimens (diameter 75 mm) exhibited inferior tensile ductility compared to dumbbell-shaped specimens (24 mm×40 mm). These differences were mainly attributed to initial imperfections, fiber orientation, stress concentration and end effects associated with different specimen geometries [52–54]. Consequently, the tensile response of ECC cannot be regarded as a pure material property independent of specimen size and geometry. To establish a quantitative relationship between material and specimen scale, Kabele [47] proposed a reduction factor for ECC's tensile parameters used to simulate the shear behavior of R/ECC beams. The best agreement between experimental and numerical results was obtained when the ECC's ultimate tensile strain was reduced to 0.01. Accordingly, the input tensile strength and strain parameters adopted in the present model have been adjusted to similar level in [47].

4. Validation of model

4.1. Test setup

Two R/C beams and four R/ECC beams were tested and simulated to validate the accuracy of the FE model. The specimen details are shown in Fig. 8, and the schematic representation of the shear test setup is illustrated in Fig. 9. A three-dimensional Digital Image Correlation (3D-DIC) system [45,46] was employed to capture the full-field strain distribution of the beams. To obtain accurate strain profiles along the reinforcement,

Table 2
Input parameters in numerical model.

Material	Parameter								
Concrete	E_c	f_{cc}			ϵ_{cuc}				
	36.1 GPa	56.6 MPa			0.0022				
ECC	E_e	f_{ce}	f_{ice}	f_{ie}	ϵ_{cue}	ϵ_{ice}	ϵ_{tue}		
	21.5 GPa	56.4 MPa	5.2 MPa	6.5 MPa	0.0039	0.0002	0.01		
Reinforcement	E_s	E_{sh}	s_2		s_f				
	200 GPa	10 GPa	1.0 mm		12.7 mm				

Note: Table 2 presents only the parameters for specimens CMN and EMN, which are discussed in detail in Section 4. Other material properties are determined from tests conducted by Gu [37] and are summarized in Table 3.

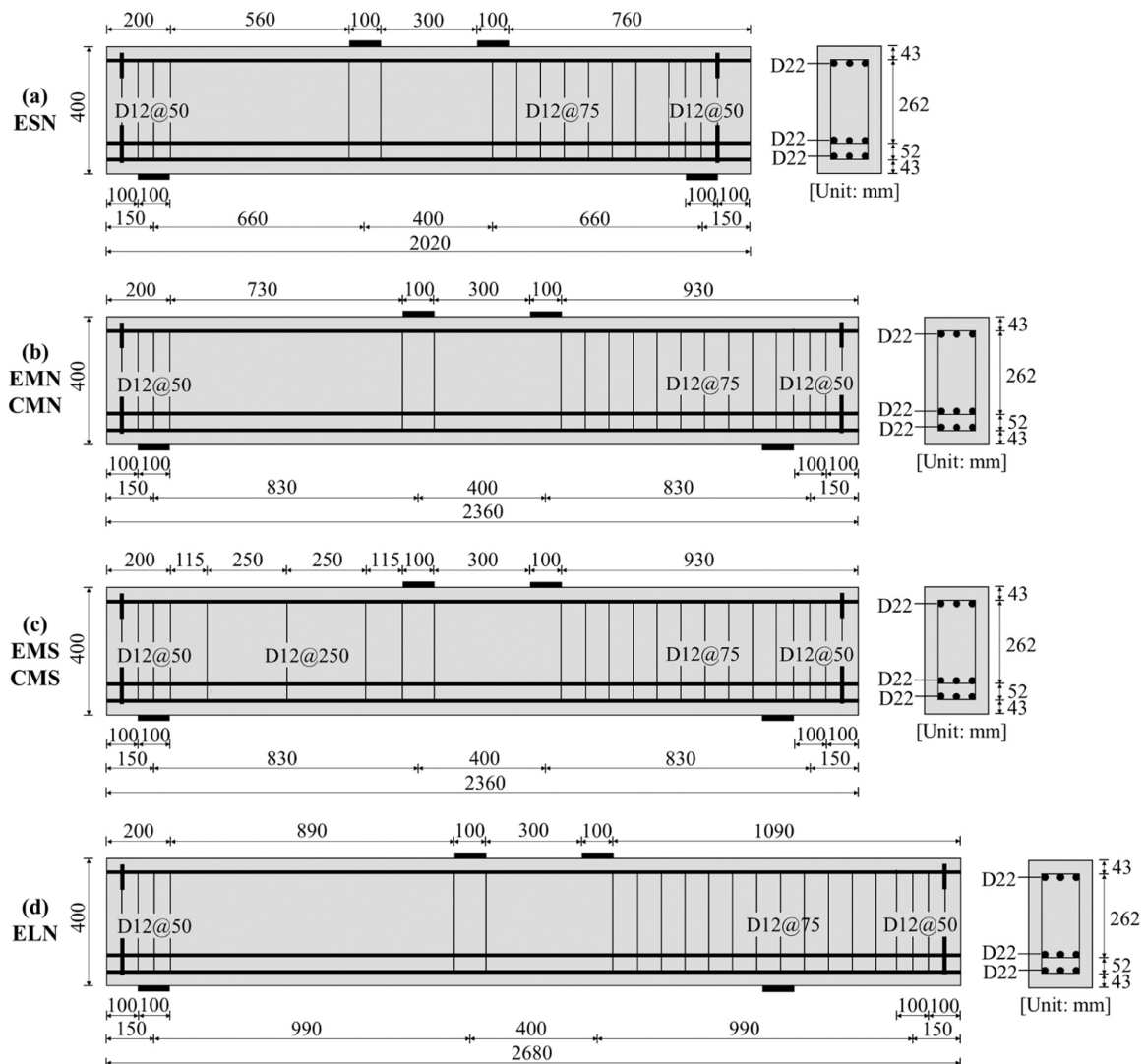


Fig. 8. Specimen and reinforcement details: (a) ESN; (b) EMN and CMN; (c) EMS and CMS; (d) ELN.

a specially designed continuous strain measurement methodology was applied. Cavities of 4×8 mm were cut along the surface of the longitudinal reinforcement, and cavities of 3×6 mm were made in the stirrup legs. Strain gauges were then continuously mounted within these cavities, as shown in Fig. 10. More details on the strain measurement system can be found in Gu et al. [35,37]. To reduce cost, only the left span of each beam was designated as the monitoring zone and equipped with the continuously distributed strain gauging system, while the right span was reinforced with dense stirrups to prevent shear failure.

