Experimental Study on Flexural Capacity of Corroded RC Slabs Reinforced with Basalt Fiber Textile

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Abstract: This experimental study investigated the flexural performance of corroded reinforced concrete (RC) slabs strengthened with basalt textile-reinforced mortar (BTRM) and basalt fiber-reinforced polymers (BFRP). Ten RC slabs were designed to achieve the expected corrosion levels (8% mass loss for moderate corrosion and 16% mass loss for severe corrosion) by accelerated corrosion methods. Two slabs served as reference specimens, and eight slabs were strengthened with BFRP or BTRM. The specimens were loaded to failure by the four-point bending method. The corrosion ratio, strengthening materials and the number of layers were tested for comparison. The failure modes, flexural capacities, load–deflection curves and deformation performances of the slabs were obtained from experiments. It was found that the use of BTRM layers was more effective in improving the flexural response than the use of the same amount of BFRP layers externally bonded with the corroded RC slabs under a state of serviceability. The results also showed that the strengthening effects of BFRP and BTRM were affected by the initial corrosion ratio and the number of textile layers. In a moderate state of corrosion, the flexural capacities and deflection capacities of RC slabs strengthened by BFRP and BTRM were increased substantially; the flexural capacities were increased by 27.81%–61.85%. In a severe corrosion state, the increase in flexural capacity of strengthened slabs is marginal but the increase in ductility indexes was 18% to 35% compared with the corresponding control slabs. By increasing the number of textile layers from three to five, the increments of the flexural capacity of strengthened slabs are almost doubled. Finally, the calculated results of the flexural capacity of the corroded RC slabs strengthened with BFRP and BTRM were found to be in good agreement with the experimental results.

Keywords: reinforced concrete (RC) slabs; accelerated corrosion method; basalt textile-reinforced mortar (BTRM); basalt fiber-reinforced polymers (BFRP); flexural capacity

1. Introduction

Reinforced concrete (RC) structures are continuously exposed to the deleterious effects of environmental attacks, which lead to the corrosion of the reinforcing bars. This is especially so in harsh environments involving high chlorine ion conditions as well as those where deicing salt aggravates the corrosion of reinforcing bars in RC structures. At present, many RC structures experience unacceptable loss in serviceability or safety much earlier than anticipated [1]. In recent years, the research and application of retrofitting and rehabilitating concrete structures with carbon fiber-reinforced polymer (CFRP) has developed incrementally and rapidly [2,3]. A number of studies have shown that the application of CFRP as an external strengthening system would effectively increase the ultimate loading carrying capacity and flexural stiffness of corroded RC beams [4–9], but the shortcoming lies in the significant reduction in the deflection capacity of beams and...
their ductility [10]. Meanwhile, some problems have emerged with the usage of externally bonded CFRP composites, including high cost, poor high-temperature resistance, no waterproofing of surfaces, incompatibility of epoxy resins and cement–substrate materials, and environmental hazards [11–16].

Numerous studies had demonstrated that the performance of externally bonded strengthened structures strongly rely on the textile and matrix type. Compared with CFRP, basalt fiber-reinforced polymer (BFRP) provides a cheaper solution and higher capability to strengthen the structural elements similarly to CFRP [17,18]. Although the improvement of the load capacity of the flexural RC member reinforced with BFRP is slightly lower than that reinforced with CFRP [19,20], the ductility of the flexural RC member reinforced with BFRP is better [21]. Wang et al. [22–24] concluded that the load capacity of a corroded RC beam could be effectively improved by strengthening the beam with BFRP, while the extent of improvement was associated with the fiber layers and the initial corrosion state of the beam. An experimental program carried out by Zhu et al. [25] focused on the shear performance and failure pattern of a corroded RC beam reinforced with BFRP and produced an accurate prediction model for shear capacity estimation of these strengthened beams. Qin et al. [26] investigated the flexural and shear performance of damaged RC beams that had been confined with a BFRP sheet and found a good correlation between experimental and analytical data.

