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Simulation of Concrete Beams Strengthened by Embedded Through-section Steel and GFRP Bars with Newly Developed Bond Model

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Abstract
This paper presents an experimental and numerical investigation on concrete members strengthened by embedded through-section (ETS) steel and glass fiber-reinforced polymer (GFRP) bars attached preferably with mechanical anchorage at the tension ends. The pullout tests to analyze the bond performance between ETS bars and concrete under various influences such as anchorage presence, embedment length, ETS bar diameter, ETS-material types, and anchorage length are carried out. An analytical method for deriving the local bond stress-slip relationship of GFRP bars-concrete interfaces is developed. The overall responses of the pullout test specimens in terms of pullout force-slip curves, failure modes and strain profiles along the embedment length are discussed. Based on a careful interpretation, the analytical results demonstrated the effectiveness of the local bond stress-slip model developed in this study. Additionally, the finite element (FE) simulation of the beams intervened with ETS bars, which were tested in a previous study by the authors, incorporating with the proposed interfacial model is conducted. Comparison between the results achieved from the FE modelling and the experiment implies that the FE method was an accurately applicable tool to assess the behaviors of the beams strengthened in shear by ETS bars.

1. Introduction

In recent years, the embedded through-section technique (ETS) can be considered a highly efficient technique, amongst the shear retrofitting methods such as externally bonding (EB) and near-surface mounting (NSM) methods, for strengthening in shear of the reinforced concrete (RC) members. The ETS method employs an adhesive to bond fiber-reinforced polymer (FRP) or steel bars embedded through pre-drilled holes into the concrete core. Owing to the fact that ETS bar is surrounded by concrete, the corrosion and fire attack to reinforcement would be limited. As reported in the experimental studies by Barros and Dalfré (2013), Chaallal et al. (2011), Mofidi et al. (2012), Breveglieri et al. (2014, 2015) and Linh et al. (2017b, 2020a, 2020b), the shear resistance of the strengthened RC beams would significantly increase using the ETS method.

Findings obtained in the previous works indicate that the behavior of the ETS shear strengthened beams has converted brittle shear failure into ductile flexural failure with either yielding of the longitudinal steel bars or the crushing of concrete in compression zone. For the numerical investigation, the studies by Godat et al. (2013) and Linh et al. (2020a) presented the finite element (FE) simulation of the concrete beams strengthened by ETS rods tested in their experimental works using DIANA (DIANA 2006) and ANSYS (ANSYS 2013) computer software. In their research, the overall responses of concrete beams retrofitted with carbon FRP (CFRP) and glass FRP (GFRP) bars such as the load-deflection relationship and the strain in ETS bars were considered. Their studies revealed that the agreement between the simulated results and the test data was acceptable although the perfect bond of the reinforcement and strengthening system to concrete was assumed. However, they did not take into account the performance of the retrofitted beams with the combined use of steel and ETS FRP bars in terms of the shear efficiency and cracking characteristics, which might be strongly affected by the bond mechanism of reinforcement to concrete.

Indeed, many studies stated that the interfacial behavior of the strengthened bars to concrete was a crucial issue that affected directly the performance of the RC members. The bonding deficiency of the ETS retrofitting system to concrete, resulting in the low effectiveness of ETS strengthening system, was depicted in the previous studies. As reported by Linh et al. (2018b, 2020a, 2020b), either the bonding performance between the ETS bars and concrete is enhanced or the bonding

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inefficiency is compensated by an additional device such as a mechanical anchorage system at the tension ends of the ETS bars in order to trigger the ultimate effectiveness of the ETS strengthening system. However, very few experimental works of the ETS method with pullout tests (Godat et al. 2012; Caro et al. 2017) were found. These studies focused on the bond-slip response of ETS CFRP and GFRP bars embedded into concrete blocks considering several factors. None has ever conducted experiments of ETS bars embedded into concrete blocks with mechanical anchorage at the tension ends nor investigated strain profiles along the ETS bars to examine the bond mechanism. Furthermore, the bond laws developed in their research representing the ETS bars-concrete interfaces have not gained high accuracy. Additionally, the use of the bond models obtained from the pullout tests for simulating numerically the bond response between the ETS bars and concrete under various effects, such as the anchorage presence, embedment length, ETS bar diameter, type of ETS material and anchorage length, are carried out and examined. This study aims to investigate the bond mechanism of concrete specimens embedded by ETS steel and GFRP bars inserting anchoring nuts at the tension ends. In addition, the objective of this work is also to concern on the powerful simulation of the ETS-strengthened beams tested in the previous research by the authors adopting the FE method. Therefore, this study is organized as follows.

(1) The pullout tests to analyze the bond mechanism of the ETS bars to concrete under various effects, such as the anchorage presence, embedment length, ETS bar diameter, type of ETS material and anchorage length, are carried out and examined.

(2) The results obtained from the experimental program in terms of the pullout force-slip relationships, failure mode and strain profiles along the embedment length are interpreted.

(3) Based on the bond test results, an analytical method for deriving the local bond stress-slip relationships of FRP bars-concrete interfaces is developed from the model proposed previously (Ueda et al. 2004; Dai et al. 2005, 2006; Linh et al. 2020a).

(4) By using the FE method, the bond model developed in this study is used to simulate numerically the ETS-strengthened beams with the anchorage systems for assessing the structural behaviors.

### Table 1 Properties of materials of the tested specimens.

| Specimens | \( f'_c \) (MPa) | \( E_y \) (GPa) | \( f_t \) (MPa) | \( E_{ad} \) (GPa) | \( f_{t,ad} \) (MPa) |
|-----------|-----------------|---------------|----------------|-----------------|------------------|
| C1-C3, C5-C9 | 38 | 50 | 1,076 | 3.1 | 21.0 |
| C4         | 38 | 200 | 400  | 3.1 | 21.0 |

Note: \( f'_c \) = Concrete compressive strength (MPa); \( E_y \) = Young’s modulus of ETS bar (GPa); \( f_t \) = Tensile strength of ETS bar (MPa); \( E_{ad} \) = Young’s modulus of adhesive (GPa); \( f_{t,ad} \) = Tensile strength of adhesive (MPa).

