

Article

Laminated Steel Fiber-Reinforced Concrete Hingeless Arch: Research on Damage Evolution Laws

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Abstract: In the context of reinforced concrete (RC) arch bridges, while the incorporation of full sections of steel fibers can enhance the bridge's toughness, cracking resilience, and bearing capacity, achieving an optimal balance between structural performance and economic viability in this manner remains challenging. This article introduces a novel computational approach—the distributed steel fiber concrete (LSFRC) arch—which considers the spatial distribution of damage in RC arches. The static performance of SFRC elements and LSFRC beams was compared and analyzed using the concrete plastic damage model (CDP) in ABAQUS software. This study validated the rationality of the model and investigated the impact of varying steel fiber volume ratios and steel fiber layer heights on the damage evolution of LSFRC arches. The results of this study demonstrate that the cracking load and bearing capacity of an RC arch can be effectively enhanced through the addition of steel fibers to a local area under static loading. Furthermore, the deflection and damage to the arch waist and arch roof can be significantly reduced. Furthermore, the incorporation of steel fibers at an increased volume rate and at a greater height within the doped section can effectively slow the rate of damage evolution within the section. This results in the inhibition of crack extensions and in an improvement in the ductility and reliability of the damage stage. The LSFRC arches offer superior economic and practical advantages over their full cross-section doped steel fiber (FRC) counterparts. This study offers novel insights and methodological guidance for the design and implementation of concrete arch bridges.

Keywords: concrete arch; steel fiber-reinforced concrete; CDP model; load-bearing performance; damage evolution



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1. Introduction

As traffic volumes increase and the frequency of overloaded vehicles escalates, reinforced concrete (RC) arch bridges are encountering increasingly severe external overloading challenges. These challenges necessitate improved material properties for the main arch ring, which is now particularly vulnerable to localized cracking damage, compromising the overall performance and safety of the structure [1]. The integration of steel fiber-reinforced concrete (SFRC) offers a compelling solution to address cracking damage within the constraints of the structural cross-section size [2]. The superior tensile strength of steel fibers is strategically employed to mitigate the stresses at the tips of micro-cracks in the concrete, which are induced by external loads. This approach effectively curtails the propagation of cracks, thereby enhancing the structural load-bearing capacity, toughness, and impact resistance [3].

In the current state of the art, the incorporation of steel fibers or high-performance concrete in the full section of RC arch bridges is a widely adopted method of design and construction. Although this method has been demonstrated to enhance structural

performance, its high cost renders it suboptimal from an economic standpoint. Furthermore, it is unable to achieve an optimal balance between structural performance optimization and economic benefits [4–6]. To address this issue, some scholars have proposed the local incorporation of steel fiber (LSFRC) method, which involves the local incorporation of steel fiber in the key cross-section area of concrete structures. This approach aims to optimize structural performance and achieve a balance between cost and benefit.

Wang et al. [7] proposed a novel pavement structure composite consisting of concrete and steel fiber-reinforced concrete. This composite design addresses the inherent limitations of conventional concrete while leveraging the high strength, impact resistance, and wear resistance of steel fiber concrete. In related studies, Wang Yimin and Jamwal [8,9] examined steel fiber composite pavements with various volume ratios. Their research demonstrated that such composites not only retain the original strength of concrete but also exhibit superior mechanical properties, increased toughness, and greater economic advantages. This methodology provides a cost-effective strategy for the design and construction of durable composite pavements. Additionally, Sinha D et al. [10] showed that integrating an optimal quantity of steel fibers into concrete pavements significantly prolongs their lifespan, particularly under conditions of heavy traffic. The research of He et al. [11] indicates that composite steel fiber concrete exhibits greater flexibility, allowing for adjustments in accordance with the thickness of the underlying steel fiber concrete layer and the quantity of steel fiber admixture, contingent on the specific geological characteristics. This approach facilitates the rational utilization of resources and optimizes structural performance.

Zhao et al. [12] conducted theoretical analyses on the load-carrying capacity of composite steel fiber-reinforced concrete beams and derived an iterative method for calculating the steel fiber boundary layer. Research by S.K. Padmarajaiah et al. [13,14] indicated that incorporating steel fibers into the tensile region of beams could yield tensile strengths comparable to those achieved through full cross-sectional incorporation. Additionally, localized fiber incorporation not only enhances structural performance but also offers the benefits of cost effectiveness and the absence of adverse effects. Hang et al. [15] used FEM ABAQUS software to perform a study on plastic damage in ply-fabricated steel fiber concrete beams. The findings showed that while the impact on the elastic phase was minimal, there was a significant increase in the ultimate load-carrying capacity compared to ordinary reinforced concrete (RC) beams. Do, Thi My Dung et al. [16] performed experiments on double-layered fiber-reinforced concrete (FRC) beams and conducted numerical simulations while considering material nonlinearities with ANSYS software. Wu et al. [17] introduced a localized composite structure by combining steel and steel fiber concrete (LCSSFRC), which was evaluated through four-point bending tests. This study assessed the effects of various parameters, such as steel fiber volume fraction, specimen length, and the thickness of the concrete protective layers on both the upper and lower flanges of the compressed steel sections. Additionally, Arif, Mohammed et al. [18] explored the bending performance of deep steel fiber-reinforced concrete beams with varying depths of fiber layers. The results revealed that both load and deflection decreased with reduced fiber layer depth. Incorporating steel fibers into the bottom quarter of the tensile zone achieved equivalent stiffness, ductility, and load-carrying capacity as full cross-sectional fiber integration, saving 20% of the costs.

The preceding analysis indicates that the current research on the local inclusion of steel fibers is predominantly focused on reinforced concrete beams and roadbed pavements. With regard to RC arch bridges, the current research is largely concentrated on the ultimate bearing capacity [19–21], corrosion [22] and reinforcement [23,24]. However, the mechanical behavior of RC arches following a local incorporation of steel fibers is rarely reported. In particular, the issue of how to introduce steel fibers according to the damaged area of the RC arch remains to be further studied.

The objective of this paper is to investigate the bearing performance and damage evolution of an LSFRC arch under static load. Firstly, the application of steel fiber-reinforced concrete in the CDP model is described, and the static properties of SFRC units and LSFRC

beams are compared and analyzed. Subsequently, the RC arch model was constructed using ABAQUS finite element software, and the steel fiber inclusion area in the LSFRC arch was identified through damage identification. The bearing capacity and damage evolution trend of the reinforced concrete arch, steel fiber-reinforced concrete arch, and low-strength fiber-reinforced concrete arch were compared and analyzed. Finally, the impact of varying steel fiber volume ratios and cross-section steel fiber heights on the LSFRC arch was demonstrated.