Five linear variable displacement transducers (LVDTs) were used to measure beam deflection, support settlement, and the rotation of the

mid-span cross section induced by asymmetric reinforcement. The strain gauges and LVDTs were connected to data loggers, and readings were recorded at an acquisition frequency of 2 Hz. A displacement-controlled loading scheme was applied at a rate of 0.4 mm/min throughout the test. The tested material properties are summarized in Table 3. For specimen notation, the first letter denoted the material type (C for concrete, E for ECC), the second letter denoted the span (S for $\lambda = 2.0$, M for $\lambda = 2.5$ and L for $\lambda = 3.0$, where λ is the shear span-to-effective depth ratio), and the third letter denoted the presence of stirrups (N for no stirrup and S otherwise).

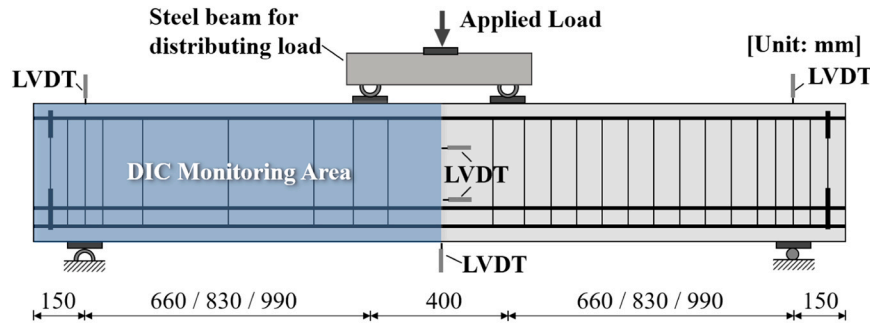


Fig. 9. Schematic representation of shear test setup.

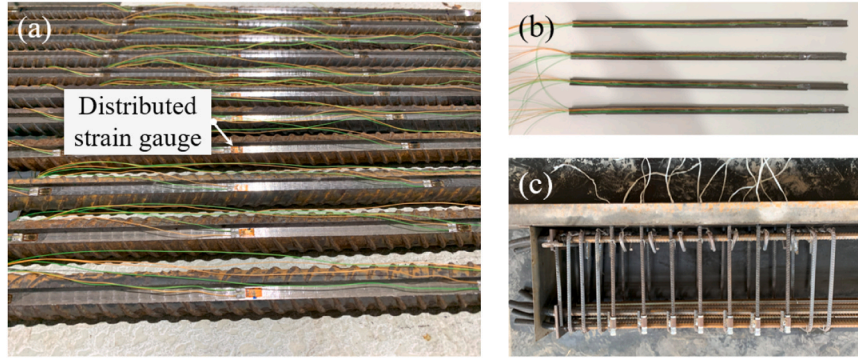


Fig. 10. Distributed strain gauging system: (a) strain gauges continuously attached along the surface of longitudinal reinforcement; (b) strain gauges continuously attached inside stirrup legs; (c) assembled steel reinforcement cage.

Table 3
Input parameters of beam specimens.

Specimen	Matrix	λ	ρ_s (%)	f_{cc} or f_{ce} (MPa)	f_{tce} (MPa)	f_{te} (MPa)
CMN	Concrete	2.5	0.00	56.6	–	–
CMS	Concrete	2.5	0.38	55.0	–	–
ESN	ECC	2.0	0.00	55.8	5.6	7.9
EMN	ECC	2.5	0.00	56.4	6.4	8.0
EMS	ECC	2.5	0.38	54.7	6.1	7.5
ELN	ECC	3.0	0.00	59.1	6.4	7.6
Reinforcement		f_{yt} (MPa)			f_{yl} (MPa)	
		440			635	

Notes: ρ_s is the stirrup ratio, f_{yt} is the yield strength of stirrup, and f_{yl} is the yield strength of longitudinal reinforcement.

4.2. Failure mode, load-deflection response and reinforcement strain distribution

The simulated and tested crack patterns are illustrated in Fig. 11. All the beam specimens failed in shear, and the dominant failing mode was shear-compression failure, i.e., the shear-carrying capacity was reached due to the crushing of matrix material in shear-compression zone. The cracks in R/ECC beams could be identified as regions with concentrated tensile strain in the 3D-DIC system.

Although ECC is well known for its strain-hardening behavior and multiple micro-cracking under uniaxial tension, the crack pattern observed at the structural level is governed by a fundamentally different stress state. In uniaxial tensile tests, the material experiences an approximately uniform tensile stress field, which enables the stable and distributed formation of multiple cracks. In contrast, within the shear span of ECC beams, the principal tensile stress field is highly non-uniform and strongly localized along the diagonal load-transfer path. Consequently, cracking tends to initiate and localize in regions where

the principal tensile stress reaches its maximum, resulting in the formation of a dominant diagonal shear crack. This localization does not contradict the multiple-cracking capability of ECC at the material level, but rather reflects the structural constraint and stress redistribution inherent to shear-critical members. Overall, the numerical model demonstrates high accuracy in predicting failure mode of R/C and R/ECC beams, although the extent of delamination numerically predicted at the reinforcement-concrete and reinforcement-ECC interface is larger than the experimentally observed.

The simulated and tested shear force-deflection relationships and strain distributions along the longitudinal and transverse reinforcement are presented in Fig. 12 and Fig. 13, respectively. The difference between the simulated shear carrying capacity and the experimental results is within 10%. Comparing beams with and without transverse reinforcement, the stirrups had no influence on the initial stiffness but enhanced the post-cracking stiffness. This was attributed to flexural deformation dominating the initial loading stage, with the contribution of stirrups becoming apparent only after the formation of diagonal cracks. A comparison between CMN and CMS showed that the shear capacity increased by 108.1%, whereas the increase for EMN and EMS was only 55.0%. These results indicate that the shear transfer mechanism was fundamentally altered when transverse reinforcement was incorporated into R/ECC beams.

In addition, the simulated stiffness of the R/ECC beams is higher than that measured in the tests. This discrepancy may have resulted from the shrinkage of ECC, which is, due to the absence of coarse aggregates and larger amount of binder, approximately three times greater than that of ordinary concrete. Drying shrinkage cracks in the R/ECC beams were observed prior to testing, which could have led to a reduction in the initial stiffness [35–37]. These observations are consistent with the numerical simulation results reported by [47].