### Table 2 Configuration, ultimate load, maximum slip and failure mode of the tested specimens (Linh et al. 2020a).

| Specimens | \( L_e \) (mm) | \( d_b \) (mm) | ETS material | Anchorage | No. of anchoring nuts | Anchorage length, \( A \) (mm) | Ultimate force, \( F_t \) (kN) | Maximum slip, \( s_m \) (mm) | Failure mode |
|-----------|----------------|---------------|-------------|------------|----------------------|-----------------------------|-----------------------------|------------------------|-------------|
| C1        | 150            | 10            | GFRP        | No         | -                    | -                           | 26.5                       | 84.5                  | 0.27        | Pullout     |
| C2        | 150            | 10            | GFRP        | Yes        | 4                    | 32                          | 30.3                       | 84.5                  | 0.42        | Rupture     |
| C3        | 120            | 10            | GFRP        | Yes        | 4                    | 32                          | 37.9                       | 84.5                  | 0.64        | Rupture     |
| C4        | 150            | 8             | GFRP        | No         | -                    | -                           | 32.1                       | 54.1                  | 1.16        | Rupture     |
| C5        | 200            | 10            | GFRP        | Yes        | 4                    | 32                          | 39.2                       | 84.5                  | 0.48        | Rupture     |
| C6        | 250            | 10            | GFRP        | Yes        | 4                    | 32                          | 37.4                       | 84.5                  | 0.98        | Rupture     |
| C7        | 120            | 12            | Steel       | Yes        | 4                    | 32                          | 60.7                       | 45.2                  | 0.20        | Rupture     |
| C8        | 120            | 10            | GFRP        | Yes        | 2                    | 16                          | 35.0                       | 84.5                  | 0.55        | Pullout     |
| C9        | 120            | 10            | GFRP        | Yes        | 6                    | 48                          | 37.1                       | 84.5                  | 0.48        | Rupture     |

Note: \( L_e \) = Embedment length (mm); \( d_b \) = ETS bar diameter (mm); \( F_t \) = Tensile force capacity of ETS bar (kN); \( s_m \) = Maximum slip (mm).

### 2. Experiment on bond mechanism between ETS bars and concrete

#### 2.1 Description of the test specimens

The design configuration and material properties for nine specimens, and the steel anchoring nuts are shown in Figs. 1(a), 1(b) and in Tables 1 and 2. The strain gauges glued on the ETS steel/GFRP bars are illustrated in Fig. 1(a) and the pullout test setup is demonstrated in Fig. 1(c). The specimens were divided into five groups to investigate the effects of anchorage presence (C1, C2), embedment length (C2, C3, C5, C6), bar diameter (C1, C4), ETS material types (C3, C7) and anchorage length (C3, C8, C9) on the bond response between the ETS bars and concrete. The concrete blocks were drilled through the depth for inserting the ETS bars by the adhesive. The concrete blocks at the two ends of the ETS bar were then drilled with a hole bigger than the remaining part. Afterwards, the anchoring nuts were installed at a tension end of ETS bar. At the other end of the ETS bar, the plastic tube with length of 30 mm was embedded to create the unbonded part. The adhesive was injected in the bond zone equal to the embedment
length of the ETS bar. The strain gauges were glued along the embedment length with the interval of 30 mm. The distance between the strain gauge closest to the anchored end and the start point of the bond zone was 60 mm.

In addition, the distance between the strain gauge closest to the loaded end and the end of the adhesive part was 20 mm. On the other hand, the maximum forces at tension fracture (ultimate forces) measured from the test of ETS steel (12 mm diameter) and GFRP bars (8 and 10 mm diameter) are shown in Table 2. In addition, together with the ultimate forces, the tensile forces calculated by multiplying the cross-section area of ETS steel/GFRP bars to the tensile strength of materials are presented in Table 2. In particular, for ETS steel case, the tensile strength is defined by the yielding strength; therefore, the tensile force capacity of ETS steel bars (specimen C7) is smaller than the ultimate force, 45.2 kN compared to 60.7 kN.

2.2 Results and discussion
In this section, the influences of the mechanical anchorage presence, the embedment length, the ETS bar diameter, the ETS type and the anchorage length on the pullout force-slip relationship are expressed. Additionally, the average bond stress-slip relationships and the strain profiles along the bond length are reported. The average bond stress ($\tau$) and the slip ($s_i$) at strain gauge (SG) closest to the loaded end are computed as the following equations.

![Fig. 1](image-url)
\[ \tau = \frac{P}{\pi d_b L_e} \]  

\[ \varepsilon_i = \frac{A_x}{2} \left( \varepsilon_0 + \frac{2}{\sum_{j=1}^{i} \varepsilon_j} \right) \]

where \( P \) (kN) is the pullout force, \( L_e \) (mm) is embedment length, \( d_b \) (mm) is ETS bar diameter, \( A_x \) (mm) is the strain gauge interval of 30 mm, \( \varepsilon_0 \) (µm) is the strain of ETS bar at the free end and \( \varepsilon_i \) (µm) is the strain at gauge of \( i \)th-order.

### 2.2.1 Effect of mechanical anchorage presence

The test results of the specimen C1 without anchoring nuts and the specimen C2 with anchorage presence are assessed. It is obvious from Fig. 2(a) that the initial response before the mechanical anchorage being activated is identical between these specimens. In Table 2, the specimen C2 with anchorage attachment results in the significantly higher maximum pullout force and maximum slip than those obtained by the test of the specimen C1 without anchorage by 14.3% (for pullout force) and 55.5% (for slip). The final failure modes of the blocks C1 and C2 are the pullout of ETS bar and the rupture of ETS bar, respectively. Clearly, at the load level equaled to the peak load of the specimen C1, at which the specimen C1 failed by pulling out of ETS bar, the anchorage in the specimen C2 started to be activated, utilizing the contribution of the ETS bar ultimately. The pullout force transfer at high load level was shifted from the adhesive to the anchorage. Thereby, the failure mode of the block C2 is different from that of the specimen C1. Indeed, Fig. 2(b) indicates that the strain of gauge (SG1) closest to the anchorage in the specimen C2 started to increase at the peak load of the test specimen C1. While the strain at SG5, which was closest to loaded end of the ETS bars, of the specimens C1 and C2 is similar at the triggering load. These imply that the use of anchorage enhanced drastically the tension capacity of the ETS rod at the anchored end. Consequently, the ETS bar worked ultimately in changing the failure mode to be efficient.

