

Article

The Effects of Bond–Slip Laws on the Debonding Failure and Behavior of Flexural Strengthened RC Slabs in Hybrid FRP Retrofit Systems

Huy Q. Nguyen ¹ , Tri N. M. Nguyen ² , Do Hyung Lee ³ and Jung J. Kim ^{1,*} 
¹ Department of Civil Engineering, Kyungnam University, Changwon-si 51767, Korea

² Campus in Ho Chi Minh City, University of Transport and Communications, No. 450-451 Le Van Viet Street, Tang Nhon Phu A Ward, Thu Duc City, Ho Chi Minh City 700000, Vietnam

³ Department of Civil, Railroad and Unmanned Systems Engineering, PaiChai University, 155-40 Baejaero, Seo-gu, Daejeon 35345, Korea

* Correspondence: jungkim@kyungnam.ac.kr; Tel.: +82-552-496-421; Fax: +82-505-999-2165

Abstract: The hybrid retrofit system using FRP and concrete overlay applied on the top of slabs has proven effective in strengthening and overcoming logistical constraints, compared with conventional strengthening techniques using externally bonded composite materials to the underside of the slabs. Nevertheless, the performance of retrofitted slabs is governed by debonding failure due to the low bond strength between CFRP and concrete overlay. Thus, this study investigates the behavior of flexural strengthened slabs with FRP retrofit systems and the effect of bond–slip laws on debonding failure. Firstly, two full-scale RC slabs with and without a retrofit system were tested in a four-point bending setup as the control specimens. Then, the same retrofitted slab was simulated by utilizing the commercial program ABAQUS. A sensitivity analysis was conducted to consider the influence of bond–slip laws to predict the failure mechanism of the retrofitted slabs based on load–deflection relationships. The results showed that the strengthened slab enhanced the load-carrying capacity by 59%, stiffness by 111%, and toughness by 29%. The initial stiffness of $0.1K_0$ and maximum shear stress of $0.13\tau_{max}$, compared with the corresponding values of Neubauer’s and Rostasy’s bond–slip law, can be used to simulate the global response of the retrofitted slab validated by experiment results.

Keywords: CFRP; bond–slip law; debonding; RC slab; retrofit; strengthen



Citation: Nguyen, H.Q.; Nguyen, T.N.M.; Lee, D.H.; Kim, J.J. The Effects of Bond–Slip Laws on the Debonding Failure and Behavior of Flexural Strengthened RC Slabs in Hybrid FRP Retrofit Systems. *Materials* **2022**, *15*, 7453. <https://doi.org/10.3390/ma15217453>

Academic Editor:
Krzysztof Schabowicz

Received: 14 September 2022

Accepted: 20 October 2022

Published: 24 October 2022

Publisher’s Note: MDPI stays neutral with regard to jurisdictional claims in published maps and institutional affiliations.



Copyright: © 2022 by the authors. Licensee MDPI, Basel, Switzerland. This article is an open access article distributed under the terms and conditions of the Creative Commons Attribution (CC BY) license (<https://creativecommons.org/licenses/by/4.0/>).

1. Introduction

Strengthening structures with external bonded materials has become one of the popular choices for rehabilitation and upgrading existing reinforced concrete (RC) structures [1,2]. Researchers and engineers are particularly interested in restoring and strengthening partially damaged RC structures to minimize the impact on environmental deterioration and cost savings [3–5]. Fiber-reinforced polymer (FRP) materials have been widely used in the field of external reinforcement in recent decades because of their lightweight, high tensile strength, and non-corrosion resistance [6–12]. New strengthened techniques and design guidelines have emerged for RC structures using FRP composite materials [13–16].

The conventional application in strengthening RC slab techniques focuses on attaching FRP to their tensile zone to maximize the tensile strength of these composite materials, which have been widely gaining acceptance in practice [17–19]. However, the premature delamination of FRP remains the main disadvantage of the strengthening techniques, as it prevents strengthened slabs from reaching their ultimate load-carrying capacity. In general, sectional and debonding failures are two widespread possible failure modes of FRP-strengthened slabs [20–22]. In sectional failure, either the failure of compressive concrete or the rupture of FRP can be predicted through strain compatibility at its limiting strain [23]. In debonding failure, it is more difficult to determine due to the complicated

failure mechanism and the factors contributing to problems such as concrete cracks and stress concentration at the FRP–concrete interface. Due to this, the behavior of externally bonded FRP in contact with a concrete surface has attracted considerable interest from the civil engineering community [24–26]. In addition, obtaining a well-prepared concrete surface for strengthening FRP on the underside of the RC slab can be a more difficult challenge, or it may not be possible due to logistical constraints.

Consequently, a hybrid retrofit system consists of carbon fiber reinforced polymer (CFRP) and concrete overlay applied on the top surface of the RC slab to overcome the shortcomings of traditionally strengthened solutions rather than exploit the high tensile strength of CFRP, as shown in Figure 1. Previous studies have demonstrated the effectiveness of the retrofit system by increasing the flexural strength and ductility of existing RC slabs. Nevertheless, the results also revealed that debonding failure between the concrete overlay and CFRP was one of the major causes of slabs losing their ultimate load-carrying capacity. Transferring interfacial stress from the slab to FRP or vice versa via a specialized adhesive layer is more efficient than from FRP to overlay members due to the characteristics of the retrofit system, leading to the premature debonding failure of the FRP/overlay interface [27–29]. A sound understanding of the behavior of FRP–concrete interface needs to be developed, with a particular interest in FRP/overlay interface for safe in practical design work.

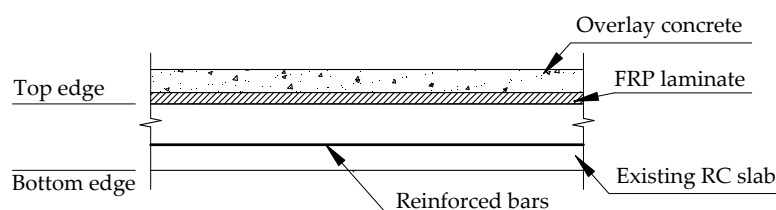


Figure 1. Hybrid FRP retrofit system for positive moment parts of strengthened RC slabs.

In response to this need, the effect of bond–slip laws on the debonding failure of flexural strengthened RC slabs with a hybrid FRP retrofit system is investigated. To the best of our knowledge, previous studies were limited in evaluating the behavior of the externally bonded FRP on a concrete surface without considering cases of CFRP located between two concrete layers of the strengthened slab with a retrofit system [30–37]. There was still considerable uncertainty and difficulty in determining the behavior of CFRP-to-concrete joints due to their intrinsic bond. Insight into an understanding of the bond–slip law between concrete and CFRP using a cohesive approach is a suitable solution for predicting strengthened structural responses [36,38]. Shear stress at the FRP–concrete interface of cohesive elements, in terms of magnitude and distribution, could be a believable explanation for debonding failure [39,40]. It can be difficult to precisely predict the variations in stresses in concrete structures, whereas the load–deflection curve is far less sensitive to crack locations and sizes. The finite element method (FEM) can reasonably predict the interfacial stresses and the delamination load corresponding to deflection more economically than laboratory tests [41]. A finite element (FE) model of the FRP flexural-strengthened slab can provide a comprehensive understanding of the various bond–slip parameters associated with the CFRP/concrete interface and be applicable in developing robust predictive equations for practice designs [42]. By combining FEM through ABAQUS software and the experimental program, the iterative adjustment of the bond–slip parameters can be easily carried out to simulate the experimental load–displacement curve and failure modes, resulting in reduced experiment time and cost [43].

In this study, the behavior of a strengthened RC slab with a hybrid FRP retrofit system is described with particular attention to debonding failure. The effectiveness of the hybrid retrofit system in improving the flexural carrying capacity, stiffness, and toughness is investigated and compared with the full-scale experimental results. A three-dimensional FE model is developed with various bond–slip models for evaluating their accuracy by

comparing numerical predictions with experimental measurements. Numerical models are also analyzed for sensitivity to the bond–slip parameters to assess their effects on the debonding failure. Following the validation of the model, the various parameters that strongly influence the behavior of retrofitted slabs are identified and discussed.

2. FRP Hybrid Retrofit System

2.1. Retrofitting Mechanism

In some cases, strengthening the underside of the reinforced concrete slabs may not be possible due to logistical constraints and hindrances by other utilities. A hybrid retrofit system combining the tensile strength of FRP and compressive strength of the concrete overlay was applied on top of the existing RC slabs to improve their strength and ductility, as shown in Figure 1. The overlay thickness of the retrofit system has a critical role in pulling the neutral axis toward the overlay zone, and FRP holds tension at failure. For this case, the retrofitting mechanism for the hybrid FRP system was estimated as recommended by ACI 440.2R [44], as shown in Figure 2. Assume that the ultimate concrete strain (ϵ_{cu}) is 0.003, and steel yields at yield stress of f_y . Without considering the tension force of concrete, the force equilibrium is calculated as follows:

$$C_H = T_s + T_F \quad (1)$$

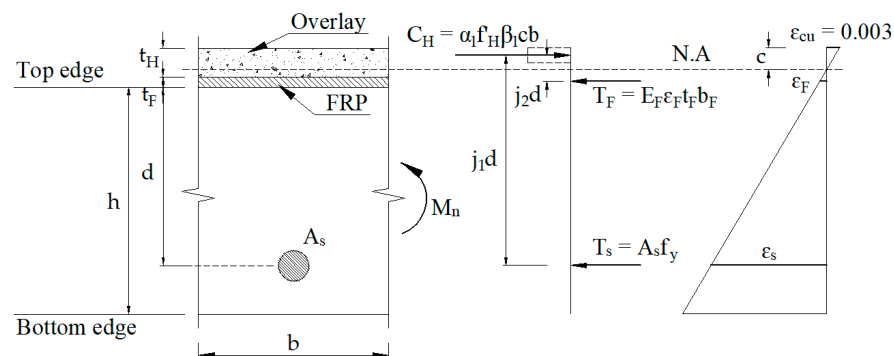


Figure 2. Calculating mechanism for slab's positive moment sections with a proposed system.

