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Effect of damage parameter variation on bond characteristics of CFRP-sheets bonded to concrete beams

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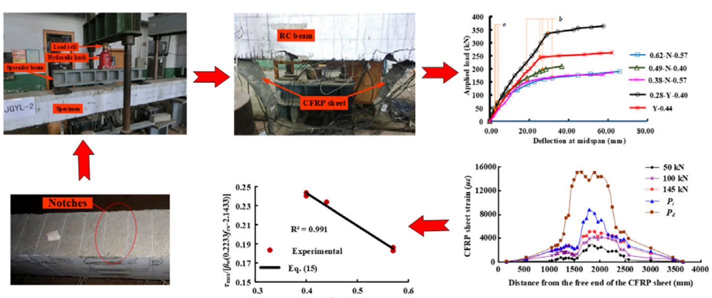
GRAPHICAL ABSTRACT



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ABSTRACT

The primary purposes of this study were to experimentally investigate the influence of the degree of damage in concrete beams on bond behavior of carbon-fiber-reinforced polymer (CFRP) sheets externally bonded to concrete beams. Five concrete beams at different damage levels (evaluated based on flexural stiffness) induced by cyclic loading were externally strengthened with CFRP sheets and were tested under four-point bending load. It was observed during this test that debonding of CFRP sheets appeared for each beam specimen, and propagated from near midspan toward the free end until fracture of CFRP sheets or crush of concrete flange occurred. It was found by comparing the test results and existing bond-slip model that the existing bond-slip model can not exactly predict the bond-slip relationship for CFRP sheets bonded to damaged concrete beams. In addition, the variation of damage parameter, prestress degree and preparation method of concrete surface (notched or non-notched) have little effect on the bond strength and the fracture energy of the CFRP-to-concrete interface, but the interfacial maximum shear stress decreased with an increase in the damage parameter.

1. Introduction

Externally bonded fiber-reinforced polymer (FRP) composites system has been verified to be an effective method for strengthening or retrofitting reinforced concrete (RC) structures [1]. Many researchers have conducted investigations on RC structures strengthened with externally bonded hybrid FRP systems [2–5]. An appropriate description of bond-slip relationship of the FRP-concrete interface provide a basement for accurate numerical and analytical solutions of externally bonded FRP composites system. Therefore, a thorough and comprehensive understanding of the bond-slip behavior of FRP-concrete interface is necessary for reasonable structural design of RC concrete structures externally bonded by FRP composites.

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Existing approaches adopted to investigate the bond behavior of FRP-concrete interface included mechanical tests (which commonly involved direct pull-out tests [6–8] and beam tests [9–12]), analytic method [13–15] and numerical simulation [16–20]. The available bond-slip models until now generally contained two stages, namely, ascending stage and descending stage, whether the model was linear or non-linear [16,21,22]. The main parameters involved in the existing bond-slip models are maximum bond shear stress, local slip at maximum shear stress, local slip when shear stress reduces to zero and interfacial fracture energy [16,23,24]. The interface bond behavior is believed to be related to various factors such as concrete strength, FRP bond length, FRP-to-concrete width ratio, the properties of FRP, the properties of the adhesive layer, temperature, loading method and the degree of the damage in concrete [16,25–31].

In recent two decades, many researchers engaged in investigations of bond-slip model for the FRP-concrete interface. Three bond-slip models of the FRP-concrete interface and expressions of the relevant model parameters, namely, maximum bond shear stress, local slip at maximum shear stress and interfacial fracture energy, were presented by Lu et al. [16] based on the predicted results of a meso-scale finite element model. These bond-slip models considered the effects of concrete strength, FRP-to-concrete width ratio and stiffness of adhesive layer. Afterwards, some other influencing factors were taken in to account in the bond-slip model. Haddad et al. [31] conducted experimental investigations on bond-slip behavior between CFRP sheets and heat-damaged concrete using pull-out tests. According to test results, a modified model was proposed based on the model presented by Lu et al. [16] considering the effect of post-heating damage of concrete. Based on pull-out test results, Biscaia et al. [32] developed a new bond-slip model for bonded joints between glass fiber reinforced polymer (GFRP) plates and concrete considering the effect of compressive stresses applied on the GFRP-concrete interface on condition that mechanical devices are used to anchor the externally bonded GFRP.