#### 2.2.2 Effect of embedment length

Figure 3 shows the pullout force-slip relationships of the specimens C2, C3, C5 and C6. In Fig. 3, generally, the ultimate pullout forces of the specimens C2, C5 and C6 were similar since the failure mode was the GFRP bar rupture as mentioned in Table 2. The specimen C2 exhibited a lower pullout force than that of the others due to the significant premature fracture. The fracture of the ETS GFRP bars in the specimens C2, C3, C5 and C6 occurred at the gripping. As shown in Table 2, the ultimate pullout forces of the blocks C2, C3, C5 and C6 were much smaller than the GFRP tensile strength (\( F_t \)) due to the premature tension rupture. These observations indicate that the premature rupture may depend on the detailing of the anchorage and gripping.

#### 2.2.3 Effect of bar diameter

The test results of the specimens C1 and C4, in which the bar diameters are 10 and 8 mm respectively, are compared. It is obvious from Table 2 and Fig. 4(a) that the specimen C1 with ETS GFRP bar diameter of 10 mm showed lower ultimate pullout force and smaller maximum slip than those of the specimen C4 with ETS...
2.2.4 Effect of ETS types

The experimental results of the specimens C3 and C7 embedded by ETS GFRP bar and ETS steel bar, respectively, are employed. Table 2 and Fig. 4(b) reveal that the concrete block embedded by ETS steel bar showed higher the maximum pullout force (ultimate force). However, at the same load level, the slip computed by Eq. (2) of the specimen C7 with ETS steel case is no greater than that in the specimen C3 with ETS GFRP case since the Young’s modulus of GFRP was small, inducing the large strain. The failure modes of these specimens are the failure with ETS rupture. In addition, as shown in Table 2, the premature rupture occurred in the specimen C3 with ETS GFRP bars since the actual ultimate force was lower than the tensile force defined by the material properties. This is because the steel gripping constraining ETS GFRP bars was significantly stiffer than the GFRP bars and caused localized stress concentration. Whereas, the specimen C7 with ETS steel bar exhibited the non-premature of ETS bar because the actual ultimate force was higher than the tensile force capacity defined by the yielding strength of steel. This is mainly because the yielding of ETS steel bar can ease the localized stress concentration effects. Besides, with the high elastic modulus of steel, the specimen with ETS steel bar (C7) resulted in higher rigidity than that of the block embedded by ETS GFRP bar (C3). Moreover, the slip development in specimen with ETS GFRP bar performs the non-linearity due to the non-linearity in strain development in ETS GFRP bar, which will be emphasized in Section 3.

2.2.5 Effect of anchorage length (number of anchoring nuts)

The specimens C3, C8 and C9 embedded by ETS GFRP bars were with the different numbers of anchoring nuts, \( A = 4 \), \( A = 2 \) and \( A = 6 \), respectively. It is clear from Fig. 5(a) and Table 2 that with the longer anchorage length (or more anchoring nuts), the specimens C3 and C9 resulted in higher pullout force than the specimen C8 with the short anchorage length. The failure modes of the specimens C3 and C9 were the fracture of ETS bars, while the specimen C8 was failed by the pullout of the ETS bar leaving the nuts in concrete [Fig. 5(b)]. This fact indicates that the two nuts are not enough to assure the full tension capacity of ETS GFRP bar.

3. Analysis of bond response between ETS bars and concrete

A development for evaluating the ETS bond behavior has been closely shown in the previous study by the authors (Linh et al. 2020a). This section only aims to highlight the prominent parts in the development of the bond model, which will be utilized for the numerical simulation in Section 4. Therefore, the necessary figures and tables in the authors’ previous work (Linh et al. 2020a) are reused to enhance the discussion and interpretation with permission obtained from the copyright holder.
holders. Figure 6(a) describes the force equilibrium in arbitrary section and the free body diagram of concrete-ETS bar interface. Obviously, from Fig. 6(a), the equilibrium equation can be obtained as below.

$$dF_r(x) = A_r d\sigma_r(x) = p_r \tau(x) dx$$

(3)

where $p_r$ is the perimeter of the bar, $\tau(x)$ is the bond stress, $A_r$ is the cross-sectional area of ETS bar, $d\sigma_r(x)$ is the tensile stress in ETS bar, and $dF_r(x)$ is the axial force in ETS bar at a segment $dx$. The uniaxial constitutive relationship for the linear elastic ETS bar elements is shown in Eq. (4).

$$F_r(x) = E_r A_r \varepsilon_r(x)$$

(4)

Fig. 5 (a) Comparison in the bond responses between the cases of anchorage length changes; (b) premature rupture failure of specimen C9 and pullout failure of specimen C8.

It can be assumed that slip at anchored end and slip at starting point of anchorage are negligible, because strain at those points is very small.

Fig. 6 Components in ETS bar: (a) free body diagram of the ETS bar interface; (b) the extrapolation of strain distribution of a representative specimen (C5) showing boundary condition in strain at anchored end could be assumed by zero (Linh et al. 2020a).
where $F_r(x)$ is the axial force along the $x$-axis, $E_r$ is the elastic modulus of bar and $\varepsilon_r(x)$ is the axial strain at a distance $x$. Additionally, from the above equations, it can be written as Eq. (5). Herein, $s(x)$ is defined for the slip function of $x$ along embedment length.

$$\frac{d^2s(x)}{dx^2} = \frac{dF_r(x)}{dx} = \frac{dE_r(x)}{dx} \frac{p_r(x)}{E_r A}$$  \hspace{1cm} (5)

To analyze the local bond stress at the interfacial locations where the strain gauges were glued, Eq. (5) can be expressed by Eq. (6). Based on the extrapolation of the strain distribution from closest loaded end to anchored end as shown in Fig. 6(b), the boundary condition is the slip at the anchored end, which could be assumed to be zero. In fact, the anchored end at $x = 0$ (mm) was constrained by the anchoring nuts [as demonstrated in Fig. 6(b)], meaning the strain can be very small in anchorage, thus the slip at starting point of anchorage is negligible.

$$\tau_i = \frac{E_r A}{p_r} (\varepsilon_i - \varepsilon_{i-1})$$  \hspace{1cm} (6)

where $\tau_i$ (MPa) is the average interfacial bond stress at the section $i$, $\varepsilon_i$ and $\varepsilon_{i-1}$ ($\mu\varepsilon$) are the strains at gauges of ($i$)th-order and ($i$-1)th-order, respectively.