These internal forces are computed as follows:

$$C_H = \alpha_1 f'_H \beta_1 c b, T_s = A_s f_y, T_F = E_F \epsilon_F t_F b_F \quad (2)$$

An evaluation of strain compatibility conditions can determine FRP strain at strength limit (ϵ_F) as follows:

$$\epsilon_F = \left[\frac{(t_H + t_F/2) - c}{c} \right] \epsilon_{cu} \quad (3)$$

The moment capacity equilibrium condition can be defined as follows:

$$M_n = T_s(j_1 d) + T_F(j_2 d) \quad (4)$$

2.2. Experimental Program

The reference slab had a 2440 mm total length, 2290 mm clear span, and 130×900 mm² cross-section. The tensile reinforcement was spaced at 185 mm spacing with five No.13 bars ($\phi 12.7$ mm) at the bottom of the RC slab. Transverse steel consisted of six No.10 bars ($\phi 9.5$ mm) spaced at 305 mm center on the center, as shown in Figure 3. The yield stress of the reinforcement used for the RC slab was 400 MPa. The concrete employed for the RC slab had a compressive strength of 27 MPa after 28 days. Table 1 provides the mechanical properties and dimensions of the reference slab.

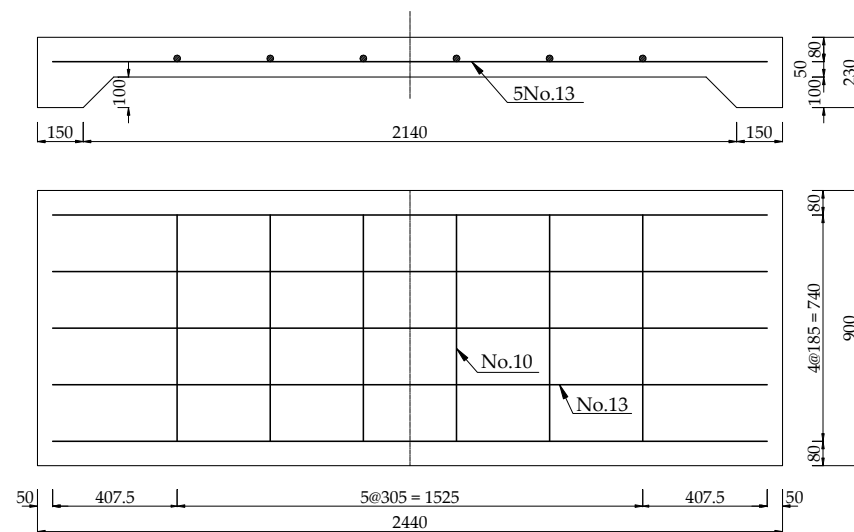


Figure 3. Reference slab reinforcement details.

Table 1. Mechanical properties and dimensions of reference slab.

h (mm)	b (mm)	A_s (mm ²)	d (mm)	f'_c (MPa)	γ_c (kg/m ³)	f_y (MPa)	E_s (GPa)
130	900	632.5	80	27	2400	400	200

A hybrid retrofit system was constructed utilizing a combination of CFRP wet layup laminate with a concrete overlay, as shown in Figure 1. The compressive strength of the concrete overlay was 50 MPa at 28-day age. CFRP had a strength of 600 MPa and an elastic modulus of 40 GPa. The mechanical properties and dimensions used in the FRP retrofit system are given in Table 2.

Table 2. Mechanical properties and dimensions for retrofit system.

Overlay		CFRP				Epoxy Resin		
t_H (mm)	f'_H (MPa)	t_F (mm)	f_{tF} (MPa)	γ_F (kg/m ³)	E_F (GPa)	t_a (mm)	E_a (MPa)	G_a (MPa)
30	50	0.5	600	1200	40	1.5	3000	2100

Two full-scale RC slabs with and without a hybrid retrofit system were tested in a four-point bending setup with two concentrated loads, as shown in Figure 4. Before measuring the deflection data, the LVDTs were reset to remove self-weight deflection. The load cell capacity applied in the tests was 5000 kN. The applied load and deflection at the mid-span section data were recorded during the experimental procedure.

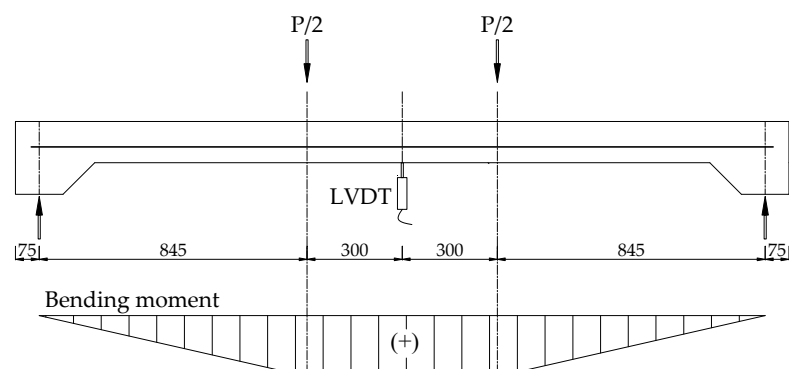


Figure 4. Four-point bending setup for slab and the corresponding moment of the applied load.

2.3. Theoretical Analysis

The experimental data were checked with the predicted analytical moment capacity and deflection at the mid-span due to the applied load of the reference and strengthened RC slabs, as recommended by ACI 440.2R [44] and ACI 318M [45]. Based on the structure, a self-weighted equivalent distributed load is computed as follows:

$$w = \gamma_c b(h + t_H) + \gamma_F b_F t_F \quad (5)$$

The distributed load due to the self-weight of the reference slab and strengthened slab is determined as follows: $W_c = 2.81 \text{ N/mm}$ and $W_s = 3.46 \text{ N/mm}$.

The prediction of the load-carrying capacity corresponding to deflection at mid-span due to the applied load for the reference RC slab is given in Table 3.

Table 3. Analysis of the reference RC slab.

Calculation	Reference Slab
Compressive concrete block depth $a = \frac{A_s f_y}{\alpha_1 f_c b}$	$a = \frac{632.5(400)}{0.85(27)(900)} = 12.25 \text{ mm}$
Moment capacity $M_{n,P} = (d - \frac{a}{2}) A_s f_y$	$M_{n,P} = (80 - \frac{12.25}{2})(632.5)(400) = 18.69 \times 10^6 \text{ Nmm} = 18.69 \text{ kNm}$
Moment due to self-weight $M_c = w_c l^2 / 8$	$M_c = 2.81(2290)^2 / 8 = 1.84 \times 10^6 \text{ Nmm}$
Mid-span section's remaining capacity for carrying additional load $P_{u,theo.} = \frac{2(M_{n,P} - M_c)}{x}$	$P_{u,theo.} = \frac{2(18.69 - 1.84)}{845/1000} = 39.9 \text{ kN}$
Deflection at yield point steel due to the applied load $\Delta_{u,theo.} = \frac{(3l^2 - 4x^2)}{24} \frac{(M_{n,P} - M_c)}{E_c I_e}$ $I_e = \frac{I_{cr}}{1 - \left(\frac{(2/3)M_{cr}}{M_{n,P}} \right)^2 \left(1 - \frac{I_{cr}}{I_g} \right)}$ $M_{cr} = f_r I_g / y_t$	$\Delta_{u,theo.} = \frac{3(2290^2) - 4(845^2)}{24} \frac{(18.69 \times 10^6 - 1.84 \times 10^6)}{24,422(22 \times 10^6)} = 16.8 \text{ mm}$ $I_e = \frac{20.4 \times 10^6}{1 - \left(\frac{(2/3)(8.17 \times 10^6)}{18.69 \times 10^6} \right)^2 \left(1 - \frac{20.4 \times 10^6}{164.8 \times 10^6} \right)} = 22 \times 10^6 \text{ mm}^4$ $M_{cr} = 3.22(164.8 \times 10^6) / 65 = 8.17 \times 10^6 \text{ Nmm} = 8.17 \text{ kNm}$

The RC slab and CFRP laminate were assumed to be in perfect bond until the load-carrying capacity was reached. The prediction of the load-carrying capacity corresponding to deflection at mid-span for the CFRP-retrofitted slab due to the applied load is shown in Table 4.

Table 4. Analysis of the retrofitted RC slab.

Calculation	Retrofitted Slab
Compressive concrete block depth $J_1 = \alpha_1 f_H b$ $J_2 = t_F b_F E_F \epsilon_c - A_s f_y$ $J_3 = -\beta_1 t_F b_F E_F \epsilon_c (t_H + t_F / 2)$ $a = \frac{-J_2 + \sqrt{J_2^2 - 4J_1 J_3}}{2J_1}$	$J_1 = 38,250 \text{ N/mm}$ $J_2 = -199,000 \text{ N}$ $J_3 = -1,131,782 \text{ Nmm}$ $a = 8.63 \text{ mm}$
Moment capacity $M_{n,P} = (d + t_H + t_F - \frac{a}{2}) A_s f_y$ $+ (t_H + \frac{t_F - a}{2}) t_F b_F E_F \epsilon_c \left(\frac{\beta_1 (t_H + t_F / 2)}{a} - 1 \right)$	$M_{n,P} = 28.87 \times 10^6 \text{ Nmm} = 28.87 \text{ kNm}$
Moment due to self-weight $M_s = w_s l^2 / 8$	$M_s = 2.27 \times 10^6 \text{ Nmm} = 2.27 \text{ kNm}$
Mid-span section's remaining capacity for carrying additional load $P_{u,theo.} = \frac{2(M_{n,P} - M_s)}{x}$	$P_{u,theo.} = 63 \text{ kN}$
Deflection at yield point steel due to the applied load $\Delta_{u,theo.} = \frac{(3l^2 - 4x^2)}{24} \frac{(M_{n,P} - M_s)}{E_c I_e}$	$\Delta_{u,theo.} = 12.3 \text{ mm}$ $I_e = 35 \times 10^6 \text{ mm}^4$ $M_{cr} = 12.5 \text{ kNm}$

3. Finite Element Modeling

3.1. Finite Element Mesh

The simulations were conducted using one-quarter of the control and strengthened slab specimens, based on specimen symmetry. The symmetric pane was simulated for the x - and z -axis by restricting translation in directions 1 and 3. The concrete slab and concrete overlay with reduced integration were constructed with eight-node solid elements (C3D8R). Two-node linear truss elements (T3D2) and four-node shell elements (S4R) were used to model the reinforcements and CFRP laminate, respectively. A mesh size of 20 mm was suggested based on preliminary mesh refinement studies, ranging from 10 mm to 25 mm, for balancing accuracy and computational cost. The embedded function in ABAQUS/CAE 2022 was implemented to simulate concrete–steel bonding. The models were studied using static analysis in ABAQUS/standard. The models under monotonic displacement loads surveyed the behavior of a slab subjected to a four-point bending test. The steel bar was discretized as a rigid part to prevent manufactured stress concentrations under the loading points, as shown in Figure 5.