2. Material Constitutive Models

2.1. Steel Fiber-Reinforced Concrete

Understanding and quantifying the interactions between steel fibers and the concrete matrix presents a significant scientific challenge in the study of the mechanical properties of steel fiber-reinforced concrete. The incorporation of steel fibers markedly influences the concrete's behavior, particularly under tensile and compressive stresses. This impact is primarily governed by the steel fibers' volume fraction, shape, orientation, and their bond strength with the concrete matrix. Despite the growing utilization of steel fiber-reinforced concrete, the academic community has yet to establish a unified constitutive model that accurately represents its complex mechanical behavior. This is particularly true when describing the elastoplastic behavior of the material under both compressive and tensile conditions, where existing models remain incomplete. Consequently, the selection of an appropriate constitutive model for steel fibers is crucial for accurately predicting the mechanical responses of steel fiber-reinforced concrete under various loading scenarios, which is essential for forecasting its structural performance.

2.1.1. Axial Tensile Stress–Strain Relationship

The stress–strain relationship of steel fiber-reinforced concrete under axial tensile state is calculated using the formula proposed in the literature [25], as shown in Equations (1) and (2):

$$y = \begin{cases} Ax + (3 - 2A)x^2 + (A - 2)x^3 & 0 \leq x \leq 1 \\ \frac{x}{a(x-1)^{1.7} + x} & x > 1 \end{cases} \quad (1)$$

$$x = \varepsilon / \varepsilon_{t0}; y = \sigma / f_{t0} \quad (2)$$

where x and y are the horizontal and vertical coordinates of the stress–strain curve, respectively; f_{t0} and ε_{t0} are the tensile strength and the corresponding peak tensile strain of the matrix concrete, respectively; σ and ε are the tensile stress and tensile strain of the steel fibers, respectively; A is the shape parameter of the stress–strain curve, and its specific expression is shown in Equation (3); a is the parameter related to the characteristic parameters of the matrix concrete and the steel fibers, and its expression is shown in Equations (4) and (5) [25].

$$A = 1.4 \frac{f_{t0} \left(\varepsilon_{t0} + 0.96 v_f \frac{l}{d} \times 10^{-4} \right)}{\varepsilon_{t0} \left(f_{t0} + 0.92 v_f \frac{l}{d} \right)} \quad (3)$$

$$a = \frac{x_{fc} (1 + y_{fc})}{(x_{fc} - 1)^{1.7}} \quad (4)$$

The values of x_{fc} and y_{fc} in Equation (4) are calculated with Equation (5) below.

$$x_{fc} = \frac{2.5 \varepsilon_{t0} + 0.8 v_f \frac{l}{d} \times 10^{-4}}{\varepsilon_{t0} + 0.96 v_f \frac{l}{d} \times 10^{-4}}; y_{fc} = \frac{0.35 f_{t0} + 0.7 v_f \frac{l}{d} \times 10^{-4}}{f_{t0} + 0.92 v_f \frac{l}{d} \times 10^{-4}} \quad (5)$$

where x_{fc} and y_{fc} are the horizontal and vertical coordinates of the critical failure point, respectively. v_f is the steel fiber volume percentage; l and d are the length and diameter of the steel fiber, respectively.

2.1.2. Axial Compressive Stress–Strain Relationship

The constitutive behavior of steel fiber-reinforced concrete under axial compression adopts the constitutive equation proposed in the literature [26], whose expression is shown in Equation (6) [26]:

$$y = \begin{cases} \frac{A_1 x - x^2}{1 + (A_1 - 2)x} & 0 \leq x \leq 1 \\ \frac{B_1 x}{1 + (B_1 - 2)x + x^2} & x > 1 \end{cases} \quad (6)$$

$$A_1 = \frac{(2.99\sqrt{f_c} + 14.27)(1.46 + 25.4v_f + 0.00779f_c)}{f_c} \quad (7)$$

$$B_1 = 2.0027 + 45.9744v_f - 0.0197f_c \quad (8)$$

where A_1 and B_1 denote the parameters related to the volume coefficients of matrix concrete and steel fibers, respectively, and their expressions are shown in Equations (7) and (8); f_c is the compressive strength of the matrix concrete.

2.2. Matrix Concrete

In this analysis, the Concrete Damage Plasticity (CDP) model implemented in FEM ABAQUS software is utilized to define the tensile and compressive constitutive relationships of concrete. This model is particularly suited for simulating the stiffness degradation of concrete during the load-holding process. The stress–strain relationship for the concrete, as defined in the model, is adopted from reference [27]. The corresponding curve is illustrated in Figure 1.

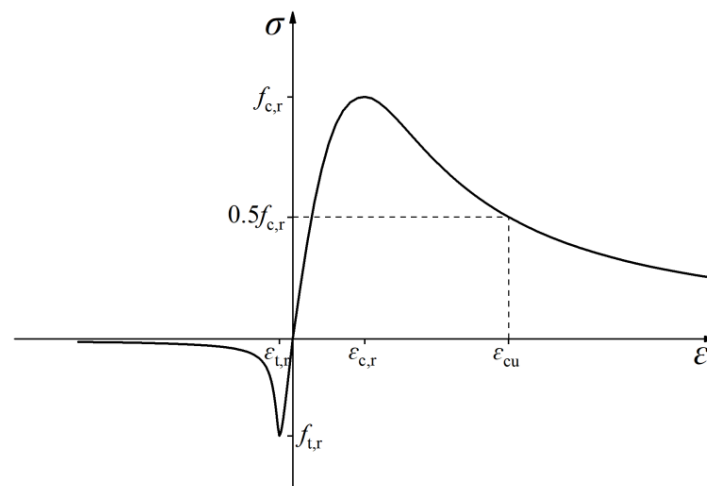


Figure 1. Stress–strain curve of matrix concrete.

The uniaxial tensile stress–strain curve for the matrix concrete is determined by applying Equations (9) through (11) [27]:

$$\sigma = (1 - d_t)E_c \varepsilon \quad (9)$$

$$d_t = \begin{cases} 1 - \rho_t [1.2 - 0.2x^5] & x \leq 1 \\ 1 - \frac{\rho_t}{\alpha_t(x-1)^{1.7} + x} & x > 1 \end{cases} \quad (10)$$

$$x = \frac{\varepsilon}{\varepsilon_{t,r}} \quad \rho_t = \frac{f_{t,r}}{E_c \varepsilon_{t,r}} \quad (11)$$

where α_t is the parameter values for the descending segment of the concrete's uniaxial tensile stress–strain curve; $f_{t,r}$ is the representative value of concrete's uniaxial tensile strength; $\varepsilon_{t,r}$ is the peak tensile strain of concrete associated with the representative value of uniaxial tensile strength; d_t is the parameter for the evolution of damage under uniaxial tension in matrix concrete.