Simulated strain distribution along the longitudinal reinforcement and stirrups is also similar to that experimentally observed (Fig. 13). Note that somewhat lower simulated values (e.g. specially Fig. 13 (c))

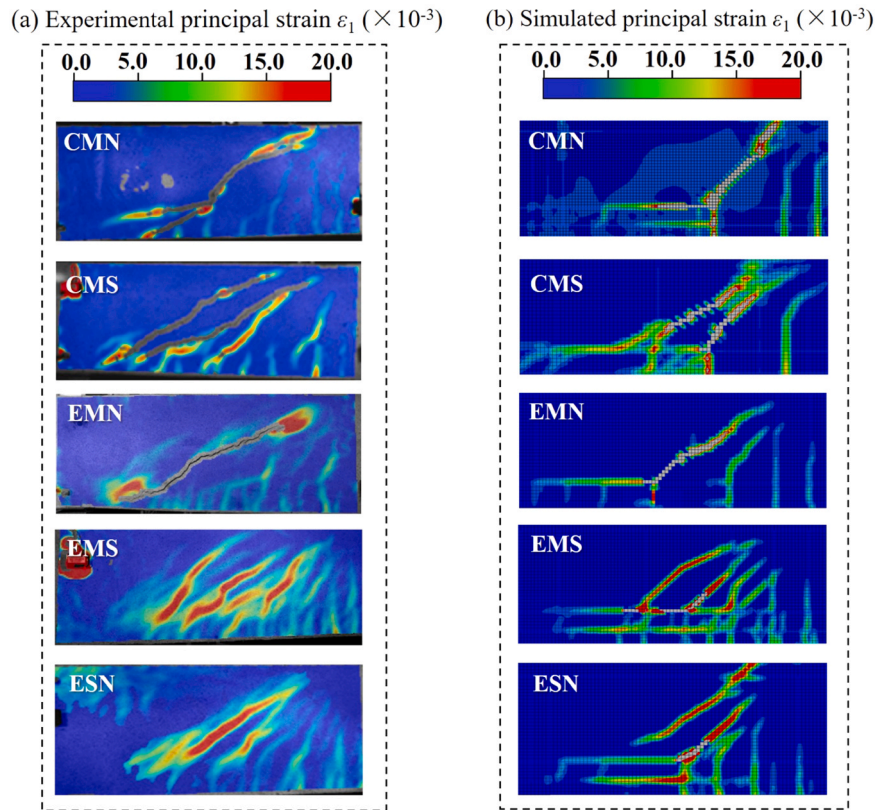


Fig. 11. Crack patterns of R/C beams and R/ECC beams after reaching peak load: (a) tested full-field strain map; (b) simulated principal strain.

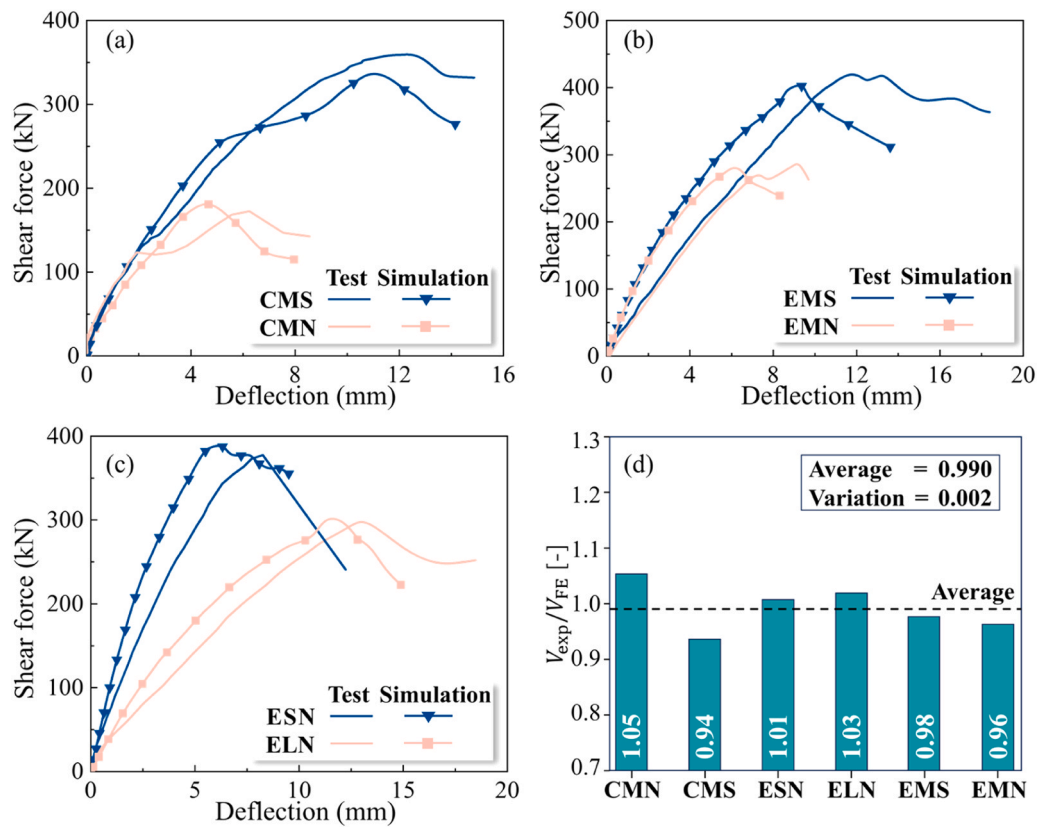


Fig. 12. Comparisons of load-deflection response between FE model predictions and experimental results: (a) R/C beams; (b) R/ECC beams with λ of 2.5; (c) R/ECC beams with λ of 2.0 and 3.0; (d) the ratio of tested to simulated shear carrying capacity V_{exp} and V_{FE} .