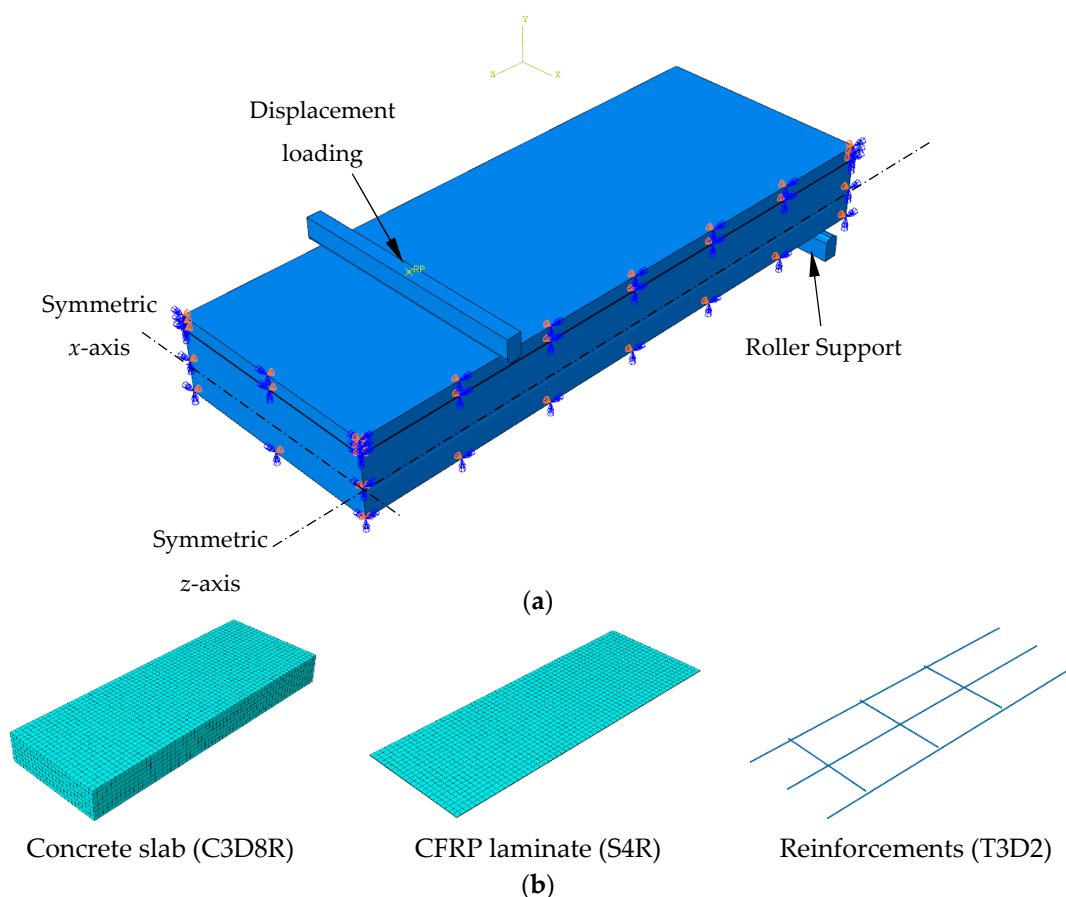


Figure 5. CFRP retrofitted slab (a) modeling and boundary conditions; (b) element types of the FEM model.

3.2. Concrete

A model of concrete plastic damage was used with a failure mechanism of compressive crushing and tensile cracking. The stress–strain curve for concrete under uniaxial compression and tension was calculated by using the CEB–FIP model [46]. Compressive strength was used to estimate the tensile strength (f_{ct}) and elastic modulus (E_c) of concrete, as recommended by ACI 318M [45].

$$f_{ct} = 0.62\sqrt{f'_c} \quad (6)$$

$$E_c = 4700 \sqrt{f'_c} \quad (7)$$

Fracture energy (G_{cr}) is an inelastic parameter associated with the softening part of the curve. In mode I, fracture energy is defined as the area under the softening curve.

$$G_{cr} = G_{f0} \left(\frac{f'_c}{10} \right)^{0.7} \quad (8)$$

Figure 6 illustrates the concrete softening curve under uniaxial tension. A linear relationship between the tension damage and crack opening (δ) is assumed to identify tensile damage. The tension damage variable ranges from zero (undamaged material) to one (total loss of strength).

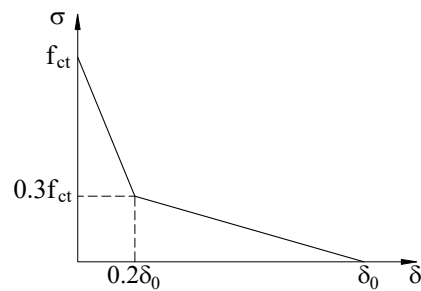


Figure 6. Concrete softening curve under uniaxial tension.

Under uniaxial compression, a linear response is observed until reaching the initial value (σ_{c0}). In the plastic regime, the response is generally characterized by stress hardening, followed by strain softening beyond the ultimate compressive stress [47].

$$\sigma_c = \frac{E_c \varepsilon_c}{1 + (R + R_E - 2)(\varepsilon_c / \varepsilon_0) - (2R - 1)(\varepsilon_c / \varepsilon_0)^2 + R(\varepsilon_c / \varepsilon_0)^3} \quad (9)$$

where $R = \left[R_E(R_\sigma - 1) / (R_\varepsilon - 1)^2 \right] - 1 / R_\varepsilon$; $R_E = E_c / E_0$; $E_0 = f'_c / \varepsilon_0$; $\varepsilon_0 = 0.0025$; $R_E = R_\sigma = 4$ [48].

3.3. Reinforced Steel

It is assumed that the reinforcement transmits force axially, and the most common perfectly linear–elastic model for reinforcement was used. Steel’s Poisson’s ratio is 0.3, and the other mechanical properties are listed in Table 2.

3.4. CFRP

For strengthening, CFRP laminate is supposed to be linear and exert isotropic behavior. CFRP failure criteria were not considered for this study because tensile stress in CFRP is too low, compared with its ultimate strength, and therefore CFRP laminate hardly ruptures for these types of structures. The material properties of CFRP, including its thickness, elastic modulus, and ultimate strength, as shown in Table 2, are provided by manufacturers.

3.5. FRP-to-Concrete Interface Model

The FRP-to-concrete interface was modeled using cohesive zones. In this study, a bilinear traction–separation model was employed to depict the bonding characteristics of linear adhesive at the interface, as shown in Figure 7 [49].

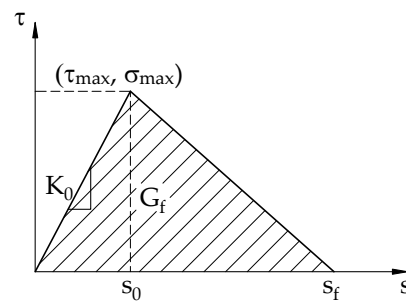


Figure 7. Bilinear traction–separation response.

The initiation and evolution of damage in the available traction–separation model were initially assumed to be a linear–elastic behavior [50,51], which can be written in a matrix form as follows:

$$\begin{Bmatrix} \tau_n \\ \tau_s \\ \tau_t \end{Bmatrix} = \begin{bmatrix} K_{nn} & 0 & 0 \\ 0 & K_{ss} & 0 \\ 0 & 0 & K_{tt} \end{bmatrix} \begin{Bmatrix} \sigma_n \\ \sigma_s \\ \sigma_t \end{Bmatrix} \quad (10)$$

For specifying the damage initiation, quadratic nominal stress and maximum nominal stress between the CFRP and concrete interface were employed. The damage criterion for quadratic stress may be depicted as follows:

$$\left(\frac{\langle \tau_n \rangle}{\sigma_{\max}} \right)^2 + \left(\frac{\tau_s}{\tau_{\max}} \right)^2 + \left(\frac{\tau_t}{\tau_{\max}} \right)^2 = 1 \quad (11)$$

The damage criterion for maximum stress can be described as follows:

$$\max \left(\frac{\langle \tau_n \rangle}{\sigma_{\max}}, \frac{\tau_s}{\tau_{\max}}, \frac{\tau_t}{\tau_{\max}} \right) = 1 \quad (12)$$

For specifying the damage evolution, the influence of power law and the Benzeggagh–Kenane (BK) fracture criteria on the structure behavior was further evaluated [52]. The power law criterion may be given by:

$$\left(\frac{G_n}{G_n^f} \right)^\eta + \left(\frac{G_s}{G_s^f} \right)^\eta + \left(\frac{G_t}{G_t^f} \right)^\eta = 1 \quad (13)$$

The BK fracture criterion may be defined as follows:

$$G_n^f + (G_s^f - G_n^f) \left(\frac{G_s + G_t}{G_n + G_s} \right)^\eta = G_f \quad (14)$$

In this study, the quantities G_n^f , G_s^f , and G_t^f were assumed to be equal. For FE modeling, first, a perfect FRP-to-concrete bond was assumed to reflect the response of the flexural strengthened RC slab using the ABAQUS program. Then, the interaction behavior of the FRP-to-concrete bond in the strengthened slab applied to the retrofit system was evaluated using the existing constitutive models, as shown in Table 5 [31–36].