Similarly, the uniaxial compressive stress–strain curve for the matrix concrete is determined by applying Equations (12) through (14) [27]:

$$\sigma_c = (1 - d_c) E_c \varepsilon_c \quad (12)$$

$$d_c = \begin{cases} 1 - \frac{\rho_c n}{n-1+x^n} & x \leq 1 \\ 1 - \frac{\rho_c}{\alpha_c (x-1)^2 + x} & x > 1 \end{cases} \quad (13)$$

$$\rho_c = \frac{f_{c,r}}{E_c \varepsilon_{c,r}}; \quad n = \frac{E_c \varepsilon_{c,r}}{E_c \varepsilon_{c,r} - f_{c,r}}; \quad x = \frac{\varepsilon}{\varepsilon_{c,r}} \quad (14)$$

where α_c is the parameter values for the descending segment of the matrix concrete's uniaxial compressive stress–strain curve; $f_{c,r}$ is the representative value of concrete's uniaxial compressive strength; $\varepsilon_{c,r}$ is the peak compressive strain of concrete associated with the representative value of uniaxial tensile strength; d_c is the parameter for the evolution of damage under uniaxial compressive in matrix concrete.

When performing structural analysis with the CDP model, it is imperative to ensure the precision of the input parameters. While the material stress–strain curve data can be fitted using the aforementioned equations, the resultant data are not directly applicable as input for the software. Instead, a conversion must be executed according to Sidiroff's energy equivalence principle, as illustrated in Equations (15) and (16) [28]. This methodology facilitates a more precise simulation of the material's behavior under diverse loading conditions, thereby bolstering the reliability and accuracy of the simulation outcomes.

$$d_k = 1 - \sqrt{\frac{\sigma_k}{E_0 \varepsilon_k}}; \quad k = t, c \quad (15)$$

$$\varepsilon_t^{in} = \varepsilon_t - \varepsilon_{0t}^{el} \quad (16)$$

where d_k ($k = t, c$) comprises the factors of tension ($k = t$) and compression ($k = c$) damage; σ_k is true stress; E_0 is the initial elastic modulus; ε_k is true strain; ε_{0t}^{el} is the tensile cracking strain, which is defined as the total strain minus the undamaged elastic strain of the material; and ε_t^{in} is the tensile inelastic strain that needs to be input into ABAQUS.

In accordance with the uniaxial compression and tensile constitutive relationship of concrete, as presented in the literature [28], the corresponding stress–strain curve ($\sigma_k - \varepsilon_k^{in}$) and damage factor ($d_k - \varepsilon_k^{in}$) curve can be fitted, as illustrated in Figure 2. The parameter is designated as $d_k \in [0, 1]$, where $d_k = 0$ indicates no damage and $d_k = 1$ represents complete damage. The uniaxial compressive strength ($f_{c,r}$) and tensile strength ($f_{t,r}$) of matrix concrete are 22.4 MPa and 1.83 MPa, respectively.

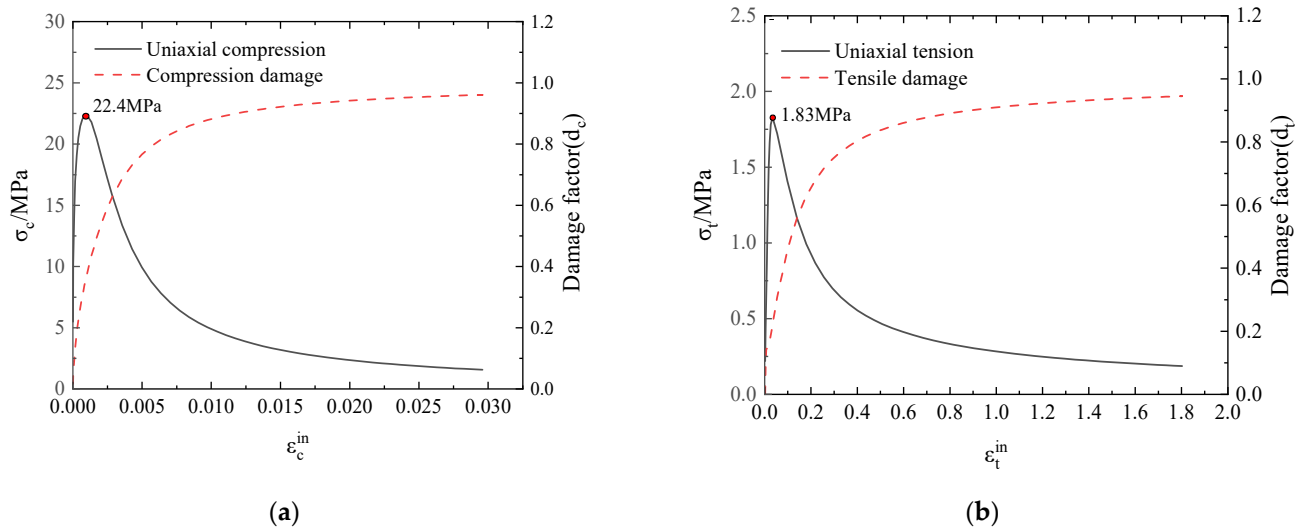


Figure 2. Stress–strain curve for damage to matrix concrete: (a) uniaxial compressive; (b) uniaxial tensile.

2.3. Rebar

The elastic/plastic intrinsic model is utilized to characterize the mechanical behavior of a steel bar. Prior to reaching the yield strength, the material exhibits linear elasticity. However, once the yield strength is exceeded, it transitions into the plastic phase, where the stress of the material increases in direct proportion to the strain. Please refer to Figure 3 for the stress–strain curve of the rebar.

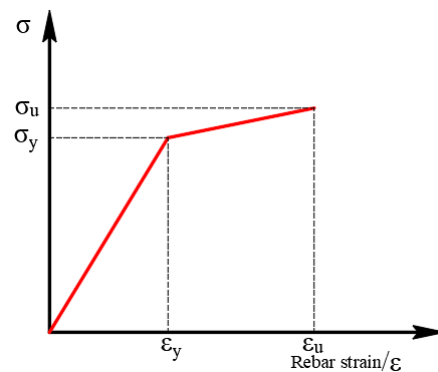


Figure 3. Stress–strain curve of rebar.