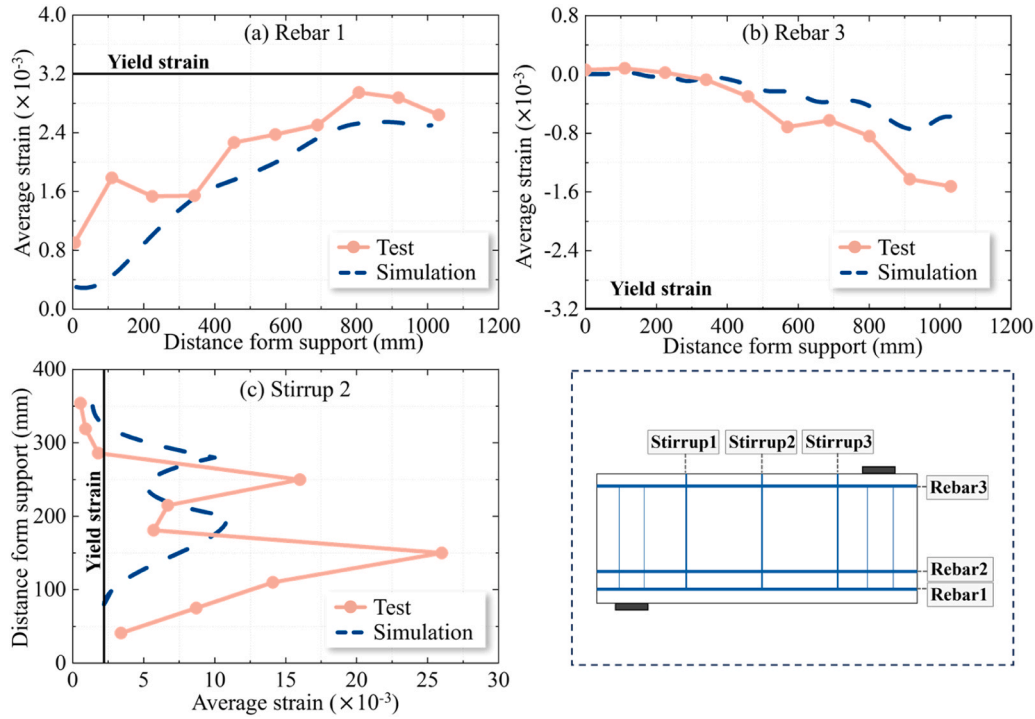


Fig. 13. Simulated and tested strain distribution along reinforcement in the beam CMS at peak load: (a) rebar 1; (b) rebar 3; (c) stirrup 2.

might indicate that in the reality the bond might be higher than the simulated, which is also in line with trends observed in fracture patterns and overestimated delamination of the reinforcement as numerically simulated (Fig. 11). The yielding behavior of longitudinal tensile reinforcement is clearly observable near the pure bending section of the beam EMS—a phenomenon that is absent in the concrete beam CMS. This difference provides direct evidence that the ECC specimen possesses optimized deformation capacity.

5. Decoupling of arch action and beam action

5.1. Decoupling method

The compressive resultant force of each element in the numerical simulation is calculated based on the mesh partitioning, the to determine the thrust line. Using Eq. (4), a power function fitting procedure is developed to evaluate arch action coefficient (α), thereby enabling a quantitative assessment of the arch action and beam action. The

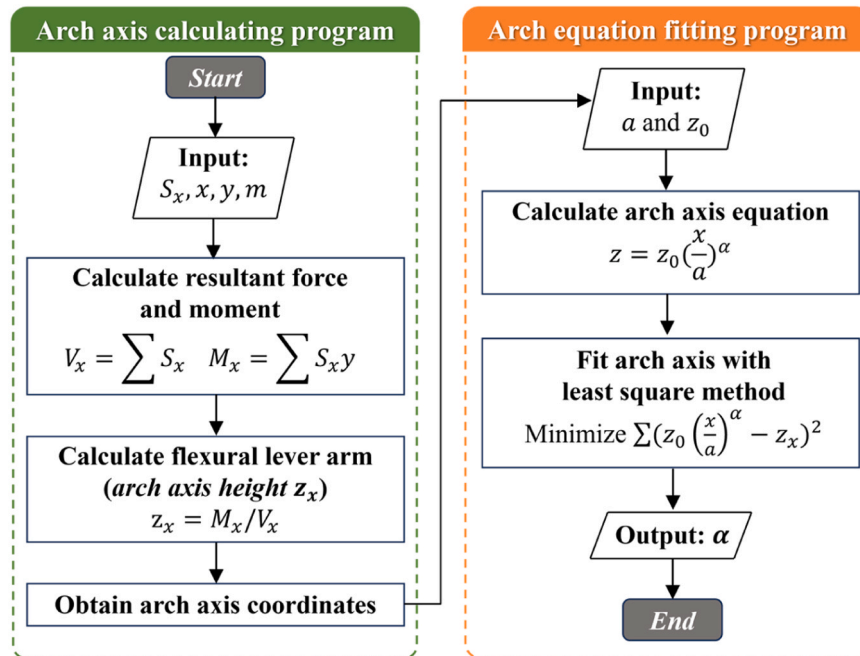


Fig. 14. Flow chart of assessing arch contribution by investigating α (S_x is the node stress along the beam length, x and y are the node coordinates, m is the mesh size, a is the length of shear span section, and z_0 is the lever arm in middle section).

implementation process is illustrated in Fig. 14.

Fig. 15 compares the analytical and fitted arch axes of the tested beams at peak load. The R/ECC beams consistently exhibit a higher arch axis than the R/C beams, indicating a more effective mobilization of the compression zone. According to Eq. (4), this implies a more significant contribution from arch action in R/ECC beams. In addition, comparison between specimens ESN and ELN shows that a shorter shear span length results in a higher arch axis, demonstrating the influence of geometric parameters on arch action. These results motivate a further parametric investigation into the arch action of R/ECC beams, which is presented in the following section.

5.2. Influencing factors of shear-resisting mechanism in R/ECC beams

The influencing factors of arch and beam actions in R/ECC beams are examined through controlled single-variable analyses of the shear span-to-effective depth ratio ($\lambda = 2.0, 2.5, 3.0, \text{ and } 3.5$) and the stirrup ratio ($\rho_s = 0\%, 0.38\%, 0.63\%, \text{ and } 0.95\%$), as shown in Fig. 16 and Fig. 17.

It is observed that λ plays a dominant role in the proportion of the contribution from arch action. When λ decreases from 3.5 to 2.0, the shear-resisting capacity increases by 24.8%, 43.1%, 63.5% and 72.7% for beams with ρ_s of 0%, 0.38%, 0.63% and 0.95%, respectively. With smaller λ the arch action clearly increases. In contrast, variation in ρ_s does not show a clear trend on arch action. Similarly, the relationship between beam action and λ is not clear: for the same ρ_s , there is only an 11.9% difference among beams with different λ values. For beams with the same λ , however, the contribution from beam action exhibits an approximately linear relationship with ρ_s , indicating that the magnitude of beam action depends primarily on the stirrup ratio.