Table 5. Existing bond–slip models.

Bond–Slip Model	
Neubauer and Rostasy [31]	$\tau = \begin{cases} \tau_{\max} \left(\frac{s}{s_0} \right) & \text{if } s \leq s_0 \\ 0 & \text{if } s > s_0 \end{cases}, \tau_{\max} = 1.8\beta_w f_{ct}, s_0 = 0.202, \beta_w = \sqrt{1.125 \frac{2-b_f/b}{1+b_f/400}}$
Nakaba et al. [32]	$\tau = \tau_{\max} \frac{s}{s_0} \frac{3}{2+(s/s_0)^3}, \tau_{\max} = 3.5f_c^{0.19}, s_0 = 0.065$
Monti et al. [33]	$\tau = \begin{cases} \tau_{\max} \left(\frac{s}{s_0} \right) & \text{if } s \leq s_0 \\ \tau_{\max} \left(\frac{s_f - s}{s_f - s_0} \right) & \text{if } s > s_0 \end{cases}, \tau_{\max} = 1.8\beta_w f_{ct}, s_0 = 2.5\tau_{\max} \left(\frac{t_a}{E_a} + \frac{50}{E_c} \right),$ $s_f = 0.33\beta_w, \beta_w = \sqrt{1.125 \frac{2-b_f/b}{1+b_f/400}}$
Dai and Ueda [34]	$\tau = \begin{cases} \tau_{\max} \left(\frac{s}{s_0} \right)^{0.575} & \text{if } s \leq s_0 \\ \tau_{\max} e^{-\beta(s-s_0)} & \text{if } s > s_0 \end{cases}, \tau_{\max} = \frac{-1.575\alpha K_a + \sqrt{2.481\alpha^2 K_a^2 + 6.3\alpha\beta_w^2 K_a G_f}}{2\beta}, s_0 = \frac{\tau_{\max}}{\alpha K_a},$ $G_f = 7.554K_a^{-0.449} (f'_c)^{0.343}, \beta_w = 0.0035K_a (E_{ftf}/1000)^{0.34},$ $K_a = G_a/t_a, \alpha = 0.028 (E_{ftf}/1000)^{0.254}$
Lu et al. [35]	$\tau = \begin{cases} \tau_{\max} \sqrt{\frac{s}{s_0}} & \text{if } s \leq s_0 \\ \tau_{\max} e^{-\alpha(\frac{s}{s_0}-1)} & \text{if } s > s_0 \end{cases}, \alpha = \left(\frac{G_f}{\tau_{\max} s_0} - \frac{2}{3} \right)^{-1}, \tau_{\max} = 1.5\beta_w f_{ct},$ $G_f = 0.308\beta_w^2 \sqrt{f_{ct}}, \beta_w = \sqrt{\frac{2.25-b_f/b}{1.25+b_f/b}}$
Obaidat et al. [36]	$K_0 = 0.16 \frac{G_a}{t_a}, \tau_{\max} = 1.46G_a^{0.165} f_{ct}^{1.033}, G_f = 0.52f_{ct}^{0.26} G_a^{-0.23}$

It is easily conceptualized that the traction–separation law, as described in Figure 7, is primarily governed by three parameters, namely initial stiffness (K_0), maximum shear stress ($\tau_{\max}$), and fracture energy (G_f) [53]. Based on the mechanical properties of the experimental materials, the bond–slip models evaluated in this study were introduced with the three characteristic parameters of the traction–separation law, as shown in Table 6.

Table 6. Calculation of characteristic parameters of the traction–separation law.

Constitutive Models	K_0 (MPa/mm)	$\tau_{\max}$ (MPa)	G_f (N/mm)
Neubauer and Rostasy [31]	28.69	3.41	0.20
Nakaba et al. [32]	100.73	6.55	0.94
Monti et al. [33]	157.03	2.24	0.14
Dai and Ueda [34]	83.90	7.10	0.90
Lu et al. [35]	76.92	3.60	0.31
Obaidat et al. [36]	224.47	17.26	0.12

3.6. Analysis Procedure and Flowchart

A three-dimensional FE model using ABAQUS/CAE 2022 software was performed to simulate the behavior of retrofitted slabs. In the initial phase, an accurate FE model of the reference slab was assessed by comparison with the experimental results. In the second phase, a retrofitted slab model was developed based on the reference slab model, with the addition of FRP laminate and concrete overlay. The well-known bond–slip models were used to simulate the FRP-to-concrete interface to accurately simulate the behavior of the retrofitted slab validated by experiment results. If the available bond–slip models were not appropriate, the bond–slip parameters were revised to show good agreement with the experimental data. A flowchart of the analysis procedure for the FE model is shown in Figure 8.

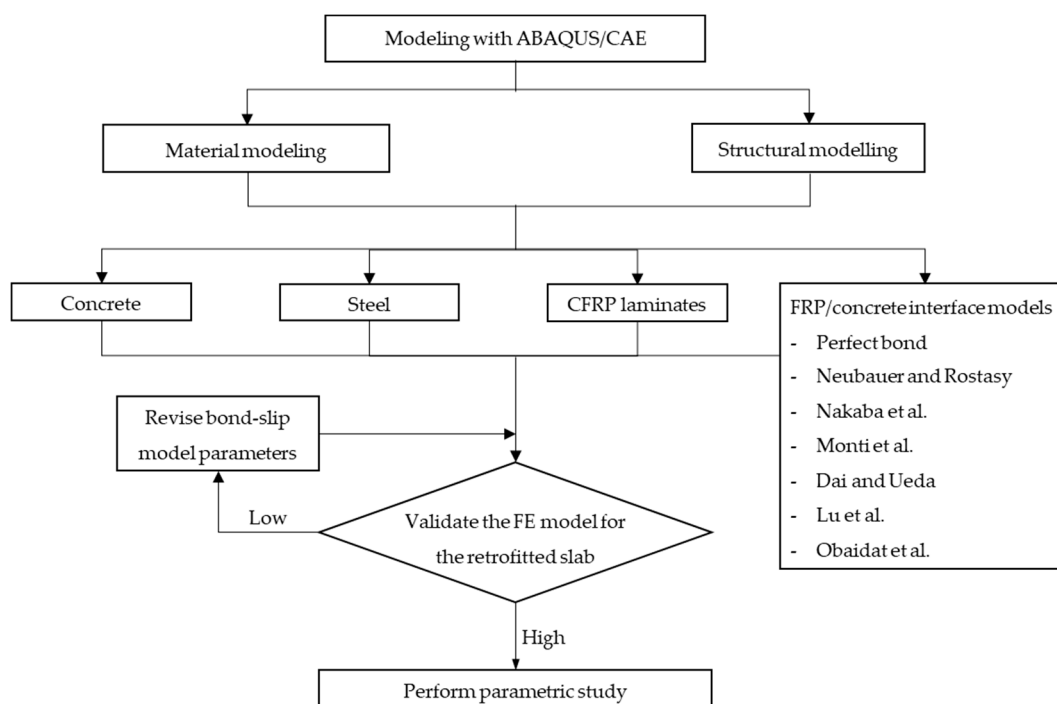


Figure 8. FE model analysis flowchart.

4. Results and Discussion

4.1. Experimental Analysis

The experimental data from the reference slab showed a good agreement with those of the theoretical analysis. The RC slab behaved linearly and elastically until the first crack appeared at 17 kN of the applied load. The observed result revealed a higher load-carrying capacity of 43.5 kN than the theoretical prediction of 39.9 kN. The ultimate loading (50.8 kN) was 27.3% greater than the predicted load. The linearly elastic–perfectly plastic behavior of the steel can be the reason for the lower strength of the prediction because strain-hardened steel shows higher tensile strength in practice. The mid-span deflection reached a maximum value of 72 mm, as shown in Figure 9. The slope of the load–deflection curve within the elastic limits depicts the stiffness (K) of the slab. The stiffness of the RC slab was determined as 10.1 kN/mm. Calculating the toughness of a structure can be accomplished by integrating the load–deflection curve. Accordingly, the reference slab had a toughness of 3129.2 kNmm.

For the retrofit slab, the cracks emerged and expanded most strongly at the mid-span zone—between the loading points, as shown in Figure 10. The retrofitted slab showed a load-carrying capacity of 69.2 kN at 14 mm mid-span deflection. This was 9% higher than the predicted load of 63 kN. The structure lost carrying capacity due to the micro-buckling of CFRP laminate at the applied load of 32 kN. Then, the applied load continued to increase until CFRP laminate was delaminated at 76.9 kN, corresponding to 22 mm of the mid-span deflection. After delamination, the global response of the concrete slab was equivalent to that of the reference concrete slab. Its failure load and mid-span deflection were 49.2 kN and 81 mm, respectively, as shown in Figure 9. It is possible to define the stiffness of the retrofitted slab at 21.3 kN/mm, which was 2.11 times that of the reference slab. The retrofitted slab had a toughness of 4034.1 kNmm, 1.29 times greater than the reference slab. Compared with theoretical analysis, Table 7 summarizes the experimental results for load-carrying capacity corresponding to the mid-span deflection of the slabs.

