Effect of steel fibers and the fiber-concrete adhesion stress on the nonlinear shear behavior of beams

Efeito das fibras de aço e da tensão de adesão fibra-concrete no comportamento de cisalhamento não linear de vigas

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ABSTRACT

In this article, a theoretical model in nonlinear elasticity is presented to analyze the influence of the percentage of steel fibers and the effect of the fiber-concrete bond stress on the shear force at the rupture of beams subjected to the combined effect of bending moment, normal force, and shear force. For a given beam section, it is defined by a succession of layers of concrete and longitudinal steel elements. Each layer is defined by its height hi, width bi, and position relative to one end of the section YGi. Each longitudinal steel element is also defined by its cross-sectional area and position relative to one end of the section. The steel fibers are defined by volume percentages of 0.5%, 1%, 1.5%, and 2%, taking into account the mechanical nonlinearity of the materials. This model is based on the multilayer analysis of sections and an iterative solution procedure for each layer and each section, considering a given longitudinal deformation state and shear stress. The global equilibrium of the sections is analyzed under the assumption of flat longitudinal deformations but with, in principle, an interdependence of longitudinal normal stresses and shear stresses. In this study, using the principle of virtual work, equilibrium equations for deformations and stresses, as well as partial compatibility equations between concrete deformations and mean deformations, are derived. Comparative examples between ordinary reinforced concrete beams with variable shapes and reinforcement details, and those reinforced with steel fibers, are presented to demonstrate the accuracy of the proposed model for simulating the nonlinear shear response of beams.
Keywords: shear, steel fibers, reinforced concrete, adhesion stress, beams.

RESUMO
Neste artigo, é apresentado um modelo teórico de elasticidade não linear para analisar a influência da porcentagem de fibras de aço e o efeito da tensão de ligação fibra-concreto na força de cisalhamento na ruptura de vigas sujeitas ao efeito combinado de momento fletor, força normal e força de cisalhamento. Para uma determinada seção de viga, ela é definida por uma sucessão de camadas de concreto e elementos de aço longitudinais. Cada camada é definida por sua altura hi, largura bi e posição em relação a uma extremidade da seção YGi. Cada elemento de aço longitudinal também é definido por sua área de seção transversal e posição em relação a uma extremidade da seção. As fibras de aço são definidas por porcentagens de volume de 0,5%, 1%, 1,5% e 2%, levando em conta a não linearidade mecânica dos materiais. Esse modelo baseia-se na análise multicamada de seções e em um procedimento de solução iterativa para cada camada e cada seção, considerando um determinado estado de deformação longitudinal e tensão de cisalhamento. O equilíbrio global das seções é analisado sob a suposição de deformações longitudinais planas, mas com, em princípio, uma interdependência das tensões normais longitudinais e das tensões de cisalhamento. Nesse estudo, usando o princípio do trabalho virtual, as equações de equilíbrio para deformações e tensões, bem como as equações de compatibilidade parcial entre as deformações do concreto e as deformações médias, são derivadas. Exemplos comparativos entre vigas de concreto armado comuns com formas e detalhes de reforço variáveis e aquelas reforçadas com fibras de aço são apresentados para demonstrar a precisão do modelo proposto para simular a resposta de cisalhamento não linear de vigas.

Palavras-chave: cisalhamento, fibras de aço, concreto reforçado, tensão de adesão, vigas.

1 INTRODUCTION
Since the early 1960s, significant research has been conducted on fiber-reinforced concrete. Several authors, including (Bartos, 1981; Spatney, 1972; Bischoff, 2001; Biswas et al., 2021; Chu; Conway, 1970; Cohen; Romualdi, 1967; Furlan; De Hanai, 1997; Krenchel, 1974; Lantsoght, 2023; Narayanan; Darwish, 1987; Roop, 1984; Rossi, 1992; Zhao; Chen; Huang, 2023), have contributed to the utilization of fibers as a means of reinforcing concrete.

The shear problem in concrete constructions has long been the subject of experimental and theoretical studies in several laboratories and research centers around the world. Lim and Oh (1999) presented study experimental and theoretical investigation on the shear of steel fiber reinforced concrete beams. Dinh et al. (2011) studied shear behavior of steel fiber-reinforced concrete beams.