5.3. Optimized design of reinforced ECC/concrete composite beams

The preceding analysis demonstrates that in general, the height of compressive zone is higher for R/ECC beams than R/C beams, which may indicate better preservation of arch mechanism and delay of the typical shear-compression failure. The shear capacity of reinforced beams can be improved effectively by increasing the number of stirrups. However, for certain shear-critical structures without web reinforcement, such as slab-type or deep beam elements, the arch action is the dominate shear-resisting mechanism, enhancing shear resistance through additional stirrups is not feasible. To address this limitation, the present study proposes a reinforced ECC/concrete composite beam in which ECC is locally applied in the compression zone to replace concrete. This approach aims to delay matrix degradation in the shear-compression region, thereby enhancing the effectiveness of arch action and improving the overall shear resistance. To validate the feasibility of the proposed design, a series of parametric numerical analyses is conducted on CMN beam.

5.3.1. Design scheme

In the composite beam design, the top concrete layer of specimen CMN is replaced with ECC at thicknesses d of 50 mm, 100 mm, 150 mm,

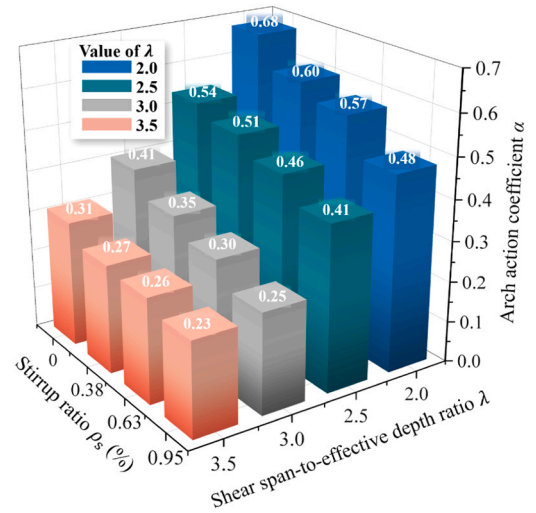


Fig. 16. Influence of shear span-to-effective depth ratio (λ) and stirrup ratio (ρ_s) on the arch action coefficient (α) in R/ECC beams.

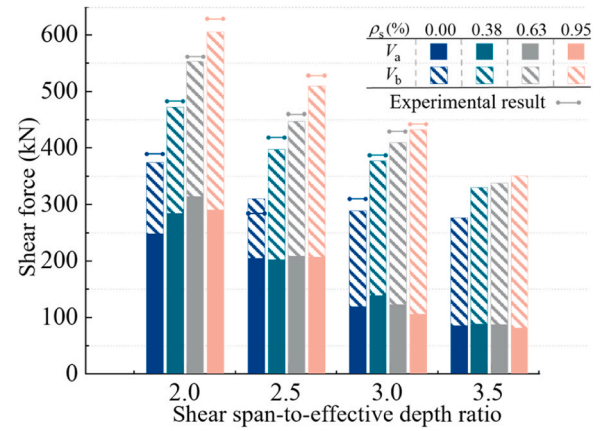


Fig. 17. Decoupling of shear-resisting mechanisms into arch action (V_a) and beam action (V_b).

and 200 mm. The interfacial bonding has a pronounced influence on the overall mechanical behavior of the specimen. Previous studies have reported cases of partial debonding of the composite layer, potentially leading to inadequate utilization of its reinforcing capacity [48,49]. Therefore, ensuring adequate interfacial bonding strength is essential for composite specimens. Past studies have shown that appropriate interface treatment can increase the bond strength between ECC and concrete by approximately 50%, and this level of bond strength is sufficient to maintain the shear transfer mechanism and mitigate brittle interfacial failure [50,51]. Accordingly, perfect bonding between

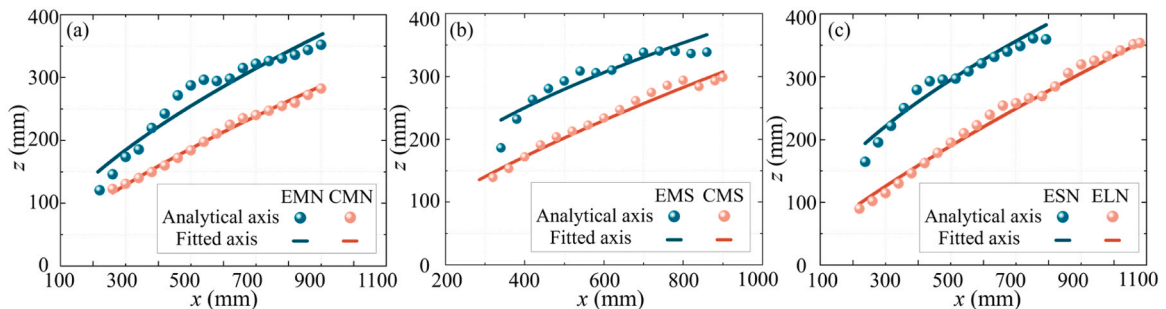


Fig. 15. Comparison between the analytical axis and the fitted axis: (a) EMN and CMN; (b) EMS and CMS; (c) ESN and ELN.

concrete and ECC was assumed in the models presented in Sections 5.3 and 5.4 using a surface-to-surface contact definition. All other configurations remain identical to those in the previous model. The design scheme is shown in Fig. 18.

5.3.2. Shear carrying capacity

Fig. 19 illustrates the relationship between shear force and deflection for the composite beams. Replacing the top concrete layer with ECC markedly enhances both the shear capacity and the deformation capacity of the beams. When d is 50 mm, 100 mm, 150 mm, and 200 mm, the shear capacity increases by 36.0%, 53.3%, 70.0%, and 105.7%, respectively. Correspondingly, the mid-span deflection at peak shear force increases by 9.4%, 24.6%, 45.9%, and 80.3%, respectively.