In recent years, a new composite material named textile-reinforced mortar (TRM) or fabric-reinforced cementitious matrix (FRCM) has been widely applied in structural retrofitting [27]. TRM or FRCM consist of high-strength fiber in form of textiles embedded into inorganic materials such as cement-based mortars or fine concrete. This kind of material has advantages over fiber-reinforced polymers, namely, high compatibility between the strengthening layer and the concrete substrates, good durability, resistance at high temperatures, good crack resistance, and ease of multi-layer arrangements. It can be used to repair defects and cracks on the surface of RC structures and reinforce the damaged member, even in harsh environments [28,29]. Many studies on the flexural performance of RC beams and slabs strengthened with TRM and FRCM have been reported, which show that various textile/fabric-reinforced cementitious materials would be a promising alternative method to FRP in retrofitting structures [30–33]. Escrig et al. [30] compared the mechanical behavior of several types of FRCM for flexural strengthening of RC beams. The results showed that FRCM contributed to an increment in flexural capacity and flexural stiffness of the strengthened beams. Loreto et al. [31] discussed the performance of RC slab-type elements strengthened with FRCM and demonstrated the technical viability of this composite material system for strengthened flexural RC members. Gopinath et al. [32] studied the effectiveness of basalt textiles as reinforcement for strengthening RC beams under monotonic and low-cycle fatigue loads. Lampros et al. [33] tested the flexural behavior of TRM-reinforced two-way RC slabs through the design of reinforcement methods, fiber types and layers, initial damage of test pieces, and other impact factors. It showed that TRM substantially increases pre-cracking stiffness, cracking load, post-cracking stiffness, and eventually the flexural capacity of two-way RC slabs. In a comparison between the performance of TRM and FRP strengthened RC beams, Raoof et al. [11] and Elsanadedy et al. [34] concluded that TRM retrofitting was slightly less effective in terms of enhancing the flexural strength of RC beams but more effective in terms of deflection ductility.

On the other hand, the feasibility of using textile/fabric-reinforced cementitious materials to strengthen corroded RC structures have received less attention. To the best of the authors’ knowledge, only a few studies, either by EI-Maaddawy et al. [35] or by Elghazy et al. [36–38] have documented the performance of using FRCM systems to restore the flexural capacity and deformation ability of corroded beams. EI-Maaddawy et al. [35] compared the flexural performance of a corroded T-beams, which was repaired with carbon and basalt-FRCM systems. It was concluded that the basalt-FRCM system could not restore the original flexural capacity of the uncorroded beam, whereas the carbon-FRCM system restored 109% of the capacity. In a series of studies by Elghazy et al. [36–38], corroded RC
beams were repaired with CFRP, carbon-FRCM, and poly paraphenylene benzobisoxazole (PBO)-FRCM. Test results showed that both carbon- and PBO-FRCM would effectively restore the original flexural capacity of the uncorroded beam. Furthermore, the PBO-FRCM-repaired specimen showed higher ultimate load-carrying capacities than those of CFRP and carbon-FRCM-repaired specimens.

In general, textile-reinforced composites are emerging as strengthening and repair materials for concrete structures, mainly due to their stability in harsh environments and their relatively low cost. However, existing studies do not adequately cover the subject of applying basalt textile reinforce materials in strengthening corroded RC members. In addition, few data are available in the literature from comparative research into the effectiveness of the two strengthening composites, basalt textile-reinforced mortar (BTRM) and BFRP, for enhancing the flexural capacity of corroded RC members. The focus of this paper is on an exploratory study to evaluate the flexural behavior of corroded RC slabs strengthened with different types of BTRM and BFRP composites. To the best of the authors’ knowledge, it is the first attempt to use basalt textile in combination with two kinds of matrix materials to repair corroded RC slabs, especially when the members are at a high level of corrosion damage. The study comprised an experimental program where 10 corroded RC slabs were constructed and then damaged by accelerated corrosion. A kind of basalt textile with mesh was selected in combination with different matrix materials to strengthen slabs with different degrees of corrosion. All specimens were tested to failure under a four-point load configuration, and the effectiveness of the two strengthening composites in enhancing the flexural performance of the corroded RC slabs were investigated. The test parameters included the initial corrosion situation, the strengthening systems (TRM and FRP), and the number of strengthening layers. Finally, a calculation method is proposed to determine the flexural capacity of strengthened slabs in which the effect of corrosion is considered.