Most sections in reinforced concrete or fiber-reinforced concrete are calculated using the linear formalism of classical elasticity theory. However, numerous tests conducted on these sections and constituent materials suggest that obtaining an exact representation of section deformability through linear calculation is not possible. The purpose of this work is to develop a calculation model based on nonlinear elasticity to study the behavior, including failure, of sections of reinforced concrete beams with steel fibers under shear force, taking into account the laws of nonlinear material behavior.

In this study, by utilizing equilibrium equations are obtained via the principle of virtual work and compatibility equations in stresses and strains, this system of equations is solved using the iterative method (step-by-step method). The objective of our work is to study the effect of steel fibers and the fiber-concrete adhesion stress on the nonlinear shear behavior of beams.

2 PROBLEM FORMULATION
2.1 MATERIAL BEHAVIOR
2.1.1 Behavior of concrete under compression in the D2 direction

The behavior of the concrete in compression, is a nonlinear elastic behavior which given by the law of Sargin (SARGIN, 1971)
\[ \sigma = f_{cj} \frac{k_b \varepsilon - (k_b - 1)\varepsilon^2}{1 + (k_b - 2)\varepsilon - k_b \varepsilon^2} \]  \hspace{1cm} (1)

Where:

\[ \varepsilon = \frac{\varepsilon}{\varepsilon_0} \]  \hspace{1cm} (2)

\( k'_b \) and \( k_b \) : dimensionless parameters fit the descending and ascending branch, respectively, of Sargin’s law

\[ k'_b = k_b - 1, \hspace{0.5cm} k_b = \frac{E_{b0}\varepsilon_0}{f_{cj}} \hspace{0.5cm} E_{b0} = 11000\sqrt{f_{cj}} \]  \hspace{1cm} (3)

Figure 1. Uniaxial constitutive law of the concrete in compression by Sargin

\[ \sigma \quad k'_b = 0 \quad k'_b = k_b - 1 \quad k'_b = 1 \]

Source: Sargin (1971)

Where \( f_{cj} \), \( \varepsilon_0 \) and \( E_{b0} \) are the compressive strength of concrete at age \( j \) peak strain corresponding to \( f_{cj} \) and elastic modulus of concrete, respectively.

The principal stress \( \sigma_{b2} \) is a function of the two principal strains \( \varepsilon_1 \) and \( \varepsilon_2 \)

\[ \sigma_{b2} = E_{b2}(\varepsilon_2, \varepsilon_1) \cdot \varepsilon_2 \]  \hspace{1cm} (4)

Using the relationship proposed by Vecchio and Collins (1986) for the behavior of concrete in the compressed strut according to D2 as a function of strain \( \varepsilon_1 \), while constraining the ratio \( f_{c2}/f_c \):
\[
\frac{f_{c2}}{f_c} = \frac{1}{1 - 0.34\varepsilon_1/\varepsilon_{b0}} \quad 0.7 \leq \frac{f_{c2}}{f_c} \leq 1
\] (5)

2.1.2 Behavior of concrete under tension in the D1 direction

Several models are proposed in the literature. In the context of this study, we used Grelat's law (Grelat, 1978)

The fibers of the stretched concrete are affected by a deformation modulus \( E_{bt} \) which is defined from the stress and the instantaneous strain of the stretched edge

\[
E_{bt} = \frac{\sigma_{bt}}{\varepsilon_{bt}}
\] (6)

Beyond the cracking in tension one takes account of a participation of the tense concrete located between two successive cracks. The stress does not suddenly vanish, but decreases according to a parabolic law (see Figure 2). One thus studies the average behavior of a zone of beam.

Figure 2. Stress – instantaneous deformation diagram of concrete on the tension edge Grelat (1978)

Where \( f_{tj} \), \( \varepsilon_{ft} \) and \( \varepsilon_{rt} \) are the tensile strength of concrete, tensile strain corresponding to \( f_{tj} \) and strain corresponding to the plasticization of the most tense steel, respectively.

\[
\sigma_{bt} = E_{b0} \cdot \varepsilon_{bt} \quad \text{for} \quad |\varepsilon_{bt}| < |\varepsilon_{rt}|
\] (7)
cracking with participation of tensile concrete

\[ \sigma_{bt} = -f_{ct} \cdot \frac{(\varepsilon - \varepsilon_{rt})^2}{(\varepsilon_{rt} - \varepsilon_{ft})^2} \quad \text{for} \quad \varepsilon_{ft} < |\varepsilon_{bt}| < \varepsilon_{rt} \quad (8) \]

cracking without tension concrete participation

\[ \sigma_{bt} = 0 \quad \text{for} \quad |\varepsilon_{bt}| > |\varepsilon_{ft}| \quad (9) \]