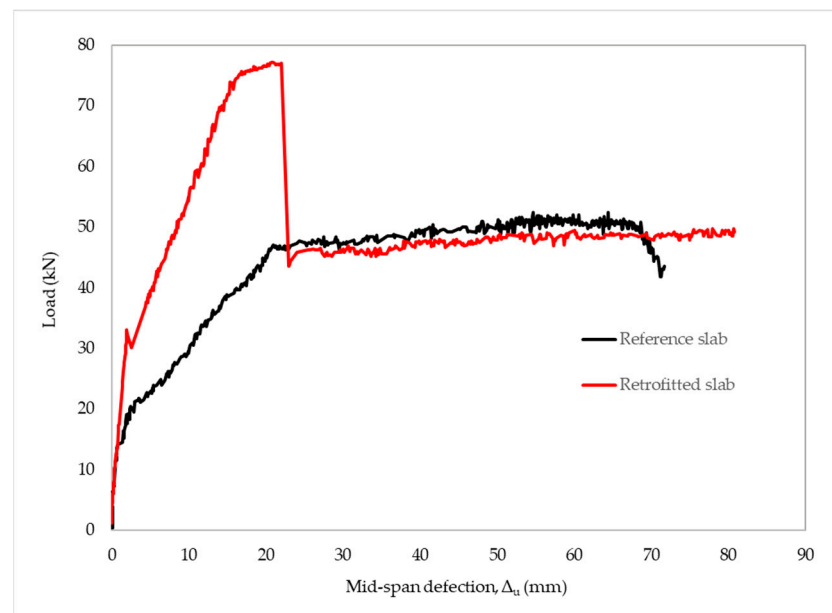


Figure 9. Load–deflection relationship for reference and retrofitted RC slabs.



Figure 10. The cracks at the mid-span of the slab: (a) front view; (b) bottom view.

Table 7. Predictions of slabs' loading-carrying capacity and corresponding mid-span deflection compared with experimental results.

Slab	Experiment		Prediction		$\frac{P_{u,theo.}}{P_{u,exp.}}$	$\frac{u_{theo.}}{u_{exp.}}$
	$P_{u,exp.}$ (kN)	$\Delta_{u,exp.}$ (mm)	$P_{u,theo.}$ (kN)	$\Delta_{u,theo.}$ (mm)		
Reference slab	43.5	18.5	39.9	16.8	0.92	0.91
Retrofitted slab	69.2	14.0	63.0	12.3	0.91	0.88

4.2. Numerical Analysis

In this part of the study, the FE model of the retrofitted slab was used to examine the failure mode and delamination load after obtaining a good agreement between the FE model of the reference slab and the experimental results using the commercial ABAQUS program. A perfect bond model and well-known bond–slip models were applied to simulate the slab behavior through the load–deflection relationship. The material properties of the experimental structure were used to calculate the input parameters for the bond–slip models, as shown in Table 6. The FE models were employed to evaluate the suitability of bond–slip laws based on the observed results. Most existing bond–slip models can predict the load-carrying capacity of the retrofitted slab when errors in the prediction of FE models are less than 7%. Nevertheless, the bond–slip models could not accurately predict the mid-span deflection corresponding to the load-carrying capacity and stiffness, with errors higher than 17% and 32%, respectively. The behavior of the well-known bond–slip models also showed a perfect bond between CFRP and concrete, even though delamination failure was observed in the experiment, as shown in Figure 11. These discrepancies may have resulted from

the improper estimations of the bond–slip parameters involving fracture energy, the initial stiffness of the interface elements, maximum interfacial stress, damage initiation criteria, and mixed-mode failure criteria. The FEM results of the load-carrying capacity corresponding to the mid-span deflection and stiffness within the elastic limits of the strengthened slab, compared with its experimental counterpart, are summarized in Table 8.

Table 8. FEM predicted results for retrofitted slab compared with experimental data.

Constitutive Models	$P_{u,mod.}$ (kN)	$\Delta_{u,mod.}$ (mm)	$K_{mod.}$ (kN/mm)	$\frac{P_{u,mod.}}{P_{u,exp.}}$	$\frac{\Delta_{u,mod.}}{\Delta_{u,exp.}}$	$\frac{K_{mod.}}{K_{exp.}}$
Perfect bond	73.7	11.53	29.7	1.07	0.82	1.39
Neubauer and Rostasy [31]	69.8	10.83	28.1	1.01	0.77	1.32
Nakaba et al. [32]	73.6	11.57	29.2	1.06	0.83	1.37
Monti et al. [33]	72.2	10.77	29.4	1.04	0.77	1.38
Dai and Ueda [34]	73.6	11.58	29.1	1.06	0.83	1.37
Lu et al. [35]	70.4	10.81	29.0	1.02	0.77	1.36
Obaidat et al. [36]	72.6	10.71	29.5	1.05	0.77	1.38

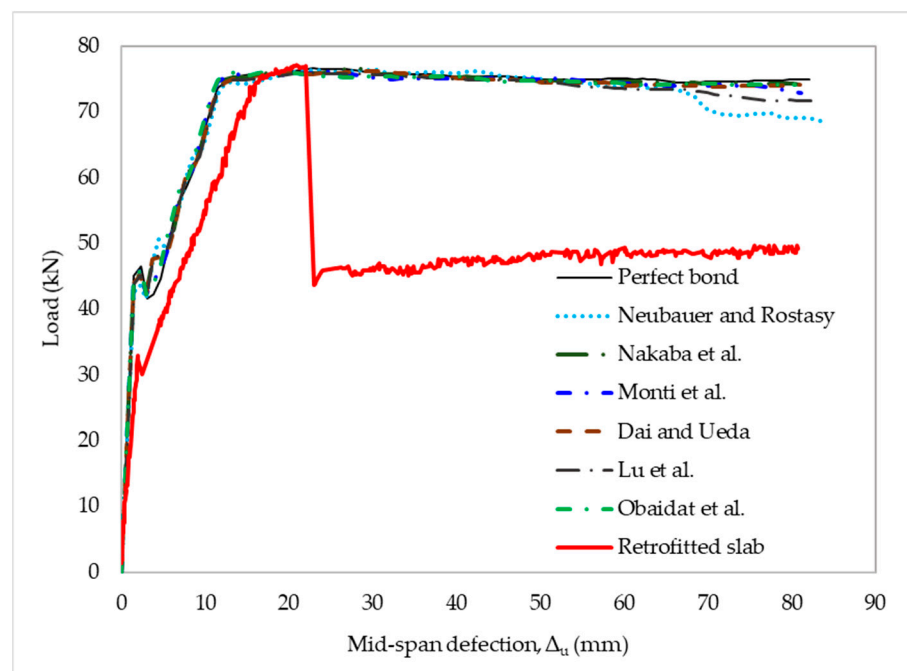


Figure 11. Experimental and FEM models of load–deflection curves for retrofitted slabs.

4.3. Bond–Slip Analysis

The existing bond–slip models tended to overestimate the bond strength of the CFRP-to-concrete interface. In this part of the study, a sensitivity analysis of the bond–slip input parameters was performed to compare its results with the structural behavior obtained from the experimental data. The bond–slip model of Neubauer and Rostasy was used to illustrate the numerical results from the strengthened slabs. Initially, the CFRP and concrete bonding interface was assumed to fail using quadratic traction. A cohesive property coefficient of $\eta = 1$ was applied to depict the influence of the opening and sliding failure modes on fracture energy. The authors investigated the sensitivity of the results in the interfacial fracture energy between CFRP and concrete. For this case, a change was made from $0.01G_f$ to G_f for the interfacial fracture energy. There was no evidence that G_f had a notable effect on the numerical results, as shown in Figure 12.

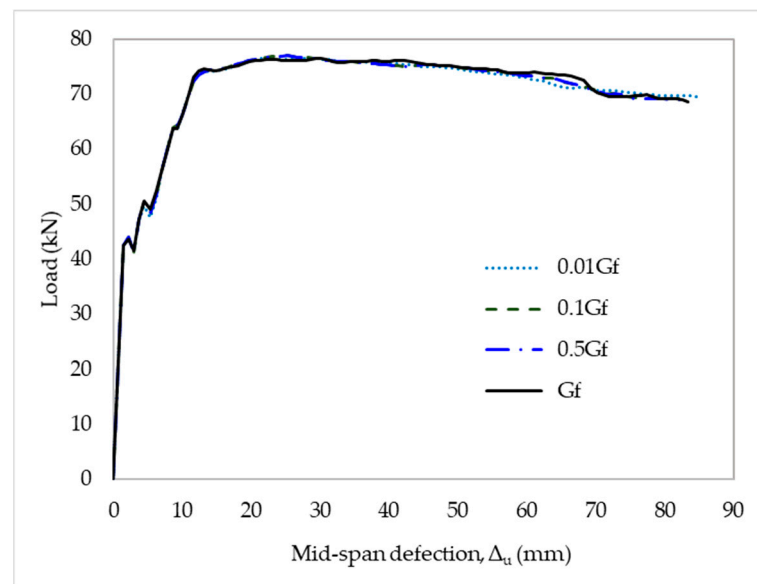


Figure 12. Sensitivity to fracture energy of CFRP-to-concrete interface.

The initial stiffness of the interface elements was changed from $0.01K_0$ to K_0 to evaluate its effect. It had a substantial influence on the load–deflection curve’s slope of the structure. Based on the simulated results, the stiffness of the retrofitted slab decreased with decreasing the initial stiffness but was not proportional. With the initial stiffness of $0.1K_0$, the global response of the slab within the elastic limits could match the experimental results, as shown in Figure 13. However, the constitutive model exhibited a perfect bond of CFRP-to-concrete interface.

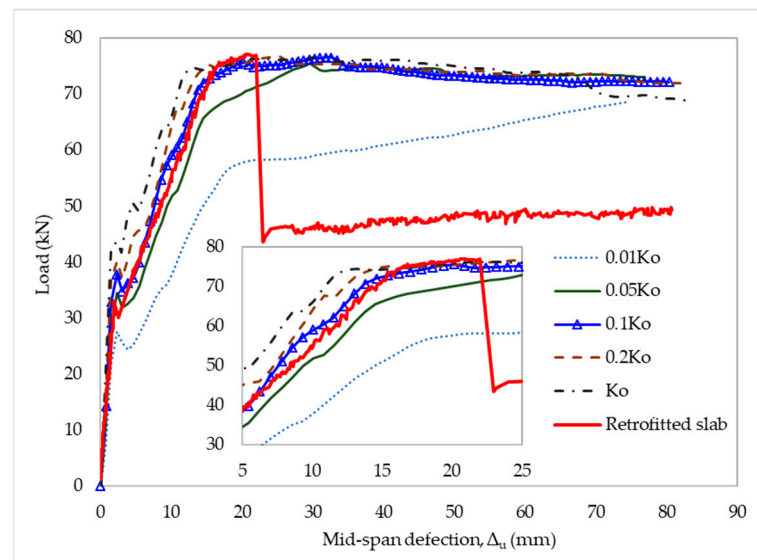


Figure 13. Sensitivity to initial stiffness of interface elements.