3. Verification of the Constitutive Model

The initial validation of the material model is a crucial step in conducting numerical simulation studies to ensure the reliability and accuracy of the computational model. In order to verify the accuracy of the values obtained from the concrete constitutive model described above, numerical simulations are conducted for the mechanical properties of steel fiber concrete materials, including SFRC unit tests and LSFRC beam bending performance tests for comparison.

3.1. SFRC Unit Tests

According to the description of the material intrinsic validation in the ABAQUS manual [29], preliminary validation of the steel fiber concrete CDP model mentioned above was conducted by subjecting a single unit to uniaxial tensile and compressive loads. The uniaxial compressive and tensile strengths of the matrix concrete are specified as $f_c = 22.4$ MPa and $f_t = 1.83$ MPa, respectively, with a steel fiber volume fraction $v_f = 1\%$, aspect ratio $l_f/d_f = 60$, modulus of elasticity $E = 3.45 \times 10^4$ MPa, Poisson's ratio set at 0.2, and using C3D8R cell type for analysis. As shown in Figure 4, the stress–strain comparison

curves of finite element and theoretical calculations when a single unit is subjected to compression and tension are shown, which are in good agreement, indicating that the adopted steel fiber concrete principal structure can be used in the subsequent analytical calculations.

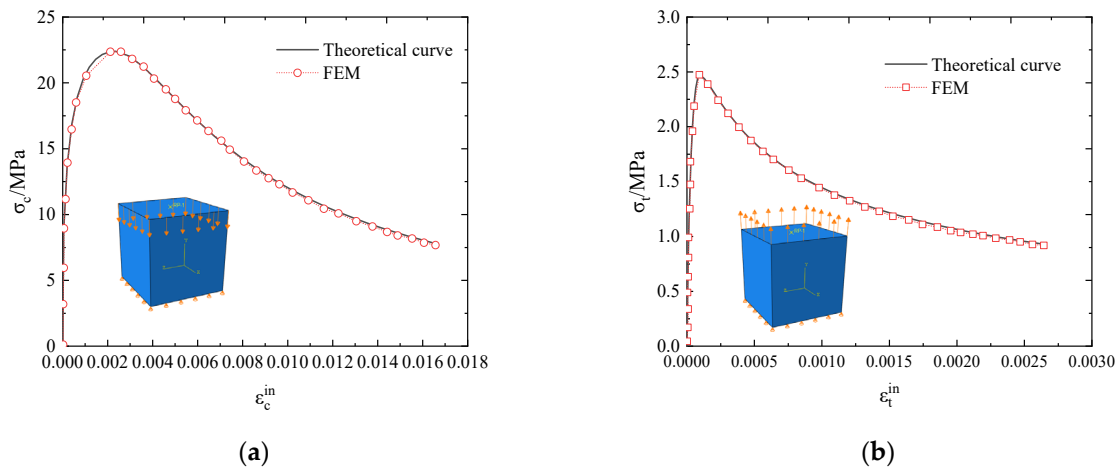


Figure 4. Validation of SFRC cell stress–strain curves: (a) axial compressive curve; (b) axial tensile curve.

3.2. LSFRC Beam Performance Test

The verification object selected for this study is the laminated fabric steel fiber concrete beam subjected to static testing, as documented in the literature [15]. To establish a 3D solid model, the structural dimensions of the beam are set to 150 mm × 150 mm × 550 mm in FEM ABAQUS software. A four-point bending load method is employed, wherein the PC and FRC units are represented by C3D8R units. Both units are embedded within the reinforcement bars using the embedded region contact method in the FRC solid model. Displacement coupling is utilized for the loading method. The material parameters used in the analysis are presented in Table 1.

Table 1. Material parameters [15].

Material	Compressive Strength/MPa	Tensile Strength/MPa	Elastic Modulus/GPa	Poisson's Ratio
RC	35.6	2.82	39	0.2
SFRC	37.3	3.54	41	0.2

As illustrated in Figure 5, the CDP model adopted in this paper was employed to predict the load–displacement curve of the layered steel fiber-reinforced concrete tension-bent beam, as presented in the literature [15], based on the test values and finite element analysis results. The simulation results presented in this paper are in close agreement with the experimental data observed in the rising section of the curve, when compared with the values reported in the literature. Secondly, following the peak load, the CDP model is able to accurately predict the reduction in stiffness of the structure, resulting in a distinct decline in the curve. As the load is increased further, the proportion of load borne by steel fibers in the tensile crack zone of concrete increases. This has the effect of slowing down the expansion of the crack and improving its toughness. Consequently, the test curve rises once more. Finally, the simulation results of the CDP model demonstrate a gradual tendency towards stability, which is in good agreement with the experimental values. This indicates that the model is reliable and accurate.

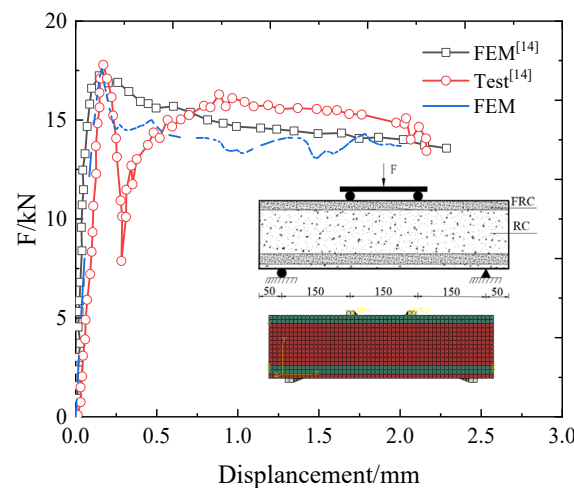


Figure 5. Comparison of load–displacement curves of LSFRC beams.

4. Calculation of Damage to RC Arch

4.1. Model Introduction

By examining the design dimensions of the main arch rings of reinforced concrete arch bridges domestically and internationally [1,30,31], it is evident that the main arch ring typically adopts a single-box double-compartment cross-section. The cross-section height usually ranges from 2.8 m to 4.6 m, while the width is between 6 m and 10 m. These dimensions generally conform to a rib height of 1/40 to 1/60 of the span diameter and a rib width of 0.5 to 2 times the rib height. For lateral stability, the width-to-span ratio varies from 1/18.75 to 1/30.32, meeting the general construction standard of a width-to-span ratio of 1/20.