Using the relationship proposed by Belarbi and Hsu (1994) for the behavior of concrete in the tension strut according to D1

\[ \sigma_1 = E_{b0} \cdot \varepsilon_1 \quad \text{for} \quad |\varepsilon_1| < |\varepsilon_{ft}| \quad (10) \]

\[ \sigma_{bt} = f_t \cdot \left(\frac{\varepsilon_{ft}}{\varepsilon_1}\right)^{0.4} \quad \text{for} \quad \varepsilon_{ft} < |\varepsilon_1| < \varepsilon_{rt} \quad (11) \]

### 2.1.3 Steel fiber concrete tensile constitutive law

The stress-strain law of the fiber concrete in tension is linear before the cracking of the concrete. After cracking, the behavior is elastic nonlinear, is given by Kachi (2006).

Before concrete cracking:

\[ \sigma = E_{cr} \varepsilon_1 \quad \text{for} \quad |\varepsilon_1| < \varepsilon_{ft} \quad (12) \]

After concrete cracking:

\[ \sigma = \sigma_{uc} - (\sigma_{uc} - f_{ft}) \frac{(\varepsilon_1 - \varepsilon_u)^6}{(\varepsilon_{ft} - \varepsilon_u)^6} \quad \text{for} \quad \varepsilon_{ft} < |\varepsilon_1| < \varepsilon_u \quad (13) \]

\[ \sigma = \sigma_{uc} \left(1 - \frac{(\varepsilon_1 - \varepsilon_u)^6}{(\varepsilon_{rt} - \varepsilon_u)^6}\right) \quad \text{for} \quad \varepsilon_u < |\varepsilon_1| < \varepsilon_{rt} \quad (14) \]

\[ \sigma = 0 \quad \text{for} \quad |\varepsilon_1| > \varepsilon_{rt} \quad (15) \]
With \( E_{cr}, \sigma_{uc} \) and \( f_{ft} \) are the initial modulus of the composite in tension, fictitious maximum stress of the composite in the ultimate state and composite tensile strength, respectively.

\[
E_{cr} = E_{po}(1 + n\theta_0\omega), \quad \sigma_{uc} = \omega\theta_0\frac{lf}{\varphi^2} \tau_u \quad \text{and} \quad f_{ft} = f_{bt}(1 + n\theta_0\omega)
\] (16)

Where:

\( n, \theta_0, \omega, l_f, \varphi, \tau_u \) and \( f_{bt} \) are the steel-concrete equivalence coefficient, fiber orientation factor, fiber percentage, fiber length, diameter of a fiber, fiber-matrix concrete adhesion stress, tensile strength of concrete, respectively.

### 2.1.4 Behavior of natural steel

They are characterized by a perfect elastoplastic law Bael (1999)

\[
\sigma = E_u \varepsilon \quad \text{for} \quad \varepsilon \leq \varepsilon_e
\]

(17)

\[
\sigma = \sigma_e \quad \text{for} \quad \varepsilon_e < \varepsilon < \varepsilon_u
\]

(18)

\[
\sigma = 0 \quad \text{for} \quad \varepsilon > \varepsilon_u
\]

(19)

The extreme deformation are fixed by the BAEL 1999 regulation at 10‰, were \( E_u, \varepsilon_e, \sigma_e \) and \( \varepsilon_u \) are the longitudinal modulus of elasticity of steel, elastic limit strain of steel and ultimate strain steel, respectively.

### 2.2 STUDY OF THE EQUILIBRIUM OF A BEAM IN NONLINEAR ELASTICITY

In nonlinear elasticity, the study of the equilibrium of a reinforced concrete beam with steel fibers involves solving a system of equations of the form:

\[
[Dp] = [K].[\Delta U]
\]

(20)

Where:
\( [\Delta P] \) is the vector representing the increment of the applied load on the beam, \( [\Delta U] \) is the vector representing the increment of node displacements of the beam, and \( [K] \) denotes the global stiffness matrix of the beam, which is constructed from the stiffness matrices of the sections \( [K_s] \)

\[
[K_s] = \begin{bmatrix}
\frac{\Delta N}{\Delta \delta u} & \frac{\Delta N}{\Delta \delta w} & 0 \\
\frac{\Delta M}{\Delta \delta u} & \frac{\Delta M}{\Delta \delta w} & 0 \\
0 & 0 & \frac{\Delta V}{\Delta Y_{moy}}
\end{bmatrix}
\] (21)

Where:

\( \Delta N \) is the increment of the normal force of the section, \( \Delta M \) is the increment of the bending moment of the section, \( \Delta \delta u \) is the increment of the deformation at the center of gravity of the section, and \( \Delta \delta w \) is the increment of the differential rotation (curvature) of the section.