By using the mathematical formulation featured in Eq. (6) for the local bond stress and in Eq. (2) for the local slip, the local $\tau$-$s$ relationships at the different interfacial locations in a pullout test of the specimen C4 are shown in Fig. 7(a). The same as what have been stated by Dai et al. (2005) and Linh et al. (2020a), the strain at each gauge attached on the ETS bar is expressed as an exponential function of the corresponding slip [Eq. (7)]. Equation (5) implies that the $\tau$-$s$ curves can be determined if the local strain and slip relationships were defined.

$$\varepsilon = f(s) = A(1 - e^{-B_s})$$  \hspace{1cm} (7)

where $A$, $B$ are experimental regressing parameters as given in Table 3. Figure 8 shows the experimental and regressed curves of the strain-slip relationships at the position closest to the loaded end of the test specimens. It is clearly observed in Fig. 8 that the exponential expression as Eq. (7) could fit the experimental results well, in which the correlation factor values $R^2$ are in the range from 0.975 to 0.999 for all the specimens (as shown in Table 3). By substituting Eq. (7) into Eq. (5), the bond stress and slip relationship can be described as follows.

$$\tau = \frac{E_r A}{p_r} A^2 Be^{-B_s} (1 - e^{-B_s})$$  \hspace{1cm} (8)

The interfacial fracture energy $G_f$ and the theoretical maximum pullout force $P_{\text{max}}$ can be defined as follows.

$$G_f = \int_0^\infty \tau \, ds = \frac{A^2 E_r A}{2 p_r}$$  \hspace{1cm} (9)

$$P_{\text{max}} = E_r A \varepsilon_{\text{max}} = E_r A \varepsilon_{\text{E}} = E_r A \sqrt{2G_f \frac{p_r}{E_r A}}$$  \hspace{1cm} (10)

where $\varepsilon_{\text{max}}$ ($\mu\varepsilon$) is the maximum strain of ETS bars corresponding to the maximum pullout force. The maximum slip ($s_{\text{max}}$) corresponding to the maxi-
Mum bond stress ($\tau_{\text{max}}$) can be derived as below.

$$\frac{d\tau}{d s} = 0 \Rightarrow s_n = \frac{\ln 2}{B} \text{ and } \tau_n = 0.5BG_f$$

Also presented by Linh et al. (2020a), Fig. 9 expresses the comparison between the analytical maximum pullout forces computed by Eq. (10) and the experimental maximum pullout loads. As shown in Fig. 9 and Table 3, the mean value of the ratio of the theoretical values to the tested values is 0.96 and the coefficient of variation is 12.1% of the mean. Therefore, the agreement between the calculation and the measurement in the pullout force is well gained. On the other hand, the experimental and analytical curves of the local bond stress and slip relations at the gauges closest to loaded end of the test specimens are depicted in Fig. 10. The results derived by Eq. (8) fitted well with the tested data, especially in the ascending branch of the curves, meaning the developed bond model is acceptable in performing the interfacial mechanism.

Various effects of anchorage availability, embedment length, bar diameter and anchorage length on the bond response are also interpreted in Fig. 10. Owing to the influence of anchorage at the bar end, the strain in ETS bar of the anchored specimen (C2) was smaller than that in ETS bar of the unanchored block (C1). This finding is that the maximum bond stress of the specimen C2 was less than that of the specimen C1. For the effect of ETS bar diameter, as reported by Dai et al. (2005), Shrestha et al. (2017) and Linh et al. (2020a), the bond stress increases as the stiffness ($E_A/p_r$) of the interfaces increased. In addition, the longer anchorage length (or more anchoring nuts) resulted in the greater bond stress because the more anchoring nuts, which implied the shorter bond length, may increase the strain at the stage before the anchorage become efficient.

The interfacial ductility index ($B$) and interfacial fracture energy ($G_f$) are affected by the various influences.
In Fig. 10 and Table 3, the specimen without anchorage (C1) increased the ductility but decreased the fracture energy compared to those terms found in the specimen with anchorage presence (C2). The specimen C1 that was with higher stiffness results in lower ductility and larger energy than those of the specimen C4 that was with lower stiffness. The increase of the anchoring nuts, considering blocks C8 (2 nuts), C3 (4 nuts) and C9 (6 nuts), offers the high interfacial energy and the low ductility index. Furthermore, the interfacial ductility index \( B \) and interfacial fracture energy \( G_f \) could be derived by Eqs. (9), (10) and (11) if the maximum strain in the ETS bars have been known. These factors are important to apply reasonably the bond model in analyzing the members strengthened by the ETS FRP bars. How the \( B \) and \( G_f \) factors to be employed in the analysis of the concrete beams intervened with ETS GFRP bars is discussed in the following section.

4. Finite element (FE) modelling of concrete beams strengthened with ETS bars

4.1 Experimental data

The experimental data of the study by Linh et al. (2020b) are utilized to carry out the FE simulation of the concrete beams strengthened by ETS shear bars and reinforced with ordinary steel shear reinforcement. The design configuration of 11 specimens for experimental program is clearly shown in Fig. 11 and Table 4. There were three reference beams (R1, R2 and R3), two internally diagonal-vertical steel reinforced beams (A1 and A2), two concrete beams strengthened by ETS steel bars (A3 and A4), and four concrete beams retrofitted with ETS GFRP bars (B1, B2, B3 and B4). Three reference beams were respectively designed as follows. The case was of concrete only in the shear span L1 (beam R1). The case was of two steel stirrups with 6 mm bar diameter with 300 mm spacing in the shear span L1 (beam R2). The case was of two steel stirrups with bar diameter of 9 mm with 300 mm spacing in the shear span L1 (beam R3). The positions of the strain gauges are also marked in Fig. 11. Besides, Table 5 describes the properties of the materials employed in the experiment.

4.2 Finite element program

Numerical analyses are conducted by a commercially available software, ANSYS version 15.0 (ANSYS 2013). A half three-dimensional FE model is applied to investigate the performance of the tested beams based on the symmetric condition as shown in Fig. 12. For this investigation, the mesh discretization is \( 25 \times 25 \) mm\(^2\). In addition, the descriptions of element types and material models for the FE program are presented as follows.
which have been shown in the previous works (Linh et al. 2018a, 2018c) of the authors. Therein, several constitutive models for concrete, steel and FRP bars, which were accommodated in the ANSYS software, have been adopted in the FE simulation. One of the prominent points of this study is to propose the way to input the

![Diagram of test beams configuration](image)

Fig. 11 Configuration of the 11 tested beams in study by Linh et al. (2020b) (dimensions in mm).
bond model, which was developed in Section 3, into an element type available in ANSYS.