The results of the aforementioned research indicate that the proposed calculation model arch rib span is 3000 mm, the vector to span ratio is 1/5, the arch axis coefficient $m = 1.988$, the arch rib section is simplified, and the proposed calculation model adopts a solid rectangular section, with dimensions of 200 mm $\times$ 150 mm (width $\times$ height). The protective layer thickness of the reinforcement is 10 mm, and the reinforcement is HRB400. The diameters of the longitudinal reinforcement N1 and stirrup are 8 mm and 6 mm, respectively, the two extremities of the arch spandrel serve as fixed supports, and the specific sizes are illustrated in Figure 6.

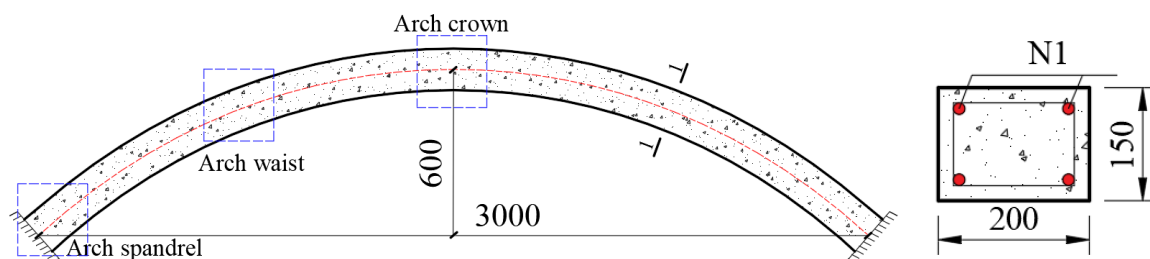
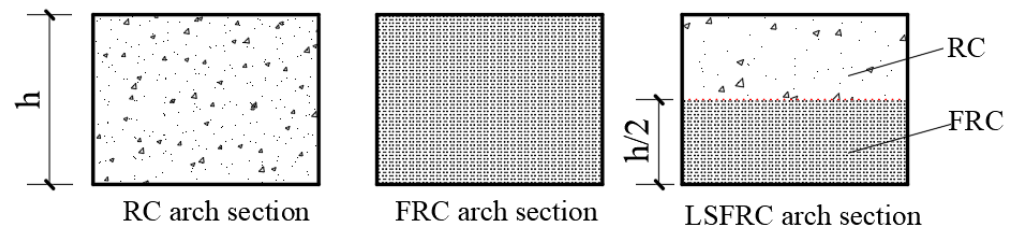


Figure 6. Calculation model of RC arch (unit: mm).

In order to ascertain the degree of influence of incorporating steel fiber on cracking load, ultimate bearing capacity, and damage evolution of RC arch, three types of calculation models were established, as shown in Table 2. Figure 7 presents a schematic diagram of the various ways in which steel fibers can be incorporated into the cross-section.

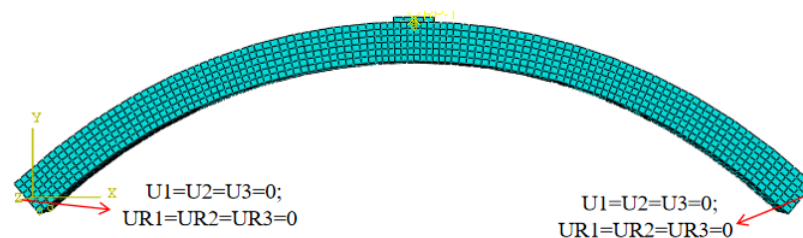
Table 2. Finite element model.

Model		Remarks
1	RC arch	Ordinary reinforced concrete hingeless arch
2	FRC arch	Uniform distribution of steel fibers throughout the arch section
3	LSFRC arch	Layering of steel fibers in the damaged cross-section area

**Figure 7.** Schematic of each model cross-section.

4.2. Calculation of RC Arch Damage Evolution

A distinct model was constructed within the ABAQUS software framework, utilizing the geometric dimensions of the arch rib depicted in Figure 6. The concrete material was represented using a three-dimensional hexahedral reduced integration element (C3D8R) with a unit size of 30 mm. Given the absence of consideration for adhesive slip, a two-node linear truss element (T3D2) was employed to simulate the steel, with a unit size of 40 mm. In order to ensure a connection between the concrete and steel, an embedded contact method was employed. Furthermore, a tie constraint was applied to constrain the concrete and loading pad. The loading pad was subjected to static loading through displacement coupling at its center. The concrete arch ring was modelled as a cantilever arch, with the end arch feet simulated as fixed supports ($U_1 = U_2 = U_3 = 0$; $UR_1 = UR_2 = UR_3 = 0$). The finite element model is depicted in Figure 8.

**Figure 8.** Finite element model for static loading of RC arch vaults.

In this model, the compressive and tensile strengths of the matrix concrete are set at $f_c = 22.4$ MPa and $f_t = 1.86$ MPa, respectively. The volume fraction of steel fibers is designated as $v_f = 1\%$, with a length-to-diameter ratio of $l_f/d_f = 60$. The yield stress of the steel reinforcement bar is 335 MPa, the ultimate stress is 432 MPa, and the elastic modulus is 203 GPa. The loading pad is defined as an elastic rigid body. The concrete damage model employs the CDP model, wherein the values for inelastic strain and damage factor ($d - \epsilon_{t,c}^{in}$) are calibrated according to the formula provided in Section 2 and are subsequently inputted. Table 3 presents the remaining plasticity parameters of the CDP model.

Table 3. The plasticity parameters of CDP model for RC and FRC materials [32].

Expansion Angle	Eccentricity	Compressive Strength Ratio	K-Factor	Coefficient of Viscosity
30	0.1	1.16	0.6667	0.0005

Figure 9 shows the tensile damage region along the arch axis of the RC arch at the moment of ultimate load-carrying capacity. When the arch is subjected to a concentrated load, the RC arch first shows cracking damage at the lower edge of the arch crown, with a cracking load of 17.2 kN, which gradually expands to the upper edge with an increase in the load. As the load reached 32.1 kN, both sides of the arch spandrel appeared damage cracking; at the same time, the top plate position of the arch waist also appeared to be subjected to slight tensile damage. At a holding load of 39 kN, the upper edge of the arch waist appeared to be subjected to obvious tensile damage. From the damage evolution process of the whole RC arch, it can be seen that when the arch is subjected to a concentrated load, the lower edge of the arch spandrel, the upper edge of the arch waist, and the lower edge of the arch crown are the key cross-sections prone to damage, and the corresponding damage areas when it reaches the peak load are $[0, L/8]$, $[L/8, 3L/8]$, and $[3L/8, 5L/8]$, with the trend of cracking damage evolution being shown in Figure 10.

