Numerical Simulation on Fracturing Bond Mechanisms of Different Basalt FRP Bars

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This paper presents experimental results and numerical simulation of direct pull-out tests of basalt fiber reinforced polymer (BFRP) bars embedded in concrete. First, two patches of experimental pull-out tests are briefly described. In the first experimental patch, the influence of surface texture configuration of BFRP bars on the bonding characteristics between BFRP bars and concrete is investigated through direct pull-out tests carried out on concrete cubes reinforced with BFRP bars. Pull-out test on ribbed steel reinforced concrete cube was also carried out for comparison. In the other experimental patch, pull-out tests were carried out on BFRP bars embedded inside pre-drilled holes into heavy concrete blocks. Through these pull-out tests, the efficiency of two different adhesive materials; namely: epoxy putty and polymer cement, and the effect of cross-section diameter of BFRP bars on the BFRP bar-concrete bond mechanism were investigated. Second, a finite element model (FEM) was employed to analyze the interfacial behavior between BFRP bars and the surrounding materials. Through the FEM, the influence of the tested parameters on the characteristics of local bond-slip models of BFRP bars was assessed by considering different material properties as well as different fracturing bond mechanisms. The experimental and numerical results showed that the bonding behavior of BFRP bars-reinforced concrete structures can be improved by treating the surface texture configurations of BFRP bars. In addition, the properties of the adhesive material between BFRP bars and concrete are key factors controlling the bond mechanism of strengthened concrete structures. Moreover, the proposed FEM was found to be capable of simulating the fracturing bond mechanism of BFRP bars.

Key Words: BFRP, bar, bond, finite element modelling, pull-out

1. INTRODUCTION

Recently, structural durability, lightweight requirement, and recoverability after earthquake events have been considered as main issues for safety and sustainability of reinforced concrete (RC) infrastructures¹⁾. Corrosion of steel reinforcement is one of the principal causes of deterioration of concrete elements subjected to aggressive environment²⁾. As a result of steel corrosion, both bond and structural performances of RC members are deteriorated and billions of dollars are spent every year to repair the associated damages. Moreover, the elasto-plastic characteristics of steel reinforcement would cause difficulty in quickly

recovering structural functions after an earthquake³⁾.

To solve the above mentioned problems and hence enhance the performance of RC structures and extend their lives, fiber reinforced polymer (FRP) composites have been widely used in reinforcing and strengthening concrete structures^{4, 5, 6)}. FRP composites like glass FRP (GFRP), carbon FRP (CFRP), and aramid FRP (AFRP) are widely known as high strength, light weight, corrosion and fatigue resistance, ease of construction materials in comparison to the conventional steel materials. Additionally, FRP composites have elastic characteristics. Therefore, by adding FRP bars to RC structures, the resulted composition could ensure a stable post-yield stiffness, which in turn

enhances the strength demand and post-earthquake recoverability of the bridges⁷). Basalt FRP (BFRP) is one of the latest FRP composites that has been developed within the last 20 years and has been proven to show advantageous characteristics in comparison to other composites. For instance, BFRP has a higher strength and modulus, a similar cost, and more chemical stability compared with E-glass FRP and a wider range of working temperatures and much lower cost than CFRP^{1,8}). Due to the above advantages, BFRP has already become an attractive alternative for replacing conventional construction materials and is expected to enhance structural safety and sustainability. However, the fundamental studies and the relevant applications are still limited due to the relatively recent development compared with other FRP composites.

During the last few decades, a wide range of experimental and numerical studies have been carried out to investigate the structural performance of RC structures reinforced or strengthened with FRP bars. The results of a major number of these studies have revealed that the bond between FRP bars and concrete governs the serviceability, ductility, and capacity of concrete structures. Test results of previous studies on the bond performance of the conventional FRP bars showed that the unique characteristics of every FRP material and the wide variety of fiber/resin interfaces made the bond behavior hard to be predicted apart from experimental investigations^{8, 9}). Proper assessment of the parameters that are commonly known to influence bond of FRP bars to concrete is crucial to understand how BFRP bars develop their bond strength. Particularly, it has been reported that bond is strongly dependent on the surface characteristics of the FRP bar and its treatment¹⁰). Furthermore, with the fast spreading of finite element modeling (FEM) of RC structures, an accurate bond-slip modeling of BFRP bars is crucially needed to be predicted to successfully simulate the behavior of RC structures that contain BFRP bars.