4.2.1 Element types
SOLID65, LINK180 and SOLID45 are used as the elements in ANSYS 15.0 for the nonlinear 3D modeling of concrete materials, reinforcement and rigid steel support, respectively. The SOLID65 element is capable of cracking in tension and crushing in compression. The SOLID65 element is defined by eight nodes. While, the LINK180 is an uniaxial tension-compression element with three degrees of freedom at each node that the translations in the nodal x, y, and z directions (Linh et al. 2018c, 2020a). Besides, the SOLID45, which has the same properties as SOLID65 except for the capability of cracking in tension and crushing in compression (Hawileh 2015; Linh et al. 2017a, 2018c), is applied for the supporting and loading plates. The non-linear spring element COMB39 in ANSYS is introduced to simulate the interfacial bond behavior of ETS bars and steel reinforcement to concrete. The COMB39 requires the average bond force-slip relationship at the interface of the reinforcement (including ETS bars and internal reinforcement) to input into ANSYS.

4.2.2 Material models
In this section, the model of Hognestad et al. (1955) is described for simulating the nonlinear behavior of concrete in compression. Equation (12) and Fig. 13(a) show the details of the Hognestad parabola.

$$f_c = f'_c \left[ 2 \left( \frac{\varepsilon}{\varepsilon_0} \right) - \left( \frac{\varepsilon}{\varepsilon_0} \right)^2 \right]$$

where $f_c$ is the compressive stress of concrete (MPa) corresponding to the specified strain ($\varepsilon$), $f'_c$ is the concrete compressive strength (MPa) and $\varepsilon_0 = \frac{2f'_c}{E_c}$ with $E_c$ being the elastic modulus of concrete (MPa).

Linh et al. (2018c, 2020a) showed the concrete behavior in tension according to the model of William and Warnke, which was recommended by ANSYS software. Figure 13(b) shows the stress-strain relationship of

### Table 4 Reference, hybrid and ETS shear strengthening configuration of the 11 tested beams.

| Beam ID     | Number of ETS/hybrid bars | Inclination of hybrid/ETS bars (°) | Hybrid bars/ETS bars spacing (mm) | Existing steel stirrups ratio (%) | Hybrid/ETS reinforcement ratio (%) |
|-------------|---------------------------|-----------------------------------|-----------------------------------|----------------------------------|-----------------------------------|
| R1-0S-0ETS  | 0                         | NA                                | NA                                | 0.00                             | NA                                |
| R2-2Sd6-0ETS| 0                         | NA                                | NA                                | 0.11                             | NA                                |
| R3-2Sd9-0ETS| 0                         | NA                                | NA                                | 0.24                             | NA                                |
| A1-2Sd6-5Sd6(90) | 5                      | 90                                | 180                               | 0.11                             | 0.18                              |
| A2-2Sd6-5Sd6(45) | 5                      | 45                                | 180                               | 0.11                             | 0.25                              |
| A3-2Sd6-5ETS Steel d12(90) | 5          | 90                                | 180                               | 0.11                             | 0.35                              |
| A4-2Sd6-5ETS Steel d12(45) | 5          | 45                                | 180                               | 0.11                             | 0.50                              |
| B1-2Sd6-5ETS FRP d10(90) | 5          | 90                                | 180                               | 0.11                             | 0.24                              |
| B2-2Sd6-5ETS FRP d10(45) | 5          | 45                                | 180                               | 0.11                             | 0.34                              |
| B3-2Sd9-5ETS FRP d10(90) | 5          | 90                                | 180                               | 0.24                             | 0.24                              |
| B4-2Sd9-5ETS FRP d10(45) | 5          | 45                                | 180                               | 0.24                             | 0.34                              |

### Table 5 Properties of materials of the 11 tested beams.

| Beam ID     | Concrete compressive strength at tested day (MPa) | Young modulus of hybrid/ETS reinforcement (GPa) | Tensile strength of hybrid/ETS reinforcement (MPa) | Young's modulus of adhesive (GPa) | Tensile strength of adhesive (MPa) |
|-------------|-----------------------------------------------|-----------------------------------------------|---------------------------------------------------|---------------------------------|-----------------------------------|
| R1-0S-0ETS  | 35                                            | NA                                           | NA                                                | NA                              | NA                                |
| R2-2Sd6-0ETS| 35                                            | NA                                           | NA                                                | NA                              | NA                                |
| R3-2Sd9-0ETS| 38                                            | 200                                          | 240                                               | NA                              | NA                                |
| A1-2Sd6-5Sd6(90) | 35            | 200                                          | 400                                               | 3.1                             | 21.0                              |
| A2-2Sd6-5Sd6(45) | 35            | 200                                          | 400                                               | 3.1                             | 21.0                              |
| A3-2Sd6-5ETS Steel d12(90) | 35          | 200                                          | 400                                               | 3.1                             | 21.0                              |
| A4-2Sd6-5ETS Steel d12(45) | 35          | 200                                          | 400                                               | 3.1                             | 21.0                              |
| B1-2Sd6-5ETS FRP d10(90) | 38          | 50                                           | 1,076                                             | 3.1                             | 21.0                              |
| B2-2Sd6-5ETS FRP d10(45) | 38          | 50                                           | 1,076                                             | 3.1                             | 21.0                              |
| B3-2Sd9-5ETS FRP d10(90) | 38          | 50                                           | 1,076                                             | 3.1                             | 21.0                              |
| B4-2Sd9-5ETS FRP d10(45) | 38          | 50                                           | 1,076                                             | 3.1                             | 21.0                              |
Fig. 12 A half typical FE model for numerical program by using ANSYS 15.0 (dimensions in mm).

Fig. 13 Material models: (a) concrete in compression (Linh et al. 2018c); (b) concrete in tension (Linh et al. 2018c); (c) proposed bond force-slip model for COMBIN39 element applied in ANSYS.
concrete in tension. The following description can be found in the study by Linh et al. (2018c). At first, the linear elasticity to the concrete tensile strength is used for concrete behavior in tension. Then, a steep drop in the concrete tensile stress by 40% is the stress relaxation in tension. The rest of model is represented as the curve which descends linearly to zero tensile stress at a strain value six times larger than strain value at the concrete’s tensile strength (Hawileh 2015).