As mentioned above, the low bond strength between the concrete overlay and CFRP was the main reason for the delamination failure of the retrofit slab. It was possible to obtain better explanations for the differences between the simulated and experimental results by modifying the maximum shear stress. Therefore, the maximum interfacial stress of the concrete overlay/CFRP was further investigated using sensitivity analysis. An appropriate value for this parameter was crucial for the accurate prediction of CFRP debonding failures. As shown in Figure 14a, the maximum shear stress was increased from $0.1\tau_{\max}$ to $\tau_{\max}$. The shear stress values had a considerable impact on the debonding failure. The delamination load of the retrofitted slab was sensitive to changes in the ranges of maximum shear stress

from $0.1\tau_{\max}$ to $0.15\tau_{\max}$, whereas a perfect bond was shown for values greater than or equal to $0.2\tau_{\max}$. It could also be easily seen that Neubauer's and Rostasy's bond-slip law, along with other equivalent models, overestimated the ultimate shear strength of the concrete overlay–CFRP interface.

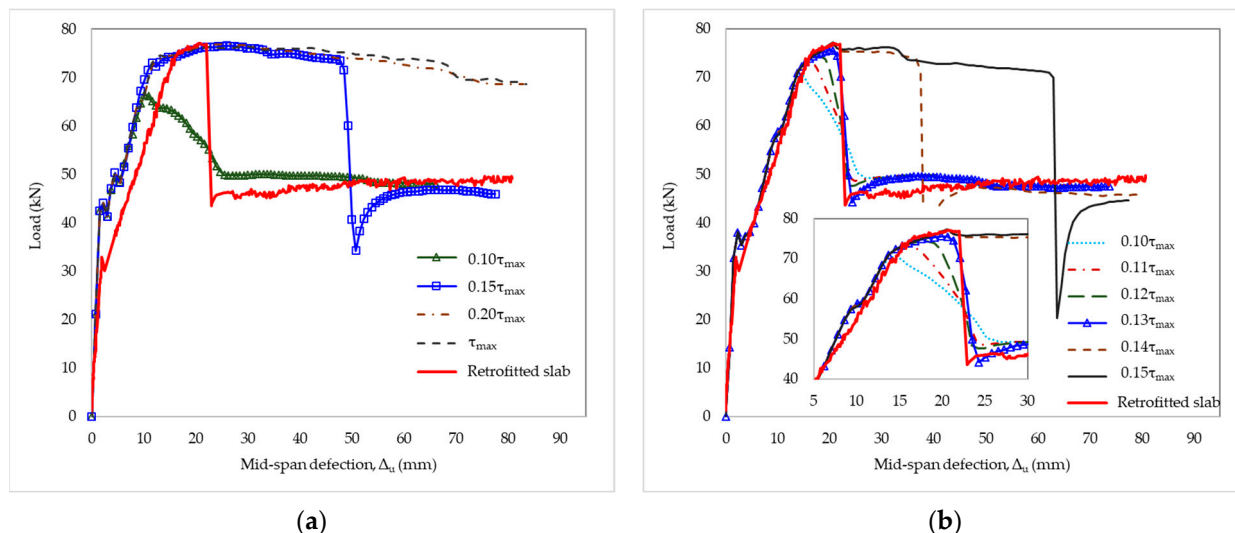


Figure 14. Sensitive to the maximum shear stress (a) initial stiffness of K_0 ; (b) initial stiffness of $0.1K_0$.

According to the simulations, the delamination loads were 66.2 kN and 74 kN, corresponding to 10.1 mm and 48 mm of the mid-span deflection, respectively, occurring with the maximum shear stresses of $0.1\tau_{\max}$ and $0.15\tau_{\max}$, respectively. The numerical results also indicated that the debonding failure emerged earlier, at $0.1\tau_{\max}$ and later at $0.15\tau_{\max}$, compared with the experimental results. Moreover, the overestimation of the initial stiffness should also be highlighted in this case. The maximum shear stress ranged from $0.1\tau_{\max}$ to $0.15\tau_{\max}$, combined with the initial stiffness of $0.1K_0$; thus, the case was further evaluated. The delamination load and mid-span deflection of the retrofitted slab increased with the increase in the maximum shear stress but was not proportional; however, there were no notable differences in the delamination load after the yield in steel. The global response of the strengthened slab was displayed as a function of the shear stress values. As a result, it was possible to reasonably simulate the behavior of the retrofitted slab and compare it with its experimental counterpart with the initial stiffness of $0.1K_0$ and maximum shear stress of $0.13\tau_{\max}$, as shown in Figure 14b. In this part of the study, we analyzed the damage initiation criteria as the quadratic and maximum nominal stresses based on the proposed results mentioned above. The quadratic stress criterion is defined by accounting for the effect of traction components, while the maximum stress criterion is established by comparing the traction components to their respective allowable values [54]. Despite having similar responses until the yield in steel, the quadratic and maximum nominal stress criteria exhibited different mid-span deflections at debonding failures of 22 mm and 28 mm, respectively, as shown in Figure 15. Poor results were obtained using the maximum stress criterion for predicting delamination in the retrofitted slabs. There was a significant interaction between stresses for decohesion elements dealing with mixed-mode delamination onset and propagation, as mentioned by Cui et al. [55]. Debonding behavior can emerge before any of the involved traction components reach their respective allowable limits. An accurate prediction of delamination failure requires taking into account the interaction. Thus, the damage initiation criterion applying the quadratic nominal stress was suggested in this case.

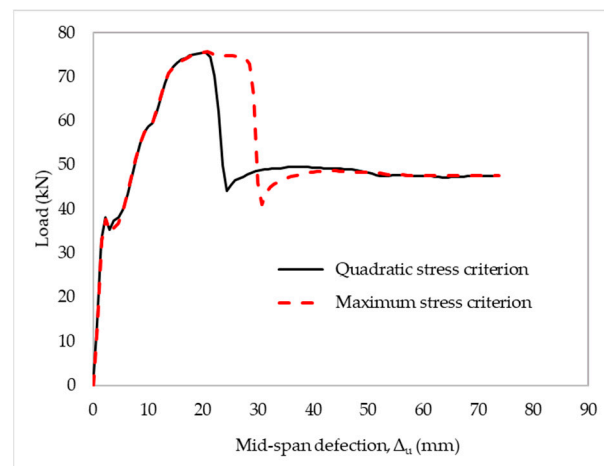


Figure 15. The effect of strengthened slabs on damage initiation criteria.

Here, we describe the effects of damage evolution based on energy. The load–deflection relationship was derived from the power law or BK law, as shown in Figure 16a. Likewise, a study of the cohesive coefficient sensitivity of FE results was also conducted, the result of which is shown in Figure 16b. Nevertheless, these criteria had no effect on the delamination load or global response of the strengthened RC slabs.

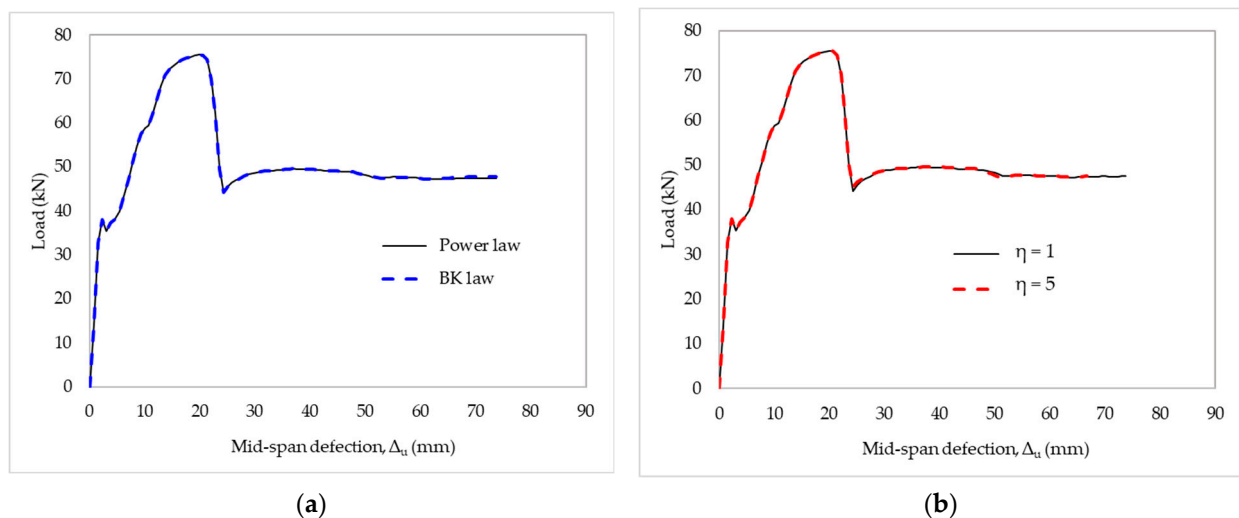


Figure 16. The effects of strengthened slabs on damage evolution: the criteria of (a) mixed-mode failure and (b) cohesive parameter (η).

In brief, these results were consistent with the previous literature regarding the influential factors on debonding failure, which is significantly affected by the maximum interfacial stress [40,56,57] but insensitive to changes in fracture energy and mixed-mode failure criteria [53]. However, disagreement can also be observed once the global response of the strengthened slab with a hybrid retrofit system was found to be impacted by changing the criteria of the stiffness of interface elements and damage initiation. Developing an appropriate bond–slip model will require further investigation to determine the precise values for notable influential factors on debonding failure.