Although plenty of studies focused on bond-slip behavior of FRP-concrete interface have been carried out by researchers as mentioned above, the bond-slip performance between FRP and concrete with damages caused by applied loading was rarely involve in. Externally bonded FRP reinforcement can be an acceptable option to strengthen concrete structures which suffered performance degradation due to historical loading. Therefore, in this paper, experimental and theoretical studies were performed on bond-slip behavior of the CFRP-concrete interface considering the effect of different degrees of damages in concrete induced by cyclic loading. The main works completed by the authors includes: (a) beam tests conducted to evaluate the bond behavior between CFRP sheets and damaged concrete after cyclic loading, and (b) analysis on the influences of different degrees of damage in concrete beams on CFRP-to-concrete interface bond strength and key parameters of interface bond-slip model, namely, maximum bond shear stress and interfacial fracture energy.

2. Experimental program

2.1. Experimental specimens and material properties

The main variables considered in this test were the degree of damage of concrete beams, the prestress degree of concrete beams and the preparation method of concrete surface where CFRP sheets would be bonded. A total of five concrete beams (including one RC beam and four partially prestressed RC beams) under different damage levels were strengthened with CFRP sheets. These five beams had been subjected to 2.5 million cycles of constant-amplitude cyclic loading before this test. In the cyclic loading test, the five test beams were was simply supported creating a total span length of 4000 mm, and loaded repeatedly at the mid-span. The loading amplitude of the five test beams were set to be different from each other, which caused different degrees of damage to these beams. The geometric dimensions of these test beams and their reinforcement details are shown in Fig. 1. In Fig. 1, the symbol Φ^F denotes hot-rolled ribbed fine-grained steel bars with yielding strength of 500 MPa (with reliability of 95% confidence interval) and average elastic modulus of 200 GPa (with variation coefficient of 0.002) (HRBF500), Φ signifies hot-rolled plain-shaped bars with yielding strength of 235 MPa (with reliability of 95% confidence interval) and average elastic modulus of 210 GPa (with variation coefficient of 0.001) (HPB235), the numbers before Φ^F or Φ indicate the amount of steel bars and the numbers after Φ^F or Φ represents the diameter of steel bars in mm, i.e., 2Φ^F10 means two HRBF500 steel bars with diameter of 10 mm, the numbers after "@" represent the spacing between steel bars in mm, i.e., Φ8@150 means 8-mm-diameter HPB235 steel bars with spacing of 150 mm, 1 × 7Φ^s15.2 indicates seven-wire strands with tensile strength of 1860 MPa, nominal area of 139 mm^2 and elastic modulus of 195 GPa which was used as prestressed tendon in the test beam. The values of yielding strength and average elastic modulus of steel bars presented above are obtained from the manufacturer. After cyclic loading test, the increase in midspan-deflection of specimens 0.62-N-0.57 and 0.38-N-0.57 were obviously greater than the increase in midspan-deflection of the other beams. The cyclic loading tests conducted on these five beams and test results were detailed in Li et al. [33] and Li et al. [34]. The damage parameter (D), which was expressed by the following equation, was chosen to define the degree of damage of each test beam:

$$D = 1 - \frac{B_n}{B_0} \tag{1}$$

where B_0 is the bending stiffness of the beam before cracking when the beam was loaded at the first time, B_n is the stiffness of the beam after suffering cyclic loading, and n (the subscript of B_n) indicates the number of the loading cycles which the beam have suffered. According to the theory of structural mechanics, the value of B_n for each beam, which was simply supported and was loaded at mid-span, can be calculated using the following expression based on the assumption that the beam is under elastic condition:

$$B_n = \frac{Pl_0^3}{48f_n} \tag{2}$$

where P is the concentrated load applied on the beam, l_0 is the span of the beam, f_n is the midspan deflection of the beam after suffering the cyclic loading with the number of loading cycles equal to n (the subscript of f_n). Although the test beam after undergoing cyclic loading was under a post-cracking situation, the decrease in the stiffness of the beam (B_n) calculated by Eq. (2) (which is based on the assumption that the beam is under elastic condition) could also reflect the actual stiffness degeneration of the beam to some extend. The value of B_0 for each beam can be determined as follows:

$$B_0 = \frac{Pl_0^3}{48f_0} \tag{3}$$

where f_0 is the midspan deflection of the beam before cracking when the beam is loaded at the first time. The prestress degree (λ) of each beam can be calculated by the following equation:

$$\lambda = \frac{f_{py}A_p h_p}{f_{py}A_p h_p + f_y A_s h_0} \tag{4}$$

where A_p is the area of prestressed reinforcement, A_s is the area of non-prestressed reinforcement, f_{py} is the design strength of pre-