5.3.3. Crack pattern

Fig. 20 shows the crack pattern of the composite beams at the peak load. When concrete in the compression zone is replaced with ECC, the propagation of diagonal cracks toward the top of the beam is effectively restrained. Consequently, the upward shift of the compressive resultant force is mitigated, allowing the arch action to maintain a substantial contribution to the total shear resistance. Moreover, increasing the ECC thickness further reduces the influence of dominant cracks on failure, leading to a more uniform stress distribution and more distributed cracking, thereby promoting a shift toward bending-dominated behavior.

Fig. 21 illustrates the variation of V_a and V_b in shear-critical composite beams with different ECC thicknesses. The shear force resisted by arch action exhibits an approximately linear relationship with ECC thickness, with its contribution increasing by 7.0%, 11.9%, 15.1%, and 18.0% for ECC thicknesses of 50 mm, 100 mm, 150 mm, and 200 mm, respectively. In contrast, the shear component from beam action remains almost constant. These results indicate that the improvement in shear carrying capacity is primarily attributed to the enhancement of arch action, while the contribution of beam action is not significantly affected by replacing the top concrete with ECC.

5.4. Optimized design of void-beam

In addition to replacing concrete with ECC in the compression zone, recent studies have suggested that introducing voids or cut-outs in R/C beams can improve structural performance by modifying the internal force-transfer mechanism. Van den Hoogen [55] conducted experimental studies on R/C beams where a trapezoidal volume was removed from the bottom tension zone, exposing the longitudinal reinforcement over a central portion of the span. The specimen with a cut-out exhibited an overall resistance approximately 60% higher than the corresponding prismatic specimen. Similar experimental investigations were also conducted by Mak [30], who examined voided beams without interrupting the longitudinal reinforcement. The results showed that a controlled removal of material can increase the overall load-carrying capacity by up to 53%. These enhancements were attributed to the intentional isolation of internal arch action. Further, numerical investigations by van den Hoogen et al. [56] demonstrated that the

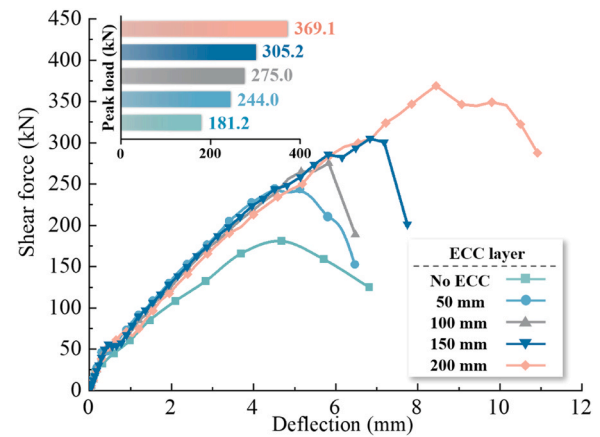


Fig. 19. Simulated shear force-deflection relationship of reinforced ECC/concrete composite beams.

strength enhancement depends on the geometry and positioning of the voids. Effective enhancement can only be achieved when the design of the void facilitates the formation of a stable internal compression strut.

Motivated by the conceptual approaches adopted in earlier studies, an optimized voided beam design is proposed and investigated through parametric analysis using the validated numerical model. In this design, the void width was chosen to fully cover the pure bending region between the two loading points, and a trapezoidal shape with a 1:1.3 inclination was adopted to approximate the expected orientation of the compression struts within the shear span.

5.4.1. Design scheme

The voided beams have a cross-sectional dimension of 400×200 mm, a shear span-to-effective depth ratio of 2.5, and no stirrups. The specimen denoted as *Void* is obtained by strategically removing material near the pure bending section from an initial solid beam, denoted as *Reference*, to isolate the arch-resisting mechanism. In addition, for the specimen denoted as *Composite*, the concrete in the pure bending section and shear-compression zone is replaced with ECC. The geometry and reinforcement details of all three beams are shown in Fig. 22.

5.4.2. Shear-resisting performance

Simulated crack patterns and shear force-deflection curves of the three beam types are shown in Fig. 23. Compared with beam *Reference*, the failure mode of beam *Void* transitions from shear failure along an inclined section to concrete crushing in the compression zone, whereas beam *Composite* is characterized by the inclined cracks initiating at the beam bottom and propagating toward the void corner. Relative to beam *Reference*, the ultimate load-carrying capacities of beams *Void* and *Composite* increase by 51.6% and 63.3%, respectively. Beam *Reference* exhibits a higher initial stiffness than the other specimens; however, once shear cracks form and segment the web into discrete shear-resisting components, the stiffness of all three specimens tends to converge.

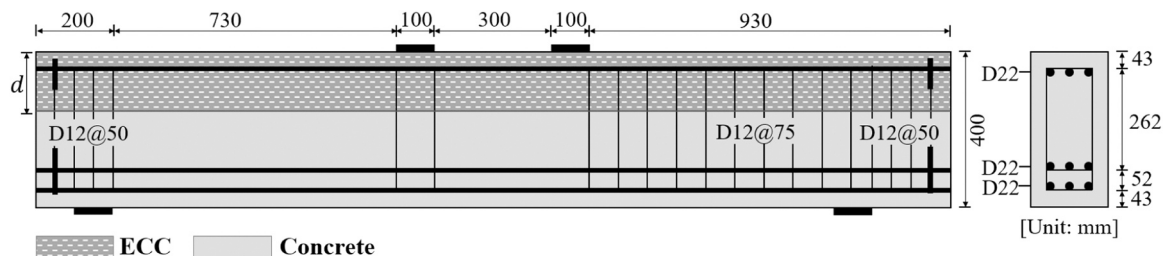


Fig. 18. Specimen design of reinforced ECC/concrete composite beam (d is 50/100/150/200 mm).

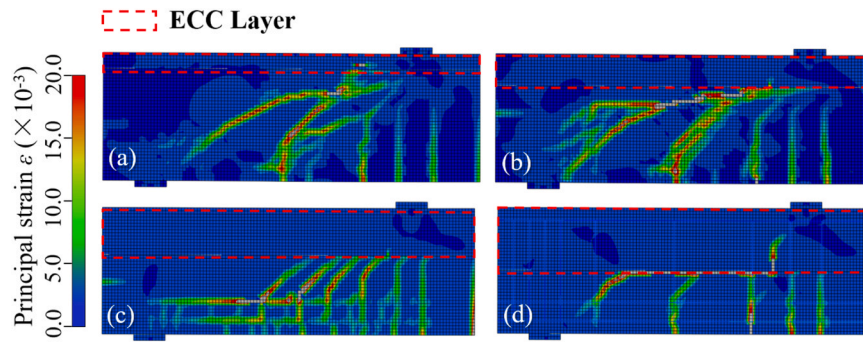


Fig. 20. Simulated crack pattern of reinforced ECC/concrete composite beams at the peak load: (a) d is 50 mm; (b) d is 100 mm; (c) d is 150 mm; (d) d is 200 mm.