The anchorage at the ETS bar ends is assigned with slip neglection at the starting point of anchorage, as demonstrated in Fig. 12. The bond model represented the interfacial mechanism of ETS bars to concrete is inserted in the part outside the anchorage. Figure 13(c) reveals that each ETS bar, which the critical crack plane passed, contributed to shear resistance of the strengthen system through the bonding mechanism between the ETS bar, adhesive and concrete. In addition, the bond behavior is considered the pullout response of each ETS bar to the covering concrete block. Number of the ETS bars, which contributed to the shear resistance, is defined by the amount of the ETS bars that were crossed by the main crack plane [Eq. (13)]. Moreover, the effective bond length at each ETS bar and average bond length in the assumed concrete blocks covering the corresponding ETS bar are calculated by Eqs. (14) and (15).

\[
N_f = \text{round off} \left[ \frac{h}{\cot \theta + \cot \alpha} \right] \tag{13}
\]

\[
L_y = \begin{cases} 
\frac{is_{\mu} \sin \theta}{\sin (\theta + \alpha)} & \text{for } x_y < \frac{h}{2} (\cot \theta + \cot \alpha) \\
L_y - is_{\mu} \frac{\sin \theta}{\sin (\theta + \alpha)} & \text{for } x_y \geq \frac{h}{2} (\cot \theta + \cot \alpha)
\end{cases} \tag{14}
\]

\[
\overline{L_y} = \frac{1}{N_f} \sum L_y \tag{15}
\]

where \(x_y = is_{\mu} \), \(\theta\) and \(\alpha\) are the crack and reinforcement angles respectively.

Since the failure mode of the specimens with GFRP is premature rupture of GFRP bars as shown previously, the interfacial fracture energy \((G_f)\) is below the bonding interfacial fracture energy. Moreover, the tested beams described in this section failed by the fracture of concrete in shear zone without ETS bar rupture. Therefore, the mathematical formulations developed by the bond tests in Section 3 can be adopted for the bond model between ETS anchored bars and concrete in the strengthened beams. It is indicated in Table 6 that the average bond lengths of the ETS bars in the tested beams are approximately 120 mm (for B1, B3) and 150 mm (for B2, B4). Besides, in Table 6, the bond constants \((A, B)\) computed from the tested beams are in range of the bond constants \((A, B)\) calculated from the pullout test varying embedment lengths. Indeed, in Table 6, the increase of average bond length from 120 mm to 150 mm results in the values of \(A\) ranging from 0.00760 to 0.01468 and \(B\) ranging between 4.12 and 2.59. For the ETS bars and steel reinforcement, which the strain gauges were glued, Fig. 13(c) presents the technique to assign the bond model proposed in Section 3 to the FE models through COMBIN39.

On the other hand, the steel reinforcement is described as the elastic fully plastic model based on the von Mises yield criterion. While, the FRP bars are simulated as elastic-brittle materials till rupture. Figure 14 shows the stress-strain relationships of steel and FRP reinforcement that is applied in the FE simulations. In conclusion, the used constitutive models comply with the element types available in ANSYS for simulating the beams strengthened with anchored ETS bars.

### Table 6 Comparison between constants \((A, B)\) in bond test and beam test applied in simulation.

| Specimens | \(L_y\) (mm) | \(E_iA_i/p_i\) (kN/mm) | Specimens | \(L_y\) (mm) | \(E_iA_i/p_i\) (kN/mm) |
|-----------|-------------|----------------|-----------|-------------|----------------|
| C3        | 120         | 125             | B1        | 121.8       | 125             | 0.01377     | 2.39       |
| C2        | 150         | 125             | B2        | 155.6       | 125             | 0.01039     | 2.47       |
| C5        | 200         | 125             | B3        | 116.1       | 125             | 0.01089     | 3.16       |
| C6        | 250         | 125             | B4        | 151.1       | 125             | 0.01025     | 2.58       |

Fig. 14 Stress-strain relationships of steel and GFRP reinforcement (Linh et al. 2018c).
analysis is the stress in internal or ETS reinforcement reaching their yield/rupture strength and the concrete strain in compression or diagonal region exceeding 0.003. The structural performance of the investigated beams by simulating using ANSYS in terms of the load-deflection response, the cracking failure and the strain in shear reinforcement are compared with the results obtained from the tests.

Table 7 Numerical and experimental results in load capacity ($P$), shear strength ($V_{\text{exp.}}$ and $V_{\text{num.}}$) and shear contribution of ETS system ($V_f$).

| Beam ID       | $P_{\text{exp.}}$ (kN) | $P_{\text{num.}}$ (kN) | Difference in load (%) | $V_{\text{exp.}}$ (kN) | $V_{\text{num.}}$ (kN) | $V_{f, \text{exp.}}$ (kN) | $V_{f, \text{num.}}$ (kN) |
|---------------|-------------------------|-------------------------|------------------------|-----------------------|------------------------|-----------------------------|-----------------------------|
| R1-0S-0ETS    | 171.8                   | 176.0                   | 2.44                   | 103.1                 | 105.6                  | NA                         | NA                         |
| R2-2Sd6-0ETS  | 223.4                   | 220.0                   | 1.52                   | 134.0                 | 132.0                  | NA                         | NA                         |
| R3-2Sd9-0ETS  | 345.4                   | 347.1                   | 0.49                   | 207.2                 | 208.3                  | NA                         | NA                         |
| A1-2Sd6-5Sd6(90) | 253.0                 | 266.0                   | 5.14                   | 151.8                 | 159.6                  | NA                         | NA                         |
| A2-2Sd6-5Sd6(45) | 335.1                 | 343.8                   | 2.60                   | 201.1                 | 206.3                  | NA                         | NA                         |
| A3-2Sd6-5ETS Steel d12(90) | 422.2             | 451.1                   | 6.85                   | 253.3                 | 270.7                  | 119.3                      | 138.7                      |
| A4-2Sd6-5ETS Steel d12(45) | 510.5             | 536.1                   | 5.01                   | 306.3                 | 321.7                  | 172.3                      | 189.7                      |
| B1-2Sd6-5ETS FRP d10(90) | 453.9             | 443.7                   | 2.25                   | 272.3                 | 266.2                  | 138.3                      | 134.2                      |
| B2-2Sd6-5ETS FRP d10(45) | 481.5             | 472.5                   | 1.87                   | 288.9                 | 283.5                  | 154.9                      | 151.5                      |
| B3-2Sd9-5ETS FRP d10(90) | 515.2             | 535.1                   | 3.86                   | 309.1                 | 321.1                  | 101.9                      | 112.8                      |
| B4-2Sd9-5ETS FRP d10(45) | 589.9             | 598.3                   | 1.42                   | 353.9                 | 359.0                  | 146.7                      | 150.7                      |

Fig. 15 Comparison between experimental and numerical results of tested beams in load-deflection relationship (continued in next page).
4.3.1 Load-deflection relationship