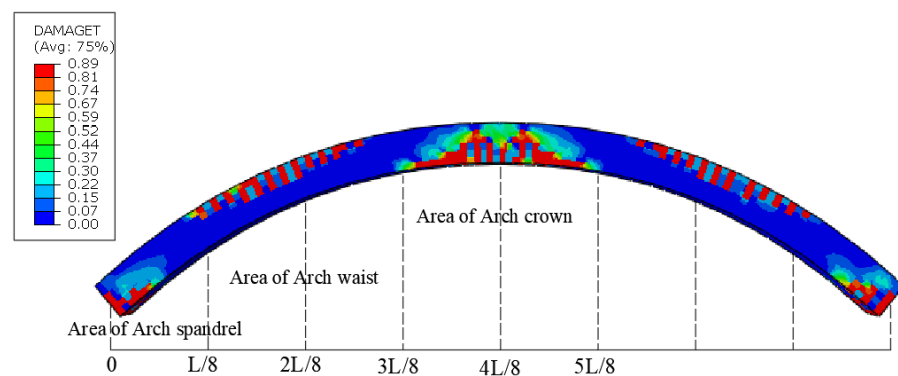


Figure 9. Regional distribution of RC arch damage.

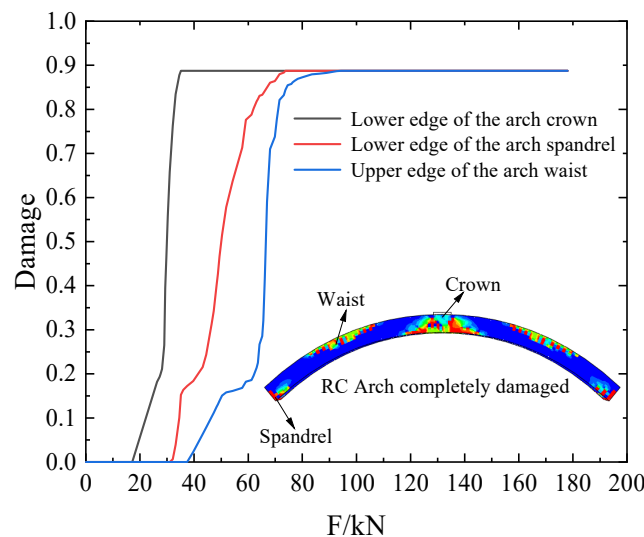


Figure 10. Damage evolution of critical sections of RC arches.

4.3. Calculation of LSFRC Arch Damage Evolution

Consequently, the specific location of the damage to the critical cross-section of the RC arch previously mentioned can be employed as a foundation for localized steel fiber doping, with the objective of enhancing the stress performance of the vulnerable cross-section of the RC arch through localized reinforcement. Therefore, the finite element model of steel fiber doping for the LSFRC arch is depicted in Figure 11.

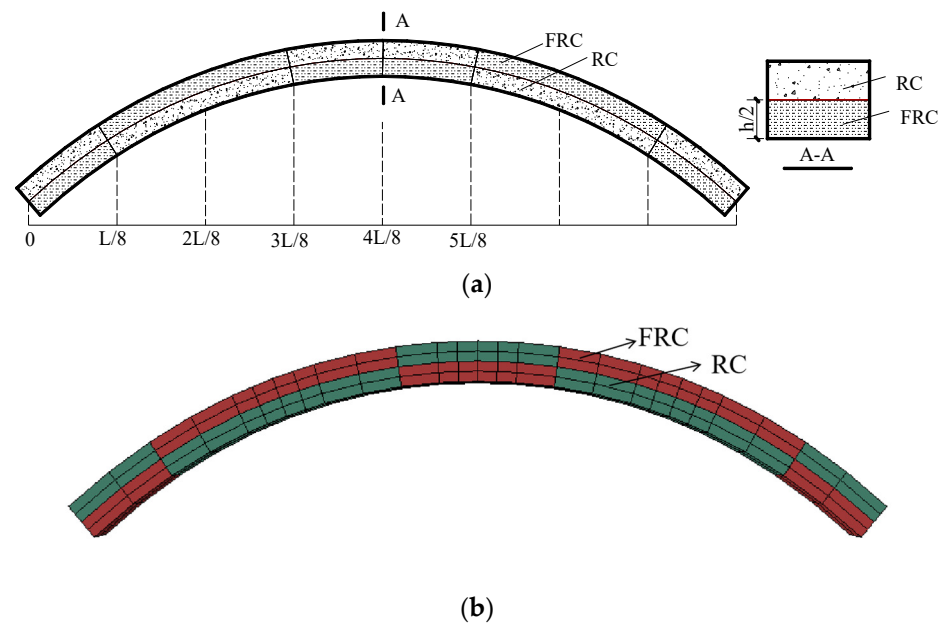


Figure 11. LSFRC arch calculation model: (a) a diagrammatic representation of the location of the incorporation section for steel fiber; (b) ABAQUS finite element model.

As outlined in Section 4.2, the damage locations of the foot, arch waist, and vault sections of the RC arch can be utilized as a foundation for local steel fiber incorporation. Local reinforcement can be implemented in the areas of $[0, L/8]$, $[L/8, 3L/8]$, and $[3L/8, 5L/8]$, with the objective of enhancing the stress performance of the damage-prone sections of the RC arch. Consequently, the schematic diagrams of steel fiber incorporation in LSFRC arches and the established FEM (Model 3) are presented in Figure 11.

Figure 12 illustrates the results of a comparison of load–displacement curves for an RC arch, an LSFRC arch, and an FRC arch. Among these, the cracking loads of the LSFRC arch and the FRC arch are 18.8 kN and 19.4 kN, respectively, which are 9.3% and 12.7% higher than the cracking load of the RC arch, which is 17.2 kN. Furthermore, their peak loads are 190 kN and 224.2 kN, representing 6.6% and 25.7% increases compared to 178.3 kN for the RC arch, respectively. The analysis indicates that the incorporation of steel fibers into the vulnerable cross-section area of the RC arch does not significantly enhance the cracking load, but it does markedly improve the load-carrying capacity. Upon reaching the peak load, the FRC arch curve does not exhibit a significant downward segment. This phenomenon can be attributed to the uniform distribution of steel fibers over the entire cross-section of the FRC arch, which results in a higher toughness of the structure at this stage as well as a higher deformation capacity after the peak load. In contrast, in the LSFRC arch, the vulnerable cross-section forces are shared by both the FRC and RC layers. Consequently, the falling section of the load–displacement curve exhibits a trend similar to that of the RC arch, yet it is more ductile after being damaged and requires a higher amount of energy dissipation.