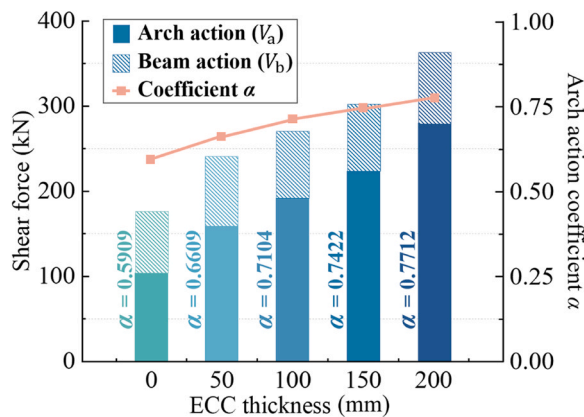


Fig. 21. Decoupling of arch action and beam action in reinforced ECC/concrete composite beams.

The void within the beam acts as a pre-defined defect that directs the development of shear cracks. This mechanism restrains crack extension toward the supports, which would otherwise compromise the compression zone. Furthermore, replacing concrete in the compression zone with strategically placed ECC substantially enhances the strength

and ductility of the composite beam while delaying failure in the pure bending section. The improvement and preservation of arch action are further confirmed by the quantified α , which are 0.59, 0.79, and 0.81 for beams *Reference*, *Void*, and *Composite*, respectively. These results are consistent with the preceding parametric analysis, verifying that strategic void formation and ECC application in the compression zone synergistically enhance arch action and, consequently, the overall shear capacity of shear-critical beams.

6. Conclusion

This research provides a numerical and analytical framework for understanding and enhancing the shear resistance of reinforced concrete, ECC, and ECC/concrete composite beams. Building upon established finite element models, it introduces a novel application of numerical simulation to quantitatively decouple and assess the contributions of arch action and beam action in shear-critical beams. By integrating experimental validation, numerical analysis, and parametric studies, the work establishes mechanism-driven strategies (such as partial ECC replacement and void configuration) that effectively improve shear performance, particularly in cases where conventional stirrup reinforcement is impractical. The main findings are as follows:

(1) The proposed finite element model, incorporating ECC constitutive behavior, reinforcement bond-slip laws, and concrete damage

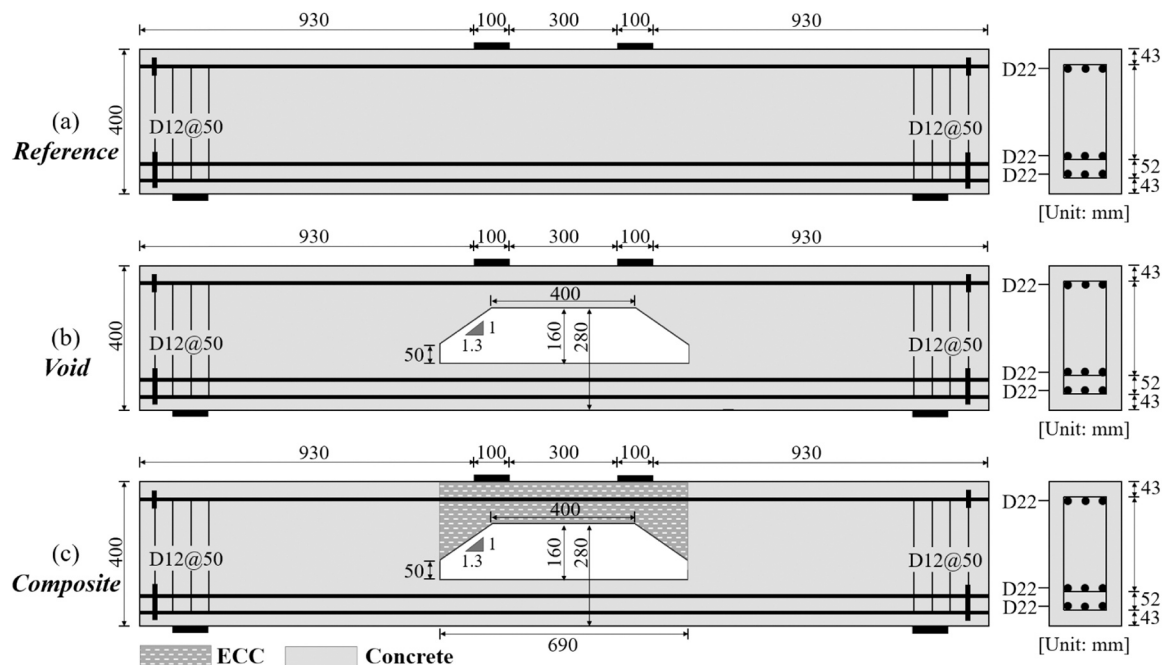


Fig. 22. Design scheme of void-beams: (a) *Reference*; (b) *Void*; (c) *Composite*.

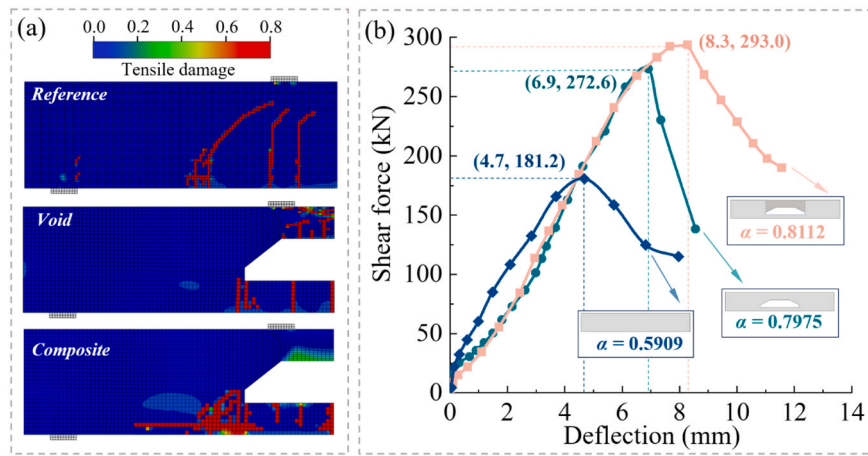


Fig. 23. Shear behavior of intact beam, void beam and composite void beam: (a) crack patterns at the peak load; (b) shear force-deflection relationship.

plasticity, accurately reproduces experimental results in terms of shear capacity, reinforcement strain distribution, and crack pattern, with peak load prediction errors within 10%. It seems though that in the model the bond between reinforcement and concrete (ECC) is slightly overestimated.