5. Conclusions

In this paper, a hybrid retrofit system considering the difficulty of accessing and installing CFRP laminates to the underside of RC slabs to enhance the flexural carrying capacity, stiffness, and toughness of the existing RC slabs was proposed. The proposed retrofitting mechanism proved suitable through a good agreement between the experimental results and theoretical analyses at the mid-span section. The global response and the effect of the bond–slip law in predicting the debonding failure of the retrofitted RC slabs were presented. The FE model using a sensitivity analysis based on the load–deflection relationship was performed to evaluate the parameters influencing the interfacial behavior between CFRP and concrete. Based on the obtained results, the following conclusions could be drawn as follows:

The efficiency of the retrofit system was verified through the experimental test of the strengthened slab in the positive moment section. The retrofit system enhanced the load-carrying capacity of the slab by 59%, stiffness by 111%, and toughness by 29%.

The neutral axis of the retrofitted slab was located within the overlay, and CFRP held tension at the mid-span section at the ultimate failure state.

The bond–slip models overestimated the criteria regarding the damage initiation between CFRP and the concrete overlay. The numerical analysis results with the initial stiffness of $0.1K_0$ and maximum shear stress of $0.13\tau_{\max}$ were compared with the corresponding values of Neubauer's and Rostasy's bond–slip law and showed a good agreement with the experimental data.

The stiffness of structures is notably impacted by the initial stiffness of the interface elements, while the delamination load and failure load can be decided by the maximum interfacial stress. Quadratic nominal stress is recommended as a criterion for damage initiation based on the considerable interaction between stresses.

The global response of the retrofitted slab was not sensitive to changes in the interfacial fracture energy and damage evolution regarding the mix-mode failure criteria and the cohesive coefficient of the interface.

Our study contributes to the evaluation of the impact of bond–slip parameters on the behavior of retrofitted slabs. Nonetheless, further research into an appropriate bond–slip model for retrofitted RC structures should consider the influence of factors regarding materials properties and geometric dimensions.

Author Contributions: Conceptualization, J.J.K.; methodology, J.J.K. and D.H.L.; software, H.Q.N.; validation, J.J.K. and T.N.M.N.; formal analysis, H.Q.N. and T.N.M.N.; investigation, J.J.K. and D.H.L.; data curation, T.N.M.N. and H.Q.N.; writing—original draft preparation, H.Q.N.; writing—review and editing, H.Q.N. and J.J.K.; visualization, J.J.K.; supervision, J.J.K. and D.H.L.; project administration, J.J.K.; funding acquisition, J.J.K. All authors have read and agreed to the published version of the manuscript.

Funding: This research was supported by a grant (21CTAP-C163626-01) from the Technology Advancement Research Program (TARP) funded by the Ministry of Land, Infrastructure, and Transport of the Korean government.

Institutional Review Board Statement: Not applicable.

Informed Consent Statement: Not applicable.

Data Availability Statement: Not applicable.

Conflicts of Interest: The authors declare no conflict of interest.

Nomenclature

A_s	Tensile steel area	$P_{u,theo.}$	Prediction of carrying additional load capacity
b	Width of RC slab	K	Stiffness of slab
b_F	Width of CFRP laminate	s	Local slip
c	Distance from extreme compression fiber to the neutral axis	s_f	Maximum local slip
d	Distance from the extreme fiber of the compression zone to the center of the steel	s_0	Local slip at τ_{max}
E_a	Elastic modulus of epoxy resin	t_a	The thickness of the epoxy resin
E_c	Elastic modulus of concrete	t_F	The thickness of CFRP laminate
E_F	Elastic modulus of CFRP	t_H	The thickness of the concrete overlay
f'_c	Compressive strength of concrete	j_1d	Moment arm length from the center of concrete stress block to a steel
f_{ct}	Tensile strength of concrete	j_2d	Moment arm length from the center of concrete stress block to CFRP
f'_H	Compressive strength of the concrete overlay	Δ_u	Mid-span deflection
f_{tF}	Tensile strength of CFRP	$\Delta_{u,exp.}$	Mid-span deflection corresponds to experimental load-carrying capacity
G_a	Shear modulus of epoxy resin	$\Delta_{u,mod.}$	Mid-span deflection corresponds to modeling load-carrying capacity
G_c	Shear modulus of concrete	$\Delta_{u,theo.}$	Mid-span deflection corresponds to the predicted loading-carrying capacity
G_f	Fracture energy	τ	Shear stress
G_n, G_s, G_t	Fracture energies in the normal, first, and second directions of shear, respectively	τ_{max}	Maximum shear stress
G_n^f, G_s^f, G_t^f	Fracture energies to induce failure in the normal, first, and second directions of shear, respectively	τ_n, τ_s, τ_t	The normal, first, and second shear stress, respectively
h	Height of RC slab	σ_{max}	Maximum nominal stress
K_a	Shear stiffness of the epoxy resin	η	Parameter for cohesive material
K_c	Shear stiffness of concrete	α	A parameter of bond–slip relationships
K_0	Initial stiffness	β_w	Width ratio factor
K_{nn}, K_{ss}, K_{tt}	Elastic stiffnesses in the normal, first, and second directions of shear, respectively	γ_c	Unit weight of concrete
$P_{u,exp.}$	Capacity for carrying additional loads in the experiment	$\langle \rangle$	Macaulay brackets, which implied that compressive stress does not lead to the initial damage
$P_{u,mod.}$	Capacity for carrying additional loads in the numerical modeling		

References

- Hariri-Ardebili, M.A.; Sanchez, L.; Rezakhani, R. Aging of Concrete Structures and Infrastructures: Causes, Consequences, and Cures (C3). *Adv. Mater. Sci. Eng.* **2020**, *2020*, 9370591. [\[CrossRef\]](#)
- Martínez, S.; De Diego, A.; Castro, V.; Echevarría, L.; Barroso, F.; Rentero, G.; Soldado, R.; Gutiérrez, J. Strengthening of Low-Strength Concrete Columns with Fibre Reinforced Polymers. Full-Scale Tests. *Infrastructures* **2020**, *5*, 91. [\[CrossRef\]](#)
- Nguyen, H.Q.; Nguyen, T.N.M.; Lee, D.H.; Kim, J.J. A Design Method to Induce Ductile Failure of Flexural Strengthened One-Way RC Slabs. *Materials* **2021**, *14*, 7647. [\[CrossRef\]](#)
- Ho, H.V.; Choi, E.; Park, S.J. Investigating stress distribution of crimped SMA fibers during pullout behavior using experimental testing and a finite element model. *Compos. Struct.* **2021**, *272*, 114254. [\[CrossRef\]](#)
- Bai, Y.; Nardi, D.C.; Zhou, X.; Picón, R.A.; Flórez-López, J. A new comprehensive model of damage for flexural subassemblies prone to fatigue. *Comput. Struct.* **2021**, *256*, 106639. [\[CrossRef\]](#)
- Amran, Y.H.M.; Alyousef, R.; Rashid, R.S.M.; Alabduljabbar, H.; Hung, C.-C. Properties and applications of FRP in strengthening RC structures: A review. *Structures* **2018**, *16*, 208–238. [\[CrossRef\]](#)
- Haji, M.; Naderpour, H.; Kheyroddin, A. Experimental study on influence of proposed FRP-strengthening techniques on RC circular short columns considering different types of damage index. *Compos. Struct.* **2019**, *209*, 112–128. [\[CrossRef\]](#)
- Barris, C.; Sala, P.; Gómez, J.; Torres, L. Flexural behaviour of FRP reinforced concrete beams strengthened with NSM CFRP strips. *Compos. Struct.* **2020**, *241*, 112059. [\[CrossRef\]](#)