Figure 13 illustrates the deflection curves of the RC arch, LSFRC arch, and FRC arch along the span direction under a load of 178.3 kN. As illustrated in Figure 13, the deflection of the RC arch, LSFRC arch, and FRC arch at the vault section ($l/2$) is 4.97 mm, 4.14 mm, and 2.56 mm, respectively, while the deflection at the waist section ($l/4$) is 1.56 mm, 1.1 mm, and 0.65 mm, respectively.

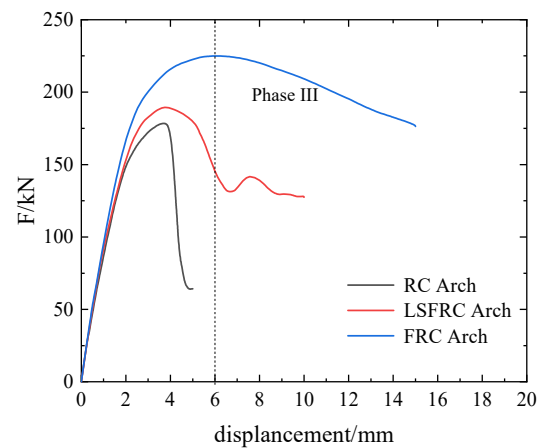


Figure 12. Comparison of load–displacement curves of RC arch with steel fiber in different sections.

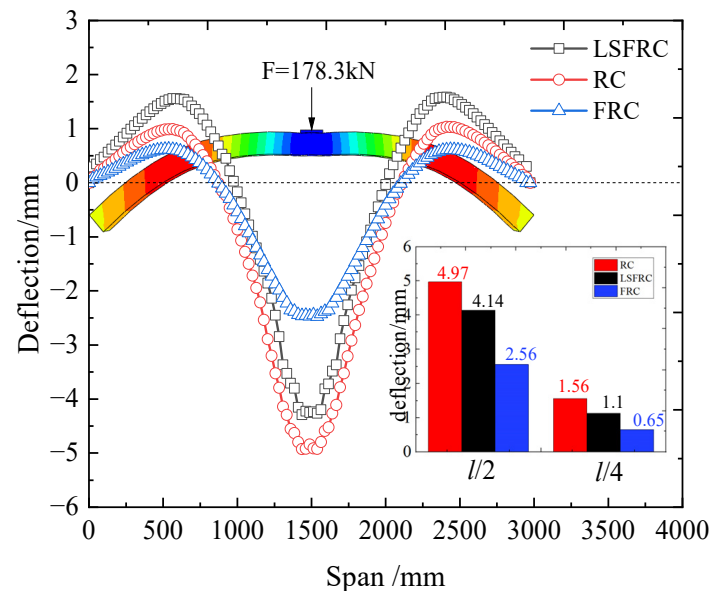


Figure 13. Deflection curves of RC, LSFRC, and FRC arches along the span direction.

Figures 14 and 15 illustrate the evolution of damage in the RC, LSFRC, and FRC arches following deformation. In comparison to the RC arch, the crack damage area of the LSFRC and FRC arches is significantly reduced. The analysis of deflection difference and the damage evolution trend indicates that the incorporation of steel fiber can enhance the ultimate bearing capacity of RC arches whilst also significantly reducing the deflection at each section, effectively reducing the degree of damage and the number of cracks. While the full-section incorporation of fibers (FRC arch) demonstrates the greatest enhancement, its economic benefit falls short of that of the LSFRC arch.

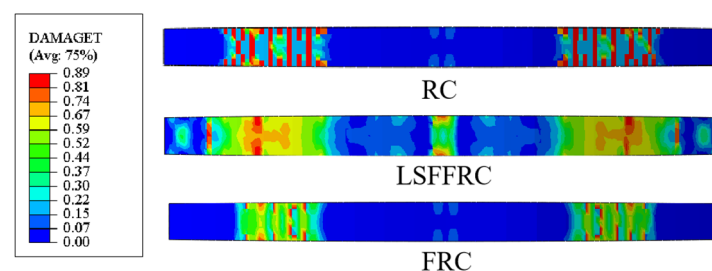


Figure 14. The region of arch extrados damage evolution.

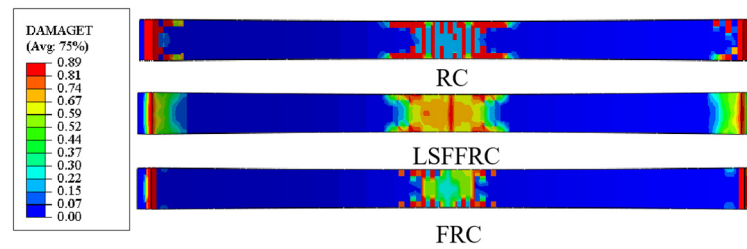


Figure 15. The region of arch intrados damage evolution.

4.4. Parametric Analysis of the LSFRC Arch

A parametric analysis of the LSFRC arch was conducted, utilizing the volume fraction (v_f) and the height of admixture along the cross-section (h_v) as variables. The steel fiber coefficients were set to 1%, 2%, and 3%, respectively, and the height of admixture along the cross-section h_v was set to $h/4$, $2h/4$, and $3h/4$, respectively. The results of the calculations are presented in Figure 16.

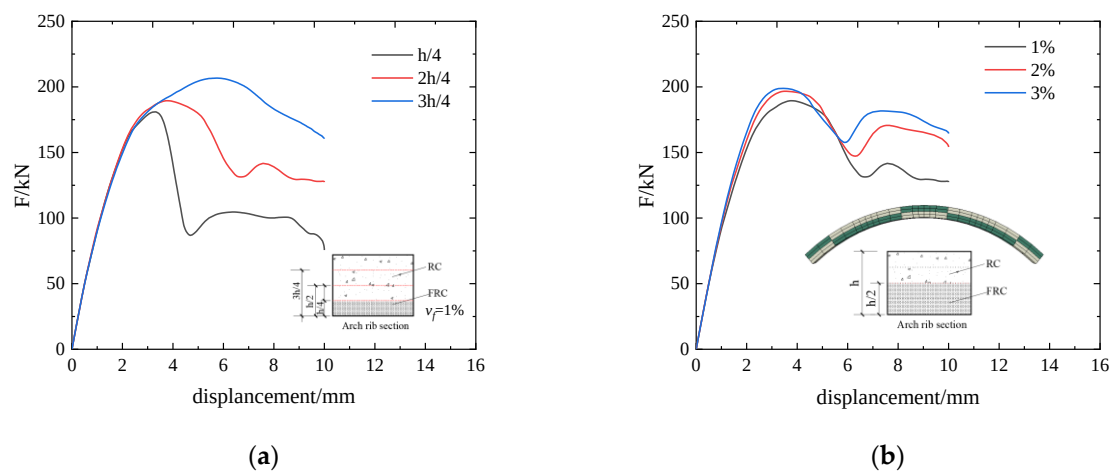


Figure 16. Comparison of load–displacement curve parameters of LSFRC arch: (a) different heights along the cross-section; (b) different steel fiber volume rates.