**Figure 15** presents the load-deflection curves of the test and the simulation results for beams. It is explicit that the FE results perform good appraisal in comparison with the tested data. A maximum deviation less than 10% in the load-carrying capacity is found from **Fig. 15** and **Table 7**. In general, the stiffness of the analyzed specimens is close to that of the tested beams because the bond-slip model for ETS method was applied in the FE simulation. As shown in **Fig. 15**, the displacement corresponding to the peak load of the simulated beams is slightly smaller than that of the tested specimens. It is the same as the experiment, the simulation results express that the rigidity of the members was enhanced as the shear reinforcement contents increased. In addition, the stiffness of the beams strengthened by ETS steel bars (A3, A4) is higher than that of the members retrofitted with ETS GFRP bars (B1, B2) due to the low Young’s modulus of GFRP bars. This discrepancy can be explained by the difference between the assumed bond-slip model and the actual bond-slip relationship. The relevant discussion can be found in the section below. In addition, the cracking caused by the bond deterioration might not be predicted in the simulated beams, resulting in the smaller displacement at high load level.

4.3.2 Shear contribution carried by ETS strengthening system

**Figure 16** and **Table 7** present the comparison between tested and simulated data in terms of shear contribution of ETS strengthening system. The shear contribution of the ETS-strengthened bars was derived through the subtraction of the shear strength of the reference beams from the shear strength of the ETS-strengthened beams, i.e., \( V_{ETS bars} = V_{strengthened beams} - V_{reference beams} \). It is obvious from **Fig. 16** and **Table 7** that the shear resistance of ETS bars computed by FE simulation agreed well...
with that of ETS-strengthened bars derived from experiment. In Fig. 16, it is similar to the test results that the shear contribution of strengthening system is enhanced as the ETS bars inclined at 45 degrees. Considering beams B1-B2 (stirrups 2Sd6) and B3-B4 (stirrups 2Sd9) with combined usage of steel and ETS FRP, the efficiency in shear of ETS rods reduced as the steel stirrups amount increased. However, as the stirrups amount increased, the decrease in the shear contribution of anchored ETS-strengthened bars were not deemed to be significant, meaning that the anchorage is an effective system for the ETS strengthening technique. These mentioned findings were also reported in the experimental investigation in the study by the authors (Linh et al. 2020b).

4.3.3 Cracking and failure mechanism
Figure 17 shows shear strain contour in XY plane of the simulated beams against the actual shear cracking pattern of the tested beams. Herein, the actual shear cracks are almost located in the zones with highest shear strain of the simulated beams. Thus, the FE method could pre-

Note: The gray color parts are the zones, where the shear strain exceeded ultimate strain. Most of experimental shear cracks passed the zone of gray color parts.

Fig. 17 Shear strain in XY plane compared to experimental shear failure cracks of analyzed specimens (continued in next page).
dict reasonably the shear failure region of beams reinforced/strengthened in shear with steel and FRP bars. Along with the experimental observation, all simulated beams failed in shear due to the significant and wider shear cracks in the beams. The rupture of the ETS bars and the debonding of the ETS bars to concrete did not occur in the simulation. Indeed, as observed in Fig. 18(b) for the specimen B1, at the failure load, the stresses in the ETS FRP bars and steel tension reinforcement did not reach rupture strength and yielding strength, respectively. While, the existing steel stirrups yielded and the concrete in shear zone heavily fractured showing the shear strain in \(XY\) plane exceeded ultimate value.

Moreover, Fig. 18(a) compares the crack pattern propagation at load steps of the ETS-strengthened beam B1. Clearly, Fig. 18(a) reveals the good agreement in the crack propagation between the test and the simula-

![Fig. 18 Structural behaviors in beam: (a) comparison between experimental and numerical results of crack propagation in shear span of a representative ETS beam B1; (b) stress evolution at maximum load in reinforcement (ETS bars, stirrups, tension bars) in shear span of a representative beam B1.]

Note: The gray color parts are the zones, where the shear strain exceeded ultimate strain. Most of experimental shear cracks passed the zone of gray color parts.
tion. The major shear cracks initiated on the beam’s web, midway between support and load point, then propagated toward both flange and support. Afterwards, the crack reached the flange and triggered an immediate failure with a quasi-horizontal crack angle. The increase of the contents of shear reinforcement resulted in more cracks and bigger shear diagonal crack angle in the ETS-strengthened beams. It is primarily because the shear resisting mechanism was significantly generated in the beams with the large amount of shear reinforcement. Since the ETS strengthening system was embedded through entire section introduced with the end anchoring nuts, the shear-resisting mechanism at the flange zone was strongly activated. Therefore, the beams strengthened with ETS steel/GFRP bars (A3, A4, B1-B4) exhibited the failure cracks in flange of beams. This phenomenon may result in the considerable ductility performance of the beams strengthened in shear with ETS end-anchored bars.

4.3.4 Strain in shear reinforcement

The comparisons of the strain in shear reinforcement for ETS bars and for steel stirrups are presented in Figs. 19 and 20, respectively. The load-strain curves in the ETS bars derived from the simulation are well fitted with those recorded from the test at the whole curves (as indicated in Figs. 19 and 20). Besides, from Fig. 19, the load activating the ETS strengthening system in the FE simulation is generally lower than the force activating the ETS strengthening system in the experiment. This difference can be explained by the possible bond-slip model discrepancy between the simulation and the experiment, as shown in section below. Numerical data indicated that for the same load, the strains in the ETS GFRP strengthened bars (in B1) were greater than those of the ETS steel strengthened bars (in A3). It is due to the low Young’s modulus (making low stiffness) of GFRP and the high fracturing strain of GFRP. Furthermore, for the beams with combined use of steel and ETS GFRP bars, the beam B1 with less percentage of steel stirrups gave the higher strains of the ETS reinforcement than those in the beam B3 with large stirrups amount. This finding implies the shear resisting force of the ETS-strengthened bars in the beams with the high amount of existing steel stirrups is smaller than that in the specimens with the less stirrups.

Fig. 19 Comparison between measured and simulated data in terms of strain in ETS reinforcement: (a) influence of existing stirrups content and strengthening inclination; (b) influence of vertical ETS strengthening types.