2. Experimental Program
2.1. Test Specimens

To investigate the flexural performance of the corroded slabs strengthened by BTRM and BFRP, 10 RC slabs were prefabricated in the laboratory. The specimens were designed according to the Code for design of concrete structures (GB50010-2010) [39], with a length of 2300 mm and a rectangular cross-section of $b \times h = 700 \text{ mm} \times 100 \text{ mm}$. The concrete cover thickness at the bottom was 30 mm. For each RC slab, 5 longitudinal HRB400 bars (diameter = 12 mm, and cross-sectional area = $113 \text{ mm}^2$) were used for tensile reinforcing ($\rho_s = 1.26\%$), and 12 transverse HRB400 bars (diameter = 6 mm, and cross-sectional area = $28.3 \text{ mm}^2$) were used for the distributed reinforcement. The tensile reinforcing bars extended out of the ends of the slab to connect the wires and the surfaces of the distributed bars were wrapped up in an insulating coating to prevent rusting. The detailed dimensions of the specimens are shown in Figure 1. To depassivate the tensile reinforcing bars, the concrete was mixed with 4% NaCl by weight of cement. The measured cubic compressive strength of C40 concrete was 43.5 MPa, and the yield and tensile strengths of the reinforcing bars were 446.3 MPa and 594.9 MPa, respectively.
2.2. Accelerated Corrosion Scheme

In order to complete the test within a reasonable time frame, the accelerated corrosion approach was adopted by the application of direct electric current and the addition of chloride to the concrete mix while keeping the specimens exposed to moisture. The tensile reinforcing bars in the slabs were connected in series and wired with the positive electrode to the power supply, and stainless-steel rods were wired with the corresponding negative electrode, as shown in Figure 2. During the test, a moist sponge was attached to the surface of the test specimen, and salt water was sprayed on it to ensure the electrochemical reaction could be maintained during the global pre-processing stage. The specimens were stacked vertically before testing and the wood blocks were used to electrically insulate the slab from the earth and other specimens, as shown in Figure 3. The electric current that corresponded to an electric current density of $(0.1 \pm 0.02) \text{ mA/cm}^2$ was impressed on the reinforcing bars. The rate of mass loss of the reinforcing bars was defined as the corrosion ratio, which could be estimated by Faraday’s law as shown in Equation (1). Two expected corrosion states were set in the test: the moderate corrosion state (expected corrosion ratio was about 8%), and the severe corrosion state (expected corrosion ratio was about 16%).

$$m = \frac{Ita}{nF},$$  

where $m$ is the mass loss; $I$ is the corrosion current; $t$ is the time of the corrosion process; $a$ is the atomic mass of iron (56 g); $n$ is the valence of iron atom, which is the number of electrons transferred during the corrosion reaction (2 in this case); and $F$ is the Faraday’s constant (96,500 C/mol).
Figure 3. Details of the accelerated corrosion process: (a) slabs wired to power supply; (b) stainless steel rods; (c) corroded specimens.

2.3. Strengthening Material Properties

Basalt textile (coated) was used as external reinforcement as shown in Figure 4. The textiles were made of fiber rovings distributed equally in two orthogonal directions, where the size of the mesh grid was 5 mm × 5 mm. The mechanical properties of basalt textile were measured by the tensile test according to Test Method for Tensile Properties of Orientation Fiber Reinforced Polymer Matrix Composite Materials (GB/T 3354-2014) [40]. The results are shown in Table 1.

Table 1. Mechanical properties of the test basalt textile.

| Textile Weight Per Area (g/m²) | Cross Section Area (mm²) | No. of Yarns | Elasticity Modulus (GPa) | Tensile Strength (MPa) | Elongation (%) |
|-------------------------------|--------------------------|-------------|-------------------------|------------------------|---------------|
| Basalt 221                   | 0.201                    | 12          | 35.1 (0.068) *          | 725.95 (0.062)         | 2.07 (0.055)  |

Note: * coefficient of variation.

The binder materials for BFRP and BTRM were JN-C3P carbon fiber adhesive and HPM-WA (M55) high-performance composited mortar, respectively. The mechanical properties of binder materials were obtained according to Technical code for safety appraisal of engineering structural strengthening materials (GB50728-2011) [41]. The tensile shear strength of the JN-C3P carbon fiber adhesive was 19.2 MPa, the elasticity modulus was 3.5 GPa, and the tested elongation ratio was 3.0%. The measured compressive strengthen of HPM-WA (M55) composited mortar was 41.6 MPa in 7 days and 58.3 MPa in 28 days, respectively.

By referring to Fiber-reinforced plastics composites determination of tensile properties (GB/T1447-2005) [42], the tensile test of BFRP and BTRM was conducted. The test coupons of BFRP and BTRM were designed as a rectangular shape according to the requirements of GB/T1447 [42], as shown in Figure 5. Three identical coupons were tested for each type of composite. A monotonic load was applied under displacement control at a rate of 2 mm/min, and the coupons failed due to fiber fracture. The results of the tensile test are shown in Table 2. It should be noted that the whole cross-sectional area of the specimens was used for calculating the mechanical properties. That was because the practical flexural test would subject the overall cross-section of BFRP or BTRM composite to tension.