The present study is a part of an ongoing large research program aiming at assessing the use of BFRP bars in reinforcing and strengthening concrete structures. This study presents the results of pull-out tests carried out on BFRP bars embedded in concrete and bonded with different adhesive materials. A nonlinear three dimensional (3D) FEM is developed to address the effect of different parameters on the bonding behavior of BFRP bars.

2. EXPERIMENTAL PULL-OUT TESTS

Two patches of experimental tests were carried

out. In the first patch (group 1), the effect of surface texture configurations on the bonding characteristics between BFRP bars and concrete was investigated through direct pull-out tests. In the second patch (group 2 and group 3), pull-out tests on BFRP bars of different diameters embedded into vertical holes drilled in heavy concrete blocks and bonded to concrete using polymer cement and epoxy putty, respectively, were carried out.

(1) Experimental tests of group 1

Pull-out tests on 10-mm-diameter BFRP bars with four different surface conditions were carried out. Designation of these bar samples are B10-S, B10-SH10, B10-SH5, and B10-DH10. Where surface of B10-S was characterized by very small indentations (i.e., factory produced and named in all parts of this study smooth BFRP bar) and the other bars' surfaces were created by helically wrapping single or double, 1-mm-thickness, BFRP strands to the smooth bar as follows: B10-SH10 and B10-SH5 are BFRP bars with surfaces created by helically wrapping single strand around the smooth BFRP bar with pitch of 10 mm and 5 mm, respectively; and B10-DH10 is a BFRP bar with a surface created by helically wrapping double strand around the smooth BFRP bars with 10 mm pitch. Additionally, for comparison, the same pull-out test was carried out on 10-mm-diameter ribbed steel bar (S10). Details of the investigated surface conditions are shown in **Fig.1** and main experimental parameters, including the mechanical material properties of both BFRP and steel bars used, are shown in **Table 1**.

Table 1. Main experimental parameters and results of group 1

Specimen	Type	D (mm)	Surface	E (Gpa)	f_u (Mpa)	P_m (kN)	f_m/f_u (%)
B10-S	BFRP	9.8	S	48	1120	24.2	29
B10-SH10	BFRP	9.8	SH10	48	1120	31.3	37
B10-SH5	BFRP	9.8	SH5	48	1120	36.9	44
B10-DH10	BFRP	9.8	DH10	48	1120	37.1	43
S10	Steel	9.7	ribbed	210	560	37.9	80

Note: D is the bar diameter; E is the modulus of elasticity, f_u is the ultimate tensile strength, P_m is the maximum pull-out force, and f_m is the maximum corresponding axial stress.

Cube molds with 100-mm-edge length were used to prepare the pull-out tests. BFRP and steel bars of 800-mm-total length, and 250-mm-end portions encased by steel pipes (refer as the loaded end in the subsequent sections), were concentrically positioned in the molds prior to concrete casting.

Concrete compressive strength of 35 MPa was determined as the average of three values obtained from compressive tests carried out on standard concrete cylinders.

Fig.1 Surface texture configurations of BFRP and steel bars

Fig.2 Schematic drawing of a typical test specimen of group 1

Pull-out arrangement is shown in **Fig. 2**. Test specimens were positioned in a universal testing machine having a load cell capacity of 500 kN. As shown in **Fig. 2**, direct tensile force is applied to the loaded end of the bar at a rate of 3 kN/min., while the displacement was recorded using a linear variable displacement transducer (LVDT) attached to the free end of the bar. During the test, a data acquisition system was used to record the applied load and the displacement readings.

(2) Experimental tests of groups 2 and 3

Two groups (2 and 3), comprising three specimens each, of pull-out tests were carried out, where epoxy putty and polymer cement were used as filling material, respectively. The used epoxy putty had a tensile strength and modulus of elasticity of 49 MPa and 3.45 GPa, respectively and the used polymer cement gave a compressive strength of 30 MPa, after seven days. Through this test, the modes of failure, the maximum loads, and the degradation behavior after the peak load of strengthened concrete blocks using the two adhesive materials were compared in three different diameters of smooth BFRP bars; namely: 6, 10, and 12 mm.