The longitudinal deformation \( \varepsilon_x \) is linear function of the transverse coordinate \( y \) (Navier-Bernoulli assumption).

\[
\varepsilon_x = \delta u + \delta w \cdot y
\] (22)

\( \Delta V \) is the increment of the shear force of the section, and \( \Delta Y_{moy} \) is the increment of the average distortion of the section.

2.3 MODELING PRINCIPLE AND GENERAL EQUATION

2.3.1 Discretization of the beams

The beam is discretized into segments assumed to be sufficiently large relative to the crack spacing. Deformations within a straight section are then expressed by the representative average values of the deformation field of the beam element. Studying the equilibrium of a segment involves examining the equilibrium of multiple cross-sectional areas. Each cross-sectional area is discretized into a succession of layers of concrete and longitudinal steel elements (see Figure 3).
The straight transverse reinforcements are assumed constant over the entire height of the section and distributed along the entire length of the segment; their area is then expressed as a percentage of the concrete area. Each layer of concrete and each longitudinal steel element are analyzed separately, but the equilibrium condition of the section is satisfied globally.

In reinforced concrete shear design, the truss model composed of inclined struts at angle $\theta$ and transverse reinforcements (see Figure 4), subjected to longitudinal sliding force per unit length $G = V/Z$, and allows for the calculation of tensile forces in the transverse reinforcements.

$$F_{at} = G \tan \theta$$

(23)

straight reinforcements in the y direction and compression in the concrete struts

$$F_{bc} = G / \cos \theta$$

(24)
2.3.2 Strain equations

The fundamental relationships between average deformations are as follows:

\[ \varepsilon_x = \varepsilon_1 \sin^2 \theta + \varepsilon_2 \cos^2 \theta \] (25)

\[ \varepsilon_y = \varepsilon_1 \cos^2 \theta + \varepsilon_2 \sin^2 \theta \] (26)

\[ \gamma = 2(\varepsilon_1 - \varepsilon_2) \sin \theta \cos \theta \] (27)

The principal direction D1 corresponds to the largest principal deformation, while direction D2 corresponds to the smallest principal deformation. The angle between D2 and the longitudinal axis x is denoted by:

\[ \theta = (\vec{G}_{D_2}, \vec{G}_x) \] (28)

In the specific directions of x and y, we have:

\[ \varepsilon_{ax} = \varepsilon_x \] (29)

\[ \varepsilon_{ay} = \varepsilon_y \] (30)

Connected to the deformations of tense and cracked concrete, the average deformity corresponds to the sum of two terms: the its own average deformation of the concrete \( \varepsilon'_b \), linked to its average stress \( \sigma_b \), and the distributed effect of the opening cracks \( w \) spaced by \( S \):

\[ \varepsilon_b = \varepsilon'_b + \frac{w}{s} \] (28)
2.3.3 Under stress

The concrete must independently balance the shear stresses \( \tau \) parallel to \( x \) and \( y \) (neglecting stresses that would be balanced by the "dowel effect" of reinforcements). The general relations between the stresses of the concrete are:

\[
\sigma_{bx} = \sigma_{b1} \sin^2 \theta_b + \sigma_{b2} \cos^2 \theta_b
\]

(29)

\[
\sigma_{by} = \sigma_{b1} \cos^2 \theta_b + \sigma_{b2} \sin^2 \theta_b
\]

(30)

\[
\tau_b = \tau = (\sigma_{b1} - \sigma_{b2}) \sin \theta_b \cos \theta_b
\]

(31)

We assume that the principal stress directions of the concrete coincide with the principal directions of the average strains

\[
\theta_b = \theta
\]

(32)

2.3.4 Equations of partial compatibility: Concrete strains – Mean strains

We assume that the strains of the concrete in the principal directions \( \varepsilon_{b1} \) and \( \varepsilon_{b2} \), which are related to the principal stresses \( \sigma_{b1} \) and \( \sigma_{b2} \), are equal to the principal mean strains:

\[
\varepsilon_{b1} = \varepsilon_1
\]