(2) A numerical algorithm is developed to determine the compressive arch axis in R/C and R/ECC beams and evaluate the corresponding shear contributions. Results show that arch action dominates in beams with shear span-to-depth ratios $\lambda < 3$, with its contribution increasing as λ decrease.

(3) Partial replacement of concrete with ECC in the compression zone significantly improves shear capacity and deformation capacity. When ECC replaces 12.5%–50% of the beam height, shear carrying capacity increases by 36.0%–105.7%, primarily due to the strengthening of arch action, while the contribution of beam action shows minimal variation.

(4) Introducing strategically positioned voids near the pure bending zone effectively isolates and activates the internal arch mechanism. When combined with ECC in the compression zone, the composite void beam achieves a 63.3% increase in ultimate shear carrying capacity and a 76.5% increase in peak-load deflection, exhibiting superior ductility and load bearing capacity.

(5) Enhancing internal arch action, either through material optimization (partial ECC replacement) or geometry modification (void configuration), provides an effective strategy for improving shear performance in R/C members, especially where conventional stirrups are impractical. For shear-critical members, pre-designed voids can be placed in regions prone to shear cracking, for example, material near the end of the shear crack can be removed to release internal stress and mitigate brittle failure. For strengthening existing concrete structures, damaged zones can be replaced or reinforced with strategically placed ECC.

Notation

α	=	arch action coefficient
α_c	=	$0.157f_{cc}^{0.785} - 0.905$
α_t	=	$0.312f_{tc}^2$
ϵ_1	=	principal strain
ϵ_{cc}	=	compressive strain of concrete
ϵ_{ce}	=	compressive strain of ECC
ϵ_{ce1}	=	the compressive strain of ECC corresponding to σ_{ce1}
ϵ_{ce2}	=	the compressive strain of ECC corresponding to σ_{ce2}
ϵ_{ce3}	=	ultimate compressive strain of ECC
ϵ_{cuc}	=	the concrete compressive strain corresponding to f_{cc}
ϵ_{cue}	=	the compressive strain of ECC corresponding to f_{ce}
ϵ_s	=	strain of reinforcement

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(continued)

α	=	arch action coefficient
ϵ_{tc}	=	tensile strain of concrete
ϵ_{tce}	=	the tensile strain of ECC corresponding to f_{tce}
ϵ_{tuc}	=	the concrete tensile strain corresponding to f_{tc}
ϵ_{tue}	=	the tensile strain of ECC corresponding to f_{te}
ϵ_y	=	yield strain of steel
ρ_c	=	$f_{cc}/E_c\epsilon_{cuc}$
ρ_t	=	$f_{tc}/E_c\epsilon_{tuc}$
σ_{cc}	=	compressive stress of concrete
σ_{ce}	=	compressive stress of ECC
σ_{ce1}	=	40% of ECC compressive strength
σ_{ce2}	=	compressive stress at the inflection point of the ECC softening curve
σ_{ce3}	=	ultimate compressive stress of ECC
σ_s	=	stress of reinforcement
σ_{tc}	=	tensile stress of concrete
τ_c	=	steel-concrete bond stress
τ_{cp}	=	peak bond stress between steel and concrete
τ_e	=	steel-ECC bond stress
τ_{max}	=	peak bond stress between steel and ECC
τ_f	=	the ultimate steel-ECC bond stress of the softening stage
a	=	shear span length
A_{st}	=	cross-sectional area of steel bars
C	=	resultant compressive force of the cross-section
d_t, d_c	=	tensile and compressive damage parameter of concrete
d_r	=	diameter of the longitudinal bars
E_c	=	elastic modulus of concrete
E_e	=	elastic modulus of ECC
E_s	=	elastic modulus of steel
E_{sh}	=	hardening modulus of steel
f_{cc}	=	compressive strength of concrete
f_{ce}	=	compressive strength of ECC
f_{tc}	=	tensile strength of concrete
f_{tce}	=	crack strength of ECC under tensile
f_{te}	=	tensile strength of ECC
f_y	=	yield stress of steel
f_{yl}	=	yield strength of longitudinal reinforcement
f_{yt}	=	yield strength of transverse reinforcement
K_{co}	=	ratio of concrete cover to rebar diameter
K_{st}	=	confinement effect parameter of stirrups on longitudinal bars
m	=	mesh size
M	=	bending moment
n_b	=	number of the longitudinal bars
s	=	slip distance
s_1	=	the corresponding slip distance when the bond stress reaches τ_{max}
s_2	=	the slip distance corresponding to the beginning of the softening stage
s_f	=	the corresponding slip distance of τ_f
S_{st}	=	spacing of transverse reinforcement
S_x	=	node stress along the length direction
T	=	resultant tensile force of the cross-section
V	=	shear force
V_a	=	shear force resisted by arch action
V_b	=	shear force resisted by beam action
V_c	=	shear force resisted by matrix material

(continued on next page)

(continued)

α	=	arch action coefficient
V_s	=	shear force resisted by transverse reinforcement
x_{cc}	=	ratio of ε_{cc} to ε_{cuc}
x_{tc}	=	ratio of ε_{tc} to ε_{tuc}
x, y	=	horizontal axis and vertical axis
z	=	internal level arm
z_0	=	the level arm in the middle section
z_x	=	the level arm corresponding to x

CRediT authorship contribution statement

Mladena Luković: Writing – review & editing, Supervision, Methodology. **Jinlong Pan:** Writing – review & editing, Supervision, Investigation, Funding acquisition, Conceptualization. **Dawei Gu:** Writing – review & editing, Writing – original draft, Supervision, Methodology, Investigation, Funding acquisition, Conceptualization. **Yusen Zhu:** Writing – original draft, Visualization, Validation, Software, Methodology, Investigation, Data curation.

Declaration of Competing Interest

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

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Data Availability

Data will be made available on request.

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