A set of concrete blocks were used to carry out the pull-out tests, where each concrete block had dimensions of 400 mm height, 300 mm width, and 1800 mm length. Normal strength concrete was used for all concrete blocks, and the average compressive strength of three tested cylinders was 30 MPa after 28 days. After hardening of concrete, three holes of 30 mm diameter and 240 mm length at equal distances were drilled into each concrete block using an electrical hammer drill. After excavation, all holes were cleaned using compressed air and Ethanol. Moreover, for cases of polymer cement as adhesive material (group 3), the perimeter of holes was painted with Primer after ethanol and left for about one week before embedding the polymer cement and BFRP bars.

The BFRP bars were cut into 1000 mm long. Before embedding the BFRP bars inside the holes, one end of the bar sample was embedded in a steel cylinder (with 300 mm long). After words, an expansive grout (cement-based grout) was used to bond the BFRP bars to the steel cylinders. To apply the pull-out forces on the tested bars without any slippage between the bars and the steel cylinders during the tests, it was very necessary to anchor the loaded ends of the bars with sufficient length and by using a strong material. To do so, preliminary tensile tests were carried out on BFRP bars of 12-mm-diameter anchored to 300 mm length inside steel cylinders by the same specified grout. Results of these tests showed sufficient bond between the bars, grout, and cylinders; that is no slippage was observed during the tests up to rupture of the bars. After hardening of the grout, a part of the free end of the considered BFRP bar was embedded in the hole and bonded with the epoxy putty or polymer cement. The pullout tests were performed using the setup shown in **Fig. 3**, where a hydraulic jack was used for pulling the BFRP bars and two displacement transducers (LVDTs) were used to measure the global displacements.

Fig.3 Schematic drawing of a typical specimen of groups 2 and 3

3. FINITE ELEMENT MODELING

The pull-out tests were modeled using ANSYS code version 13¹¹). In addition to predict the response of the concrete blocks reinforced or strengthened with BFRP bars, the FEM was employed to give insight into the effect of the investigated parameters on the bond characteristics of BFRP bars with concrete. In order to achieve this objective, a regression analysis was performed to identify the bond-slip parameters for each specimen by fitting the pull-out force versus displacement recorded in the experimental tests. A 3D FEM for each group was constructed with dimensions similar to the experimental specimens as shown in **Fig. 4**. For specimens in group 1, only a quarter of the specimen was modeled considering two planes of symmetry. For the other two groups, a cylinder core with 300-mm-diameter was considered and again only a quarter of this core was modeled.

For specimens in group 1, concrete cracks as well as crushing at the loaded end of the pull-out specimens concentrated in a small cylindrical portion (i.e., with approximately 40-mm-diameter) around the bars. Therefore, FE model for the specimens in this group consisted of: (1) outer solid concrete elements with elastic material properties, (2) inner solid concrete elements with nonlinear material properties for the inner square concrete core with 50-mm-length, (3) bar elements for BFRP and steel bars, (4) solid elastic steel elements for the steel plate (i.e., located between the frame of the testing machine and the concrete specimens), and (5) interface elements between the bars and nonlinear concrete elements.

Fig.4 FEM of specimens in group a (a) and groups 2 and 3 (b)

During testing the specimens in the other groups (groups 2 and 3), small cracks appeared in a concrete layer with approximately 30-mm-thickness outside the adhesive material. Moreover, a very thin concrete was observed attaching to the pulled epoxy putty after the tests of specimens in group 3. Therefore, elements of specimens in groups 2 and 3 were also divided into five different elements: (1) outer solid elastic concrete elements outside internal nonlinear concrete elements, (2) internal solid nonlinear concrete elements with 50-mm-thickness around the adhesive material, (3) solid elements for the adhesive materials, (4) bar elements for BFRP bars, and (5) interface elements.