9. Basaran, B.; Kalkan, I. Development length and bond strength equations for FRP bars embedded in concrete. *Compos. Struct.* **2020**, *251*, 112662. [\[CrossRef\]](#)
10. Mukhtar, F.M.; Arowojolu, O. Recent developments in experimental and computational studies of hygrothermal effects on the bond between FRP and concrete. *J. Reinf. Plast. Compos.* **2020**, *39*, 422–442. [\[CrossRef\]](#)
11. Nguyen, T.N.M.; Moon, J.; Kim, J.J. Microstructure and mechanical properties of hardened cement paste including Nylon 66 nanofibers. *Constr. Build. Mater.* **2020**, *232*, 117134. [\[CrossRef\]](#)
12. Wang, X.; Yang, Y.; Yang, R.; Liu, P. Experimental Analysis of Bearing Capacity of Basalt Fiber Reinforced Concrete Short Columns under Axial Compression. *Coatings* **2022**, *12*, 654. [\[CrossRef\]](#)
13. Chaliouris, C.E.; Kosmidou, P.-M.K.; Papadopoulos, N.A. Investigation of a New Strengthening Technique for RC Deep Beams Using Carbon FRP Ropes as Transverse Reinforcements. *Fibers* **2018**, *6*, 52. [\[CrossRef\]](#)
14. Hosen, M.A.; Jumaat, M.Z.; Alengaram, U.J.; Sulong, N.H.R.; Islam, A.B.M.S. Structural performance of lightweight concrete beams strengthened with side-externally bonded reinforcement (S-EBR) technique using CFRP fabrics. *Compos. Part B Eng.* **2019**, *176*, 107323. [\[CrossRef\]](#)
15. Moshiri, N.; Czaderski, C.; Mostofinejad, D.; Hosseini, A.; Sanginabadi, K.; Breveglieri, M.; Motavalli, M. Flexural strengthening of RC slabs with nonprestressed and prestressed CFRP strips using EBROG method. *Compos. Part B Eng.* **2020**, *201*, 108359. [\[CrossRef\]](#)
16. Triantafillou, T.; Matthys, S.; Audenaert, K.; Balázs, G.; Blaschko, M.; Blontrock, H.; Czaderski, C.; David, E.; di Tomasso, A.; Duckett, W.; et al. *Externally Bonded FRP Reinforcement for RC Structures*; International Federation for Structural Concrete: Lausanne, Switzerland, 2001; p. 138.
17. Salman, W.D.; Mansor, A.A.; Mahmood, M. Behavior of Reinforced Concrete One-Way Slabs Strengthened by CFRP sheets in Flexural Zone. *Int. J. Civ. Eng. Technol.* **2018**, *9*, 10.
18. Torabian, A.; Isufi, B.; Mostofinejad, D.; Ramos, A.P. Flexural strengthening of flat slabs with FRP composites using EBR and EBROG methods. *Eng. Struct.* **2020**, *211*, 110483. [\[CrossRef\]](#)
19. Triantafillou, T. Strengthening of existing concrete structures: Concepts and structural behavior. In *Textile Fibre Composites in Civil Engineering*; Elsevier: Amsterdam, The Netherlands, 2016; pp. 303–322.
20. Monsef, S.A.; Ortega, A.; Turon, A.; Maimí, P.; Renart, J. An efficient method to extract a mode I cohesive law for bonded joints using the double cantilever beam test. *Compos. Part B Eng.* **2019**, *178*, 107424. [\[CrossRef\]](#)
21. Naser, M.Z.; Hawileh, R.A.; Abdalla, J. Modeling Strategies of Finite Element Simulation of Reinforced Concrete Beams Strengthened with FRP: A Review. *J. Compos. Sci.* **2021**, *5*, 19. [\[CrossRef\]](#)
22. Ghorbani, M.; Mostofinejad, D.; Hosseini, A. Experimental investigation into bond behavior of FRP-to-concrete under mixed-mode I/II loading. *Constr. Build. Mater.* **2017**, *132*, 303–312. [\[CrossRef\]](#)
23. Vu, H.D.; Phan, D.N. A framework for predicting the debonding failure modes of RC beams strengthened flexurally with FRP sheets. *Innov. Infrastruct. Solutions* **2022**, *7*, 292. [\[CrossRef\]](#)
24. Pešić, N.; Pilakoutas, K. Concrete beams with externally bonded flexural FRP-reinforcement: Analytical investigation of debonding failure. *Compos. Part B Eng.* **2003**, *34*, 327–338. [\[CrossRef\]](#)
25. Azzawi, R.; Varughese, N. Flexural behavior of preflex ssrc-encased steel joist composite beams. *Results Eng.* **2020**, *7*, 100122. [\[CrossRef\]](#)
26. Kotynia, R.; Baky, H.A.; Neale, K.W.; Ebead, U.A. Flexural Strengthening of RC Beams with Externally Bonded CFRP Systems: Test Results and 3D Nonlinear FE Analysis. *J. Compos. Constr.* **2008**, *12*, 190–201. [\[CrossRef\]](#)
27. Kim, J.J.; Noh, H.-C.; Taha, M.M.R.; Mosallam, A. Design limits for RC slabs strengthened with hybrid FRP-HPC retrofit system. *Compos. Part B Eng.* **2013**, *51*, 19–27. [\[CrossRef\]](#)
28. Moon, J.; Taha, M.M.R.; Kim, J.J. Flexural Strengthening of RC Slabs Using a Hybrid FRP-UHPC System Including Shear Connector. *Adv. Mater. Sci. Eng.* **2017**, *2017*, 4387545. [\[CrossRef\]](#)
29. Mosallam, A.; Taha, M.M.R.; Kim, J.J.; Nasr, A. Strength and ductility of RC slabs strengthened with hybrid high-performance composite retrofit system. *Eng. Struct.* **2012**, *36*, 70–80. [\[CrossRef\]](#)
30. Yuan, H.; Teng, J.; Seracino, R.; Wu, Z.; Yao, J. Full-range behavior of FRP-to-concrete bonded joints. *Eng. Struct.* **2004**, *26*, 553–565. [\[CrossRef\]](#)
31. Neubauer, U.; Rostasy, F.S. Bond Failure of Concrete Fiber Reinforced Polymer Plates at Inclined Cracks—Experiments and Fracture Mechanics Model. *ACI Symp. Publ.* **1999**, *188*, 369–382.
32. Nakaba, K.; Kanakubo, T.; Furuta, T.; Yoshizawa, H. Bond Behavior between Fiber-Reinforced Polymer Laminates and Concrete. *ACI Struct. J.* **2001**, *98*, 359–367.
33. Monti, G.; Renzelli, M.; Luciani, P. FRP adhesion in uncracked and cracked concrete zones. In *Fibre-Reinforced Polymer Reinforcement for Concrete Structures*; World Scientific Publishing Company: Singapore, 2003; pp. 183–192.
34. Dai, J.G.; Ueda, T. Local bond stress slip relations for FRP sheets-concrete interfaces. In *Fibre-Reinforced Polymer Reinforcement for Concrete Structures*; World Scientific Publishing Company: Singapore, 2003; pp. 143–152.
35. Lu, X.Z.; Teng, J.G.; Ye, L.P.; Jiang, J.J. Bond-slip models for FRP sheets/plates bonded to concrete. *Eng. Struct.* **2005**, *27*, 920–937. [\[CrossRef\]](#)
36. Obaidat, Y.T.; Heyden, S.; Dahlblom, O. Evaluation of Parameters of Bond Action between FRP and Concrete. *J. Compos. Constr.* **2013**, *17*, 626–635. [\[CrossRef\]](#)

37. Wu, Y.-F.; Jiang, C. Quantification of Bond-Slip Relationship for Externally Bonded FRP-to-Concrete Joints. *J. Compos. Constr.* **2013**, *17*, 673–686. [\[CrossRef\]](#)
38. Bocciarelli, M.; Pisani, M.A. Modified force method for the nonlinear analysis of FRP reinforced concrete beams. *Compos. Struct.* **2015**, *131*, 645–653. [\[CrossRef\]](#)
39. Tounsi, A.; Daouadji, T.H.; Benyoucef, S. Interfacial stresses in FRP-plated RC beams: Effect of adherend shear deformations. *Int. J. Adhes. Adhes.* **2009**, *29*, 343–351. [\[CrossRef\]](#)
40. He, X.J.; Zhou, C.Y.; Wang, Y. Interfacial stresses in reinforced concrete cantilever members strengthened with fibre-reinforced polymer laminates. *Adv. Struct. Eng.* **2020**, *23*, 277–288. [\[CrossRef\]](#)
41. Mostafa, A.A.; Razaqpur, A. Finite element model for predicting post delamination behaviour in FRP-retrofitted beams in flexure. *Constr. Build. Mater.* **2017**, *131*, 195–204. [\[CrossRef\]](#)
42. Godat, A.; Chaallal, O.; Obaidat, Y. Non-linear finite-element investigation of the parameters affecting externally-bonded FRP flexural-strengthened RC beams. *Results Eng.* **2020**, *8*, 100168. [\[CrossRef\]](#)
43. Cameselle-Molares, A.; Vassilopoulos, A.P.; Renart, J.; Turon, A.; Keller, T. Numerically-based method for fracture characterization of Mode I-dominated two-dimensional delamination in FRP laminates. *Compos. Struct.* **2019**, *214*, 143–152. [\[CrossRef\]](#)
44. American Concrete Institute; Ascione, L.; Baratta, A.; Bastianini, F.; Battista, U.; Benedetti, A. Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures. In *ACI 440.2R-08*; American Concrete Institute: Farmington Hills, MI, USA, 2008.
45. ACI Committee. Building code requirements for structural concrete (ACI 318-19): An ACI standard. In *Commentary on Building Code Requirements for Structural Concrete (ACI 318R-19)*; ACI Committee: Farmington Hills, MI, USA, 2019.
46. Comité Euro-International du Béton-Fédération Internationale de la Précontrainte. *CEB-FIP Model Code 1990: Design Code*; T. Telford: London, UK, 1993.
47. Saenz, L.P. Equation for the stress-strain curve of concrete in uniaxial and biaxial compression of concrete. *Open J. Civ. Eng.* **1965**, *61*, 1229–1235.
48. Hu, H.; Schnobrich, W.C. Constitutive Modeling of Concrete by Using Nonassociated Plasticity. *J. Mater. Civ. Eng.* **1989**, *1*, 199–216. [\[CrossRef\]](#)
49. Geubelle, P.H.; Baylor, J.S. Impact-induced delamination of composites: A 2D simulation. *Compos. Part B Eng.* **1998**, *29*, 589–602. [\[CrossRef\]](#)
50. Camanho, P.P.; Davila, C.G.; de Moura, M.F. Numerical Simulation of Mixed-Mode Progressive Delamination in Composite Materials. *J. Compos. Mater.* **2003**, *37*, 1415–1438. [\[CrossRef\]](#)
51. Nguyen, X.T.; Park, J.S. Flexural Behavior of Steel Beams Strengthened with CFRP under Fire. *Int. J. Steel Struct.* **2022**, *1*–17. [\[CrossRef\]](#)
52. Benzeggagh, M.L.; Kenane, M. Measurement of mixed-mode delamination fracture toughness of unidirectional glass/epoxy composites with mixed-mode bending apparatus. *Compos. Sci. Technol.* **1996**, *56*, 439–449. [\[CrossRef\]](#)
53. Lezgy-Nazargah, M.; Dezhangah, M.; Sepehrinia, M. The Effects of Different FRP/Concrete Bond-Slip Laws on the 3D Nonlinear FE Modeling of Retrofitted RC Beams—A Sensitivity Analysis. *Steel Compos. Struct.* **2018**, *26*, 347–360.
54. Camanho, P.P.; Dávila, C.G. *Mixed-Mode Decohesion Finite Elements for the Simulation of Delamination in Composite Materials*; NASA/TM: Hampton, VA, USA, 2002; Volume 211737.
55. Cui, W.; Wisnom, M.; Jones, M. A comparison of failure criteria to predict delamination of unidirectional glass/epoxy specimens waisted through the thickness. *Composites* **1992**, *23*, 158–166. [\[CrossRef\]](#)
56. Kim, N.; Shin, Y.S.; Choi, E.; Kim, H.S. Relationships between interfacial shear stresses and moment capacities of RC beams strengthened with various types of FRP sheets. *Constr. Build. Mater.* **2015**, *93*, 1170–1179. [\[CrossRef\]](#)
57. Xie, J.; Waas, A.M.; Rassaian, M. Analytical predictions of delamination threshold load of laminated composite plates subject to flexural loading. *Compos. Struct.* **2017**, *179*, 181–194. [\[CrossRef\]](#)