Fig. 20 Comparison between measured and simulated data in terms of strain in steel stirrups: (a) influence of inclined ETS retrofitting types; (b) influence of stirrups amount.
The strains in steel stirrups of the two beams A4 and B2 with different cases of ETS reinforcement (steel and GFRP) by means of the experiment and simulation are shown in Fig. 20(a). It is obvious that the FE results fitted well with the measured data. Additionally, the same as the experimental measurement, the strains determined by the FE simulation at the same position of the stirrups of the members A4 and B2 are almost similar at all points before yielding. Therefore, the different ETS types seemly disaffect the strain response of the existing transverse steel. Moreover, Fig. 20(b) indicates the good agreement in the comparison between tested data and simulated results in terms of strains in the transverse steel at same location of the three beams B2, B3 and B4. Considering the strain response of stirrups of the beams B2 and B4 (different existing stirrups amount, 2Sd6 for B2 and 2Sd9 for B4), the stirrups in these specimens yielded before the beams failed. In addition, the strains in the transverse steel of 9 mm bar diameter are no greater than those in the stirrup of 6 mm bar diameter over the entire load range. Therefore, the shear resistance of the ETS bars is reasonably distributed with the steel stirrups of bigger bar size, leading to a small contribution of the ETS bars in shear. In the beams B3 and B4 with the same existing steel stirrups and different inclinations of the ETS bars, the strains at SG3 in stirrups derived from the FE modeling are similar to the measured results for all the load range before yielding. This finding means that the ETS inclination did not affect the efficiency of stirrups.

4.3.5 Effect of bond behavior between ETS bars and concrete on structural response

As demonstrated from Fig. 21(a), the effect of bond interface between ETS bars and concrete is examined in three cases as follows. The ETS bars and concrete interfaces were with perfect bond, with anchored ends and without bond, and with bond-slip model consideration and anchored ends. The beam B3 is utilized to simulate the bonding influence. The interfacial models for internal stirrups and tension reinforcement to concrete are the same as those assigned in the simulation above. Figure 21(b) shows the load-deflection curves of three investigated cases considering effect of bond model on the performance of the beams strengthened with ETS bars. The initial behavior of beams with different ETS bond models is identical because the shear cracking load almost depends on the concrete property. After the occurrence of the shear diagonal cracks, the shear stiffness of the specimen assigned perfect bond is slightly higher than that of the specimen with the bond model because the fully bond of ETS bars to concrete caused the decrease of reinforcement strain as shown in Fig. 21(c).

Moreover, the overall stiffness is not much distinguished between the examined beams with perfect bond

![Diagram showing bond behavior](image-url)

Fig. 21 Effect of bond model on structural responses: (a) three investigated cases; (b) comparison in load-deflection curves; (c) comparison in load-ETS strain relationship.
and interfacial bond model. It is because the same bond model for tension bars-concrete, and the anchorage at tension ends of ETS bars were used. Additionally, since the anchorage was employed in the cases with fully bond and with bond model, the shear capacity of the investigated specimens is similar. Whereas, the simulated specimen with anchored ends only (no bond between ETS bars and concrete) offers the less shear stiffness and shear capacity compared to the other specimens. This finding is probably because the ETS bars were triggered by only anchorage, not occurrence of the shear transfer mechanism between ETS bars and concrete, resulting in the beam failed by premature concrete shear fracture. Moreover, to complement for the mentioned finding, Fig. 21(c) presents that the strain development in the ETS bars of the beam with anchored end only is earlier than that in the ETS bars of the others. This indicates that the specimens with anchored end only exhibited a premature shear deformation, causing the earlier concrete failure.

In addition, the simulated crack patterns of the three analyzed beams at ultimate load are shown in Fig. 22. It is obviously observed from Fig. 22 that the simulated cracking mechanism of the beam with the ETS bond model is well reflected the actual behavior. It means that the real cracks passed the zone where the shear stress in the simulation exceeded the ultimate values. While, also displayed in Fig. 22, the experimental crack pattern did not fit the maximum stress region in the simulated beam cases with perfect bond and single anchored ends. Additionally, in the case with single anchored ends, the cracking behavior shows heavy damage at the beams flange and web. Obtained finding is because the anchorage triggered the development in the strain in ETS bars (or the shear deformation) earlier, resulting in the premature concrete fracture.

In conclusion, the specimen with the developed bond model offers good agreement with the actual test in terms of structural behaviors. The reason is that the bond model of ETS bars to concrete considered the interfacial degradation response due to intersection by the main shear failure cracks, which was formulated from the experimental results. However, the actual degradation of bond stress may be occurred during the load steps, when the shear cracks formed to be intersected with the ETS bars. Therefore, the advanced analysis on this point is needed to capture the degradation of bond model. Furthermore, in the future study, to assess the effect of the difference bond model on the performance of the beams strengthened with anchored ETS bars, the simulation of the number of specimens should be carried out. Additionally, the simulation should consider the extensive factors such as the ratio between shear span and beams’ effective depth and the fatigue behaviors of the beams.

5. Conclusions

Based on the analyses, conclusions can be drawn as follows:

(1) Before the mechanical anchorage being activated, the initial response of the specimen embedded by
end-anchored ETS bars is the same as the corresponding specimen embedded by unanchored ETS bars. The specimen with anchorage presence results in the significantly higher maximum pullout force than that obtained by the specimen without anchorage. The ultimate pullout force is much smaller than the ultimate tensile force \( (F_t) \) based on GFRP tensile strength since the premature tension rupture at the gripping occurred.

(2) The specimen with ETS GFRP bar diameter of 10 mm offered low ultimate pullout force and small slip while the specimen with bigger bar diameter induced the weak interface leading the pullout failure. The concrete block embedded by ETS steel bar provided higher pullout force than the specimen embedded with ETS GFRP bar due to the premature rupture of GFRP bar. The more anchoring nuts specimen resulted in greater pullout force than fewer anchoring nuts one. The specimen with two anchoring nuts failed by the pullout of the ETS bar leaving the nuts. Therefore, to assure the full tension capacity of ETS GFRP bar, the number of anchoring nuts should be greater than two.

(3) The developed bond model based on the method of Dai et al. (2005) and that of Linh et al. (2020a) can satisfactorily capture the interfacial mechanism between the ETS FRP bars and concrete.

(4) The FE simulation with the developed bond model is a convenient tool to analyze the structural performance of the beams intervened in shear with ETS bars. Considering different bond mechanisms of ETS bars to concrete, the ETS-strengthened beam with single ETS-anchored ends resulted in the premature concrete failure due to the earlier shear deformation. Whereas, the performance of the strengthened beam analyzed numerically with ETS bond model agreed correctly with the actual structural responses. Aside from the developed interfacial mechanism, the further bond model improvement to consider the effects of bond degradation by shear crack intersection is needed.

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