In the proposed FE models, element types, material models, and boundary conditions were carefully nominated and employed to simulate the performance of the tested specimens. Details of the FE strategy are as follows:

(1) Element types

From the element library of ANSYS code version 13¹¹), four elements were nominated and employed to simulate the behavior of the pull-out specimens. 3D 8-node solid structural element (SOLID65) was used to model the model the inner nonlinear concrete portion at the pull-out regions and the polymer cement. SOLID65 is used for the 3-D modeling of solids with or without reinforcing bars. The solid is capable of cracking in tension and crushing in compression. In concrete applications, for example, the solid capability of the element may be used to model the concrete while the rebar capability is available for modeling reinforcement behavior. The element is defined by eight nodes having three degrees of freedom at each node: translations in the nodal x, y, and z directions and

isotropic material properties. Up to three different rebar specifications may be defined. The most important aspect of this element is the treatment of nonlinear material properties. The concrete is capable of cracking, crushing, plastic deformation, and creep. The following assumptions should be considered when dealing with SOLID65 element: cracking is permitted in three orthogonal directions at each integration point; If cracking occurs at an integration point, the cracking is modeled through an adjustment of material properties which effectively treats the cracking as a “smeared band” of cracks, rather than discrete cracks; The concrete material is assumed to be initially isotropic; whenever the reinforcement capability of the element is used, the reinforcement is assumed to be “smeared” throughout the element; and in addition to cracking and crushing, the concrete may also undergo plasticity, with the Drucker-Prager failure surface being most commonly used. In this case, the plasticity is done before the cracking and crushing checks.

The outer concrete portion, the epoxy putty, and the steel plate were modeled as elastic materials using a 3D 8-Node structural element (SOLID185) which is defined by eight nodes having three degrees of freedom at each node: translations in the nodal x, y, and z directions and has an excellent ability to model elastic materials.

BFRP and steel bars were represented by 3D 2-node structural bar elements (LINK180). It is noteworthy that this element has three degrees of freedom at each node (translations in the x, y, and z directions) and has capability of plastic deformations.

To simulate the fracturing bond mechanism, the spring element (COMBIN39) of zero length was used to connect the nodes of the two different materials located at the weakest interface plane, based on the observations from the experimental tests. For instance, the weakest planes of specimens in groups 1 and 3 were considered at the bar-concrete and bar-polymer cement interfaces, respectively, while it was considered at the epoxy putty-concrete interface in specimens of group 2. By adopting generalized force-deflection curves, bond-slip relationship of the different cases studied could be obtained, as it will be addressed in detail in the following sections. It is worth to mention here that the concurrent nodes between the two adjacent surfaces representing the weakest interface planes were coupled in both the transverse and lateral directions. Bond between other adjacent surfaces was assumed perfect (i.e., the nodes of the elements were coincided).

(2) Material models

To simulate the nonlinear plastic behavior of concrete and polymer cement, ANSYS requires input data for material properties as follows: (1) elastic modulus (E_c); (2) uniaxial compressive strength (f'_c); (3) uniaxial tensile strength (modulus of rupture, f_t); (3) Poisson's ratio; shear transfer coefficient for open and closed cracks (β_i and β_c); and (4) compressive uniaxial stress-strain relationship for the concrete. In this study, the nonlinear plastic behavior of concrete in compression was defined using the well-known Hognestad model¹²⁾, **Fig. 5.a**, while the concrete tensile stress-strain response was modeled using the ACI model 13), **Fig. 5.b**. In both models, f'_c is the maximum concrete compressive strength and ϵ_{co} is the corresponding axial strain $= 2f'_c/E_c$, and f_t is the ultimate tensile strength $= 0.62(f'_c)^{0.5}$. The concrete poison's ratio was assumed equal to 0.2.