(33)

\[
\varepsilon_{b2} = \varepsilon_2
\]

(34)

2.3.5 Equations equilibrium

2.3.5.1 Local equilibrium of the layer

In the \( x \) and \( y \) directions, the stresses \( \sigma_x \) and \( \sigma_y \) result from the sum of the terms due to reinforcements and concrete, taking into account the respective areas:
\[ \sigma_x = \rho_x \sigma_{ax} + \sigma_{bx} \]  
\[ (35) \]

\[ \sigma_y = \rho_y \sigma_{ay} + \sigma_{by} = 0 \]  
\[ (36) \]

2.3.5.2 Global equilibrium of the section

The equilibrium must be ensured between the external loads \( N, M, V \) and the internal stress resultants \( N_{int}, M_{int}, V_{int} \). For combined bending, the sum of the contributions of longitudinal stresses in the reinforcements and in the concrete is considered.

\[ N_{int} = \sum_i A_i \sigma_{axi} + \sum_i b_i h_i \sigma_{bxi} \]  
\[ (37) \]

\[ M_{int} = \sum_i A_i \sigma_{axy} y_i + \sum_i b_i h_i \sigma_{bxy} y_i \]  
\[ (38) \]

For shear force, only the shear stresses in the concrete are considered.

\[ V_{int} = b_i h_i \tau_{bi} \]  
\[ (39) \]

2.4 CALCULATION STEPS

Studying the equilibrium of a layer of reinforced concrete or fiber-reinforced concrete with known values of \( \varepsilon_x \) and \( \tau \) requires the use of a system of 10 equations with 10 unknowns: \( \varepsilon_y, \varepsilon_1, \varepsilon_2, \gamma, \theta, \sigma_{ay}, \sigma_{bx}, \sigma_{by}, \sigma_{b1} \) and \( \sigma_{b2} \). To solve this system, an iterative method is employed. Given a specific distribution of longitudinal strains and assuming \( \varepsilon_2 \) as known, we seek the value of the angle \( \theta \) that satisfies the compatibility and equilibrium conditions of the layer.

The tangential stress \( \tau(y) \) are calculated by the equilibrium of two neighboring sections. The longitudinal strain \( \varepsilon_x \) is derived from Navier's equation, where \( \delta u \) and \( \delta w \) result from the overall equilibrium in composite bending under \( N, M \), with coupling to the shear force \( V \). Finally, the medium distortion of the section is calculated based on the efforts in the successive layers.
3 NUMERICAL RESULTS AND DISCUSSION

The developed model was used to analyse the shear behaviour of several beams tested by various researchers at the University of Toronto (Vecchio, 1982). The beams had either solid or hollow rectangular sections made of reinforced concrete. The geometric characteristics and properties of the different materials are provided in Table 1.

<table>
<thead>
<tr>
<th>Beam</th>
<th>Dimensions</th>
<th>Concrete</th>
<th>Transverse Reinforcements</th>
<th>Longitudinal Reinforcements</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>OUT mm</td>
<td>INN mm</td>
<td>fc Mpa</td>
<td>Bar Ø (mm)</td>
</tr>
<tr>
<td>SA3</td>
<td>305x610</td>
<td>152x406</td>
<td>40.0</td>
<td>9.5</td>
</tr>
<tr>
<td></td>
<td>4 x 22</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SA4</td>
<td>305x610</td>
<td>152x406</td>
<td>40.0</td>
<td>9.5</td>
</tr>
<tr>
<td></td>
<td>4 x 22</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SK3</td>
<td>305x610</td>
<td>-</td>
<td>28.2</td>
<td>9.5</td>
</tr>
<tr>
<td>SK4</td>
<td>305x610</td>
<td>121x381</td>
<td>28.2</td>
<td>9.5</td>
</tr>
</tbody>
</table>

Source: Vecchio (1982)

The beams are reinforced with steel fibers at a percentage ranging from 0.5 to 2% by volume. The properties of the fibers are provided in Table 2.