As illustrated in Figure 16a, the load-carrying capacity of steel fiber-reinforced lightweight concrete (LSFRC) arches was observed to increase with the height of doping across the cross-section. The ultimate load capacities at doping height positions of $h/4$, $2h/4$, and $3h/4$ were measured at 181 kN, 190 kN, and 206.5 kN, respectively. These results demonstrate that a gradient increase in the steel fiber concentration along the height of the cross-section significantly enhances structural stress performance, thereby improving the tensile strength and toughness of the reinforced concrete (RC) arch. Figure 16b presents the load–displacement curves for LSFRC arches with varying steel fiber volume ratios at $2h/4$. While the differences in the load capacity are relatively minor, it is evident that increases in steel fiber volume enhance structural ductility during the post-peak phase, resulting in higher energy absorption during the damage phase.

As illustrated in Figure 16a, the incorporation height of LSFRC arch steel fiber at $h/4$, $2h/4$, and $3h/4$ results in ultimate bearing capacities of 181 kN, 190 kN, and 206.5 kN, respectively. As the steel fiber content increases along the section, the ultimate bearing capacity increases by 5.6% and 14.7% compared to the LSFRC- $h/4$. This demonstrates that the addition of steel fiber along the section height can effectively enhance the mechanical properties of the structure, improving the bearing capacity and toughness of the RC arch. Figure 16b displays the load–displacement curves of the LSFRC- $2h/4$ arch at varying steel fiber volume rates of 1%, 2%, and 3%. As illustrated in Figure 1, the carrying capacity of the specimen was found to be 190 kN, 196.6 kN, and 198.8 kN, respectively. Upon reaching a volume rate of 3%, the lifting effect of the volume rate on the carrying capacity of the LSFRC

arch begins to diminish. Nevertheless, in the subsequent decline phase, following the peak load, an increase in the steel fiber volume rate is accompanied by a more ductile response of the LSFRC arch to failure, although this is accompanied by a greater energy consumption.

As illustrated in Figure 17, this study comparatively analyzed the damage evolution of LSFRC-2h/4 and LSFRC-3h/4 arches at the location of the arch footing under the same load using the LSFRC-h/4 arch structure at a peak load of 181 kN. The analysis results indicate that at the initial stage of damage (damage state value of 0.2), the corresponding damage rates of each stage are not significantly different. Upon reaching a load of 181 kN, the degree of damage of the three components at the foot of the arch cross-section is 0.89, 0.76, and 0.61, respectively. The results demonstrate that, with the increase in the height of steel fiber doping, the cross-section damage evolution rate of LSFRC arches is effectively slowed down. This can inhibit crack extensions more effectively and slow down the process of structural attenuation. Consequently, an optimal increase in the cross-sectional height when incorporating steel fiber can markedly decelerate the process of RC arch damage and enhance structural reliability.

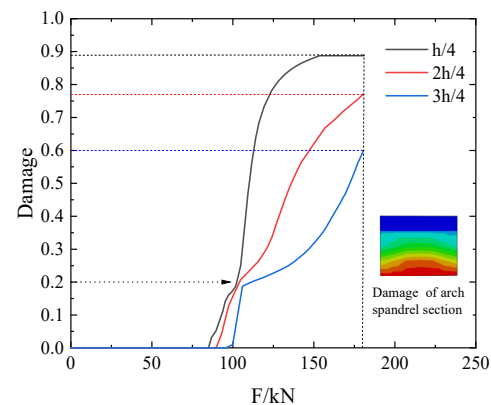


Figure 17. Damage evolution trend of LSFRC arch spandrel with different steel fiber incorporation heights.

As illustrated in Figure 17, the damage evolution of the arch foot position of the LSFRC-20/4 and LSFRC-3h/4 arches under the same load was compared and analyzed based on the peak load of the LSFRC-h/4 arch, which was found to be 181 kN. The results of the analysis indicate that when the damage is small (damage state value is 0.2), the corresponding damage rates are not different. However, the corresponding cracking loads at the foot edge of the arch are 86.8 kN, 93 kN, and 99 kN, respectively, which are increased by 7.1% and 14.1% compared with the LSFRC-h/4 arch. Upon reaching a load of 181 kN, the degree of damage of the three components at the arch foot section is 0.89, 0.76, and 0.61, respectively. This indicates that with the increase of steel fiber height, the damage evolution rate of the LSFRC arch section is effectively slowed down. This can more effectively inhibit crack expansion and delay the decay process of the structure. It follows, therefore, that an appropriate increase in section height and the steel fiber incorporation volume rate can significantly delay the failure process of reinforced concrete (RC) arches, enhancing structural dependability.

5. Conclusions

This study proposes a new computational model, namely the distributed steel fiber concrete arch (LSFRC), which considers the distribution of damage to the RC arch. The objective is to achieve a balance between the structural performance and economic benefit of the full-section steel fiber arch (FRC). The rationality of the LSFRC arch was verified through a numerical simulation of the concrete plastic damage model (CDP) in ABAQUS software. Furthermore, the influence of different steel fiber volume ratios and steel fiber

layer heights on the performance of the LSFRC arch was deeply discussed. The research results are as follows:

1. In the context of static arch loading, the damage to the RC arch section manifests as cracking at the lower edge of the arch crown, the foot edge of the arch, and the upper edge of the arch waist. The corresponding stress damage areas are $[3L/8, 5L/8]$, $[0, L/8]$, and $[L/8, 3L/8]$, respectively.
2. A local incorporation of steel fibers can effectively increase the cracking load and bearing capacity of the RC arch, reduce the deflection of the arch waist and vault, and improve the damaged area of the section. The damage evolution rate of the LSFRC arch section slows down significantly with the increase in the steel fiber volume ratio and the height of the doped section, and crack propagation is effectively inhibited, thus improving the ductility and reliability of the failure stage.
3. In terms of a balance in performance, a local incorporation of steel fibers enhances the performance of RC arches while reducing the quantity of materials employed in comparison to the full-section incorporation of steel fiber (FRC) arch. Consequently, the LSFRC arch is more economical and practical. This research study offers a novel concept and methodology for the design and implementation of concrete arch bridges, which are anticipated to be widely adopted in future engineering practices.

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