Fig.5 Concrete compressive stress-strain model (a) and tensile stress-strain model (b)

By using the uniaxial compressive strength (f'_c) and the uniaxial tensile strength (f_t), ANSYS adopts the Willam and Warnke model¹⁴⁾ to define the failure of the concrete. In this model, both cracking and crushing failure modes are accounted for. A 3D failure surface for concrete is shown in **Fig. 6**. The most significant nonzero principal stresses are in the X and y directions, represented by σ_{xp} and σ_{yp} , respectively. Three failure surfaces are shown as projections on the σ_{xp} - σ_{yp} plane. The mode of failure is a function of the sign of σ_{zp} (principal stress in the Z direction). For example, if σ_{xp} and σ_{yp} are both negative (compression) and σ_{zp} is slightly positive (tensile), cracking would be predicted in a direction perpendicular to σ_{zp} . However, if σ_{zp} is zero or

slightly negative, the material is assumed to be crushed.

Fig.6 3D failure surface for concrete ¹¹⁾

After cracking, the tension stress of the concrete element is set to zero in the direction normal to the crack plane. The shear transfer coefficient β_t for open cracks and β_c for closed cracks determines the amount of shear transferred across the cracks. The value of these parameters range from 0.0 to 1.0, with 0.0 representing a smooth crack (complete loss of shear transfer) and 1.0 representing a rough crack (no loss of shear transfer). The value of β_t used in many studies of RC structures, however, varied between 0.05 and 0.25¹⁸⁾. In this study, the values of β_t and β_c were assumed based on the work of Kachlakev et al.¹⁸⁾ as 0.2 and 0.9, respectively.

The outer concrete of the specimens was simulated with elastic concrete material properties having the same modulus of elasticity and Poisson's ratio of the concrete of the column.

The nonlinear response of the steel bar was assumed to be bilinear elasto-plastic with a strain-hardening ratio of 0.01¹⁸⁾, see **Fig. 7.a**. Steel Poisson's ratio was specified as 0.3. The FRP bars and epoxy putty were modeled as simplified uniaxial elastic material, **Fig. 7.b**. Finally, the rigid steel plate of loading were simulated with elastic steel material properties having a modulus of elasticity and Poisson's ratio of 200 GPa and 0.3, respectively.

(3) Loading and boundary conditions

Loading for all models was applied at the same locations of the experimental tests, where displacement-controlled load was applied to the loaded end of the tested bars while displacement was recorded at the free and loaded ends of the bars of specimens in group 1 and groups 2 and 3, respectively. Ultimately, symmetry boundary conditions were applied to the planes of symmetry. Details of the 3D FE models are shown in **Fig. 4**.

Fig.7 Bilinear stress-strain model for reinforcing steel (a), and elastic stress-strain model of FRP bars (b)

(4) Bond-slip modeling

To obtain the bond-slip responses for FRP bars in concrete, several experimental and analytical procedures have been reported in the last decades. The well-known BPE bond-slip model¹⁵⁾ for deformed steel bars was adopted by many researchers^{16,17)} to represent the local bond stress-slip relationship of FRP bars. This model consisted of ascending curve up to the peak bond stress, then a plateau over which the peak bond stress is maintained followed by a descending branch with a steady-constant line at the end of the relationship. In this study, the BPE model was employed to investigate the effect of the considered variables on the bond behavior of BFRP bars as a reinforcing and strengthening tool for RC concrete structures.

Fig.8 Local bond-slip model¹⁵⁾

As shown graphically in **Fig. 8**, six parameters may be used to describe the bond-slip law of steel and BFRP bars; namely: τ_1 , S_1 , S_2 , τ_2 , S_3 , and α , where τ_1 is the peak bond stress, S_1 is the corresponding slip, S_2 is the slip at the end of the horizontal plateau zone, τ_2 and S_3 are the residual bond stress and corresponding slip, respectively, and α is a parameter used to define the ascending branch (see **Fig. 8**). Other definitions for the ascending and descending branches are by using the ascending (K_a) and descending (K_d) stiffness, respectively.

The parameters of the bond-slip law were identified by altering their values until a suitable agreement between the experimental and numerical load-displacement curves were achieved.

4. RESULTS AND DISCUSSIONS

(1) Specimens in group 1

Table 1 presents the main data and results of the tested specimens in group 1, and **Fig. 9** shows the load-displacement relations registered during the experimental pull-out tests together with those obtained from the numerical analysis. Where: D is the diameter of steel or BFRP bar, E is the elastic modulus, f_u is the ultimate tensile strength, p_m is the maximum load, and f_m is the maximum corresponding axial stress.