<table>
<thead>
<tr>
<th>Elastic modulus</th>
<th>Fiber length</th>
<th>Fiber diameter</th>
<th>Ultimate strain</th>
<th>Fiber orientation factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_a$ (Mpa)</td>
<td>$l_f$ (mm)</td>
<td>$\phi$ (mm)</td>
<td>$\varepsilon_u$ (%)</td>
<td>$\theta$</td>
</tr>
<tr>
<td>2 105</td>
<td>60</td>
<td>1</td>
<td>0.74</td>
<td>0.405</td>
</tr>
</tbody>
</table>

Source: Authors

The results obtained are presented in the form of comparative curves which represent the influence of the steel fibers and the fiber-concrete adhesion stress of the beams. It should be noted that this is the relationship shear force V – distortion average of a beam zone subjected to shear (possibly combined with normal effort).

The variation of ultimate shear force as a function of fiber percentage with a fiber-concrete adhesion stress of $\tau=7$ MPa, the percentage of fibers varied from 0.5% to 2%, is represented in Tables 3 for beams SA4, SK3 and SK4.
Table 3. Variation of ultimate shear force as a function of fiber percentage with a fiber-concrete adhesion stress of \( \tau = 7 \) Mpa for beams SA4, SK3 and SA4

<table>
<thead>
<tr>
<th>Beams</th>
<th>Fiber percentage w(%)</th>
<th>Ultimate shear force V(KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SA4</td>
<td>0% (Reinforced concrete)</td>
<td>552.853039</td>
</tr>
<tr>
<td></td>
<td>0.5%</td>
<td>567.071434</td>
</tr>
<tr>
<td></td>
<td>1%</td>
<td>571.078592</td>
</tr>
<tr>
<td></td>
<td>1.5%</td>
<td>575.085751</td>
</tr>
<tr>
<td></td>
<td>2%</td>
<td>579.092909</td>
</tr>
<tr>
<td>SK3</td>
<td>0% (Reinforced concrete)</td>
<td>690.566155</td>
</tr>
<tr>
<td></td>
<td>0.5%</td>
<td>735.444175</td>
</tr>
<tr>
<td></td>
<td>1%</td>
<td>750.151961</td>
</tr>
<tr>
<td></td>
<td>1.5%</td>
<td>762.391576</td>
</tr>
<tr>
<td></td>
<td>2%</td>
<td>779.567533</td>
</tr>
<tr>
<td>SK4</td>
<td>0% (Reinforced concrete)</td>
<td>613.06644</td>
</tr>
<tr>
<td></td>
<td>0.5%</td>
<td>630.799274</td>
</tr>
<tr>
<td></td>
<td>1%</td>
<td>636.127748</td>
</tr>
<tr>
<td></td>
<td>1.5%</td>
<td>641.456222</td>
</tr>
<tr>
<td></td>
<td>2%</td>
<td>646.784696</td>
</tr>
</tbody>
</table>

Source: Authors

Figure 5. Shear force-deformation curve of the SK4 beam with an adhesion stress \( \tau = 7 \) Mpa

Source: Authors
Figure 6. Shear force-deformation curve of the SK3 beam with an adhesion stress $\tau=7$ Mpa

Source: Authors

Figure 7. Shear force-deformation curve of the SA4 beam with an adhesion stress $\tau=7$ Mpa

Source: Authors
Figure 8. Shear force-deformation curve of the SA3 beam with an adhesion stress $\tau=7$ Mpa

![Shear force-deformation curve of the SA3 beam with an adhesion stress $\tau=7$ Mpa](image)

Source: Authors

Figure 9. The effect of the fiber-concrete adhesion stress on the shear behavior of the SA3 beam with a fiber percentage $w=0.5\%$

![The effect of the fiber-concrete adhesion stress on the shear behavior of the SA3 beam with a fiber percentage $w=0.5\%$](image)

Source: Authors
Figure 10. The effect of the fiber-concrete adhesion stress on the shear behavior of the SA4 beam with a fiber percentage w=2%.

![Graph showing shear force vs. strains for different fiber-concrete adhesion stresses.]

Source: Authors

Figure 11. The effect of the fiber-concrete adhesion stress on the shear behavior of the SK4 beam with a fiber percentage w=1%.

![Graph showing shear force vs. strains for different fiber-concrete adhesion stresses.]

Source: Authors
The variation of the ultimate shear force as a function of concrete fiber-concrete adhesion stress, with different percentages of fibers, is represented in Tables 4 and 5 for beams SA4 and SK4.