Figure 4. Details of the basalt textile.
Table 1. Mechanical properties of the test basalt textile.

| Textile | Weight Per Area (g/m²) | Cross Section Area (mm²) | No. of Yarns | Elasticity Modulus (GPa) | Tensile Strength (MPa) | Elongation (%) |
|---------|------------------------|--------------------------|--------------|-------------------------|-----------------------|----------------|
| Basalt  | 221                    | 0.201                    | 12           | 35.1 (0.068) *          | 725.95 (0.062)        | 2.07 (0.055)   |

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Table 2. Mechanical properties of BFRP and BTRM materials.

| Types | Layers (n) | Nominal Thickness * (mm) | Tensile Strength (MPa) | Elongation Ratio (%) | Elasticity Modulus (GPa) |
|-------|------------|--------------------------|------------------------|----------------------|------------------------|
| BFRP  | 3          | 1.071 (0.043) **         | 147.53 (0.062)         | 2.87 (0.053)         | 4.964 (0.063)          |
| BFRP  | 5          | 1.785 (0.047)            | 153.10 (0.069)         | 3.00 (0.048)         | 5.032 (0.059)          |
| BTRM  | 3          | 10.000                   | 17.97 (0.071)          | 2.21 (0.071)         | 0.798 (0.070)          |
| BTRM  | 5          | 10.000                   | 23.61 (0.066)          | 2.45 (0.064)         | 0.918 (0.076)          |

Note: * The nominal thickness of BFRP was the mean value of specimen by measurement; the nominal thickness of BTRM was a 10 mm design thickness of the specimen. ** coefficient of variation.

2.4. Strengthening Scheme

During the test, the strengthening materials (BFRP or BTRM) were externally bonded to the bottom of the corroded RC slabs in a range of 2000 mm as shown in Figure 6. It should be emphasized that the strengthening layers did not extend into the support range and at the end of the strengthening layer there was no other special anchoring treatment. The different flexural performance was investigated by changing the experimental variables including the initial corrosion state, the strengthening materials (BFRP or BTRM) and the layers of basalt textile. Details of the specimens are shown in Table 3. The first letter in the specimen name (M or S) represents the specimen with a moderate or severe corrosion state, respectively. The 2nd letter denotes the strengthening schemes. (C) represents non-strengthened slabs which served as control specimens, and letter (P or M) represents strengthened with BFRP or strengthened with BTRM, respectively. The last Arabic number (3 or 5) represents 3 or 5 layers of textile, respectively. For example, ‘M-P-3’ denotes that a moderately corroded RC slab was strengthened by 3 layers of BFRP.
The wet layup application was adopted, and the strengthening procedures included the following steps.

1. The concrete surface was prepared as follows: firstly, marking out the areas where the BFRP or BTRM should be bonded; then, roughening the surface concrete with a bush hammer for to a depth of approximately 3 mm; and finally cleaning the dust with compressed air.

2. The procedure for BFRP-strengthened specimens included: brushing textile with adhesive layer by layer and then compacting the layers with a plastic roller. To satisfy the adhesive soaking sufficiently and remove the excess polymer, each layer of the composite material was rolled 5 times by using a plastic roller.
The procedure for BTRM-strengthened specimens included: (a) wetting the concrete surface with water; (b) paving mortar and textile layer by layer to ensure good impregnation of mortar for the textile; and controlling overall thickness of BTRM by 10 mm. In detail, the first and last layers of BTRM were mortar layers, the thickness of each being 2 mm. The thickness of intermediate layers was uniform and related to the number of textile layers. For example, if the BTRM contained 3 textile layers, the thickness of each intermediate layer would be 2 mm.

2.5. Test Setup and Instrumentation

The flexural capacity test was conducted in the Structural Laboratory of Hunan Agricultural University. The four-point loading scheme was conducted based on the suggestions of Standard for test method of concrete structures [43], where the corresponding parameters for the applied loads, displacement responses, strain responses and the crack and failure patterns were observed and recorded during the test.

The test instrumentations are shown in Figure 7. To measure the strain response at the bottom of the test slab in the mid-span, three strain gauges (defined as S1 to S3) were arranged on the surface of the strengthening layer, two gauges (defined as S4 to S5) were arranged at the concrete bottom, and three gauges (defined as S6 to S8) were arranged at the side surface of the concrete. Meanwhile, six linear variable differential transformers (LVDTs) (defined as D3 to D8) were installed at the bottom of the slab, and another two dial gauges (defined as D1 to D2) were utilized to measure the boundary displacement at the specimen supports.