The failure of all specimens was occurred at the bar-concrete interface. Minor cracks formed at the concrete surrounding the bar at early stage of loading. The cracks propagated and distributed to a wider zone with the increase in the applied load and at failure, a small amount of concrete has detached and pulled-out with the bar. The detached concrete in specimens with rough surface textures was clearer than that of specimens with smooth surface texture. Moreover, in specimens with rough surface textures, as the applied load became very close to the peak value, some ribs were broken and caused a decrease in the ascending stiffness up to the peak load, in which a plateau was formed followed by the debonding failure.

It is very clear from **Fig. 9** that the load-displacement relation of all specimens consists of an ascending curve up to the peak load, then a stable zone over which the peak load is approximately maintained followed by a descending branch that is ended by a flouting curve.

By comparing the load-displacement relations of the tested specimens, it could be observed that all specimens shared almost the same stiffness up to approximately 45% of the maximum load of the specimen S10 (with deformed steel bar).

Fig.9 Experimental and numerical P - δ curves of group 1

At this level, the stiffness of the specimen B10-S (with smooth BFRP bar) became smaller while the other specimens continued with similar stiffness as that of S10 up to approximately 85% of the maximum load.

The maximum attainable loads of specimens B10-S, B10-SH10, B10-SH5, and B10-DH10 were approximately 62%, 82%, 96%, and 97% of that of specimen S10, respectively. It is also clear that the degradation in the strength of the specimen B10-S after the peak load is more stable than that of other BFRP specimens with rough surface textures and the degradation of specimens with BFRP bars has a much stability than that of the specimen with deformed steel bar.

The values of the bond-slip parameters obtained from the inverse analysis using the FEM for each experimental curve are listed in **Table 2**. Moreover, a comparison between the obtained bond-slip curves of all specimens is shown in **Fig. 10**. It is clear from this figure that the proposed bond-slip law could be used successfully to simulate the global behavior of all tested specimens with various surface textures of BFRP bars. Furthermore, the main trim of the bond-slip parameters, **Table 2**, agrees well with the experimental observations.

As it is evident in both **Table 2** and **Fig. 10**, the bond strength (τ_m) and the ascending stiffness (K_a) of the smooth BFRP bar (B10-S) was approximately 60% and 35% of those of the ribbed steel bar (S10), respectively. By adding a single spiral winding with 10-mm-spacing to the surface of the smooth bar (B10-SH10), the values of τ_m and K_a increased to 78% and 55% of those of the ribbed steel bar, respectively. While by adding a single spiral winding with closer spacing (B10-SH5) or by adding double crossing winding (B10-DH10), the average values of τ_m and K_a became 94% and 90% of those of the ribbed steel bar, respectively. Additionally, the stability at the peak strength, that is expressed by the ratio S_2/S_1 , and works as a ductility and fracture energy index for the bond mechanism of the BFRP bars had values greater than that of the steel bar. Similarly, the slope of the descending branch of the bond strength-slip curve of all specimens with BFRP bars was much less than that of the steel bar, and this slope increased when adding roughening to the bar surface.

Table 2. Bond-slip model parameters of specimens in group 1

Specimen	τ_m (Mpa)	K_a (N/mm ³)	S_2/S_1	K_d (N/mm ³)	τ_2/τ_1	α
B10-S	7.2	6.5	2.0	0.60	0.50	0.5
B10-SH10	10.9	6.4	1.7	0.74	0.55	0.5
B10-SH5	11.2	12.4	4.0	0.78	0.50	0.5
B10-DH10	12.2	5.5	1.6	0.85	0.5	0.5
S10	12.1	12.1	2.0	2.05	0.3	0.4

Fig.10 Numerical τ - S curves of specimens in group 1

(2) Specimens in groups 2 and 3

Table 3 presents the experimental parameters and results of the specimens in groups 2 and 3.