Table 4. Variation of ultimate shear force as a function of fiber-concrete adhesion stress with different fiber percentage values for beam SA4

<table>
<thead>
<tr>
<th>Fiber percentage w(%)</th>
<th>adhesion stress τ(Mpa)</th>
<th>Ultimate shear force V(KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0% (Reinforced concrete)</td>
<td>/</td>
<td>552.853039</td>
</tr>
<tr>
<td>0.5%</td>
<td>7</td>
<td>567.071434</td>
</tr>
<tr>
<td></td>
<td>10.5</td>
<td>569.367784</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td>571.664134</td>
</tr>
<tr>
<td>1%</td>
<td>7</td>
<td>571.078592</td>
</tr>
<tr>
<td></td>
<td>10.5</td>
<td>575.671292</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td>580.263992</td>
</tr>
<tr>
<td>1.5%</td>
<td>7</td>
<td>575.085751</td>
</tr>
<tr>
<td></td>
<td>10.5</td>
<td>581.974801</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td>588.863851</td>
</tr>
<tr>
<td>2%</td>
<td>7</td>
<td>579.092909</td>
</tr>
<tr>
<td></td>
<td>10.5</td>
<td>588.278309</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td>597.463709</td>
</tr>
</tbody>
</table>

Source: Authors

Table 5. Variation of ultimate shear force as a function of fiber-concrete adhesion stress with different fiber percentage values for beam SK4

<table>
<thead>
<tr>
<th>Fiber percentage w(%)</th>
<th>adhesion stress τ(Mpa)</th>
<th>Ultimate shear force V(KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0% (Reinforced concrete)</td>
<td>/</td>
<td>613.06644</td>
</tr>
<tr>
<td>0.5%</td>
<td>7</td>
<td>630.799274</td>
</tr>
<tr>
<td></td>
<td>10.5</td>
<td>633.830917</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td>636.862561</td>
</tr>
<tr>
<td>1%</td>
<td>7</td>
<td>636.127748</td>
</tr>
<tr>
<td></td>
<td>10.5</td>
<td>642.191035</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td>648.254322</td>
</tr>
<tr>
<td>1.5%</td>
<td>7</td>
<td>641.456222</td>
</tr>
<tr>
<td></td>
<td>10.5</td>
<td>650.551153</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td>659.646083</td>
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<tr>
<td>2%</td>
<td>7</td>
<td>646.784696</td>
</tr>
<tr>
<td></td>
<td>10.5</td>
<td>658.91127</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td>671.037845</td>
</tr>
</tbody>
</table>

Source: Authors

We can see that the value of the ultimate shear force increases with the increase in fibers percentage on the one hand. On the other hand, this value also increases with the increase in the fiber-matrix adhesion stress of concrete. The improvement in the shear force is observed in all figures, in the last phase of the behavior of the reinforced concrete beams, namely, after the plasticization of the tension reinforcements (see Figure. 5, 6, 7, 8, 9, 10 and 11).
4 CONCLUSIONS

We conclude that the developed model is capable of estimating the behavior of sections under shear until failure. It is also capable of simulating failure both in bending and in shear, specifically in the case of reinforced concrete with steel fibers.

The shear behavior of a beam reinforced with steel fibers can be decomposed into three phases:

The first phase (phase 1) occurs before concrete cracking. This phase corresponds to a steep slope of the shear strain curve. It is observed that in this phase, the shear behavior of the beams is not influenced by the presence of fibers.

The second phase (After concrete cracking) is characterized by a significant decrease in the slope of the shear strain curve due to the appearance of the first crack. Shear cracking occurs when the tensile stress perpendicular to the compressed concrete struts reaches the shear force of the concrete. In this phase, there is minimal influence of fibers on the shear force of the beams. The third phase of this behavior is the failure phase, which is reached just after the yielding of the tensile reinforcements. It is noted that in this phase, the presence of steel fibers significantly improves the ultimate shear force of the beams. Additionally, increasing the bond stress between the fiber-concrete adhesion stress enhances the shear force at beam failure.

On an academic level, these research efforts open up new avenues of study and investigation in the field of simulating the nonlinear shear behavior of steel fiber-reinforced concrete beams. Additionally, the findings of this study can lay the groundwork for interdisciplinary research projects involving disciplines such as engineering and industry. Looking forward, there is potential to enhance the current model developed in this study. Proposed improvements could include:

- Exploring enhancements to the current formulation in future work to incorporate different types of fibres.
- Investigating various behaviors of steel fiber-reinforced concrete beams, such as thermomechanical shear and dynamic behavior of beams.
- Examining different shapes and geometries, such as arc beams.
REFERENCES


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