Figure 6. Details of the test specimen with strengthening materials: (a) strengthening scheme (all dimensions in mm); (b) BFRP-strengthened specimen; (c) BTRM-strengthened specimen.
Table 3. Details of the test strengthened specimens.

| Specimen ID | Initial Corrosion State (Expected Corrosion Ratio) (%) | Number of Textile Layers | Strengthening Material |
|-------------|--------------------------------------------------------|--------------------------|------------------------|
| M-C         | 8                                                      | –                        | –                      |
| M-P-3       | 8                                                      | 3                        | BFRP                   |
| M-P-5       | 8                                                      | 5                        | BFRP                   |
| M-M-3       | 8                                                      | 3                        | BTRM                   |
| M-M-5       | 8                                                      | 5                        | BTRM                   |
| S-C         | 16                                                     | –                        | –                      |
| S-P-3       | 16                                                     | 3                        | BFRP                   |
| S-P-5       | 16                                                     | 5                        | BFRP                   |
| S-M-3       | 16                                                     | 3                        | BTRM                   |
| S-M-5       | 16                                                     | 5                        | BTRM                   |

Figure 7. Test setup and instrumentation: (a) loading scheme; (b) test points arrangement; (c) details of loading test.
3. Experimental Results and Discussion

3.1. Accelerated Corrosion Results

Corrosion crack widths were measured at 100 mm intervals along the length of the zone (the middle 2000 mm of the slab) by using a microscope, and the maximum crack width of the slabs were recorded and the mean values calculated. It can be seen that both maximum and mean values of crack widths basically increased with the corrosion ratio. Figure 8 presents the detailed corrosion cracks for all slabs before testing. Longitudinal cracks were located at the bottom side of all slabs along the direction of tensile reinforcements. Generally, the widths of the longitudinal cracks decreased from the middle of the span to the two support ends. A few corrosion cracks were noted on the N and S lateral sides of the slabs. After the flexural test, the corroded tensile reinforcements were extracted from the slabs and cleaned with diluted hydrochloric acid and a wire brush. After weighing, the mass loss of bars was calculated. The results of the accelerated corrosion test are shown in Table 4.

[Figure 8. Corrosion cracks and flexural cracks of the slabs.]
Corrosion Ratio * is an average steel mass loss of five longitudinal steel bars in a slab.

3.2. Load–Deflection Curves

The load was applied incrementally at a rate of 2.5 kN per step until the tensile reinforcement yielded. The loading process was then controlled by deflection, and the increment was 2.5 mm per step. The load versus mid-span deflection is shown in Figure 9. The control specimens M-C and S-C presented the typical flexural failure by yielding of the tensile reinforcing bars, followed by concrete crushing in the compression zone. However, the bearing capacity and ultimate elongation of specimen S-C were significantly reduced due to severe corrosion.

![Figure 9](image-url) Load–deflection relationship of test specimens: (a) moderate corrosion; (b) severe corrosion.

The differences between the curves of the strengthened slabs and the control slabs are attributed to the contribution of strengthening materials to the flexural performance of the slabs (refer to Figure 9). Before reaching the load peak, the load–deflection curves of the strengthened slabs increased monotonously. At this stage, the curves consisted of three segments with two turning points, indicating the initial cracking in concrete and the yielding of the tensile reinforcement. When the peak point of the curves was reached, a sudden drop of load occurred due to the strengthening layers fracturing or debonding near the mid-span. At the post-peak stage, the loading capacity of the strengthened slabs dropped to the value which was higher than or close to that of the control specimens. The major effect of the strengthening layers had almost been lost at this stage, and the load remained almost constant due to the tensile reinforcing bars yielding. The tests were terminated when the compressive concrete zone was crushed, which indicated that the slabs had reached their ultimate limit state. At the post-peak stage, most of the strengthened specimens underwent large deformation before failing. However, the deformation stage of M-P-5 was extremely short because the slab failed shortly after it reached the peak load.
3.3. Flexural Capacity Analysis

Table 5 summarizes the loads obtained in the experiments and contains: yield load ($F_Y$) and ultimate load ($F_u$). The yield load was defined as the load corresponding to the tensile reinforcement yield. The ultimate load was defined as the peak load observed on the load