Table 3. Main experimental parameters of groups 2 and 3

Specimen	Adhesive	D (mm)	E (Gpa)	f_u (Mpa)	P_m (kN)	f_m/f_u (%)	Failure mode
E-B6	Epoxy	5.9	47	1030	25.6	95	R
E-B10	putty	9.8	48	1113	73.1	88	R&EC
E-B12		11.7	49	1115	92.6	74	EC
P-B6	Polymer	5.9	47	1030	14.3	56	BP
P-B10	cement	9.8	48	1113	22.9	28	BP
P-B12		11.7	49	1115	29.1	24	BP

Note: BP is bond failure at bar-cement past interface, EC is bond failure at putty-concrete interface, R is rupture failure of BFRP bar, and R-EC is a combination of partial rupture and bond failure.

During the experimental tests, full rupture of the BFRP bars with the smallest diameter and bonded using epoxy putty (E-B6) led to a sudden drop of the axial load. With increasing the bar diameter to 10 mm (E-B10), the epoxy putty-concrete interface became weaker, in which the failure mode of this sample was characterized by partial rupture of BFRP bar followed by failure of the putty-concrete interface. With additional increase in the bar diameter (E-B12), the failure mode was characterized by a complete breakage of the bond surface between concrete and the epoxy putty, which resulted in a brittle pull-out failure. It was also observed that a thin layer of the concrete attached to the epoxy putty was crushed and pulled-out with the putty. On the other hand, the failure mode of the pull-out tests using polymer cement as a filling material (group 3) was characterized by cracks in the cement mortar and breakage at the BFRP bar-mortar interface. Experimental and numerical load-displacement relations of the specimens in groups E and P are shown in **Figs. 11** and **12**, respectively.

Fig.11 Experimental and numerical P - δ curves of group 2

Another main aspect of this test is that, as shown in Fig. 13, the ultimate loads for specimens with polymer cement as a filling material were 45 to 70% less than their counterparts with epoxy putty. Moreover, a very stable and gradual decrease in the strength after reaching the ultimate load was clearly observed in specimens with polymer cement compared with a sudden decrease in the specimens with epoxy putty.

Table 4. Parameters of τ - S models of groups 2 and 3

Specimen	τ_m (Mpa)	K_a (N/mm ³)	r	K_d (N/mm ³)	τ_r/τ_m	α
E-B10	11.8	1.2	1.2	-----	-----	0.9
E-B12	10.5	0.9	1.2	-----	-----	0.9
P-B6	2.8	0.7	1.2	0.07	0.5	0.7
P-B10	3.0	0.6	1.2	0.13	0.3	0.7
P-B12	2.7	0.4	1.2	0.09	0.6	0.7

The results of the experimental tests was confirmed by that of the numerical modeling, as presented in **Table 4** and **Figs. 11, 12, and 13**. The

values of the bond strengths and ascending stiffness of BFRP bars bonded by polymer cement (group 3) was much less than the corresponding values of BFRP bars bonded by epoxy putty (group 2). Furthermore, the post-peak behavior of the bond-slip curves of specimens in group 3 was characterized by a very stable descending stiffness, while it was characterized by a sudden drop in the case of specimens in group 2.

Fig.12 Experimental and numerical P - δ curves of group P

Fig.13 Comparison between P - δ curves of groups E and P

Although the experimental results showed that as the diameter of BFRP bar increases the maximum load strength increases, the numerical modeling show that by changing the diameter of BFRP bar, only slight differences in the bond strength were resulted.

5. CONCLUSION

The experimental investigations as well as the numerical analysis of this study showed that the configuration of surface textures of BFRP bars has a significant effect on the ascending stiffness, peak bond strength, stability at the peak strength, and the descending branch of the bond-slip behavior of BFRP bars. By enhancing the surface of BFRP bars, the bond strength increased significantly and became comparable to that of deformed steel bars and the descending curve becomes sharper than that of the smooth bars. The mechanical properties of the adhesive material that used to bond BFRP bars to concrete control the bonding mechanism of BFRP bars. Bonding BFRP bars to concrete by epoxy putty is characterized by high bond strength followed by rupture of BFRP bar or brittle failure at the epoxy-concrete interface. By using polymer cement, the behavior was characterized by low peak strength followed by ductile bond failure at the bar-cement interface. On the other hand, by employing a 3-D FE modeling, it was possible to identify the bonding behavior of BFRP bar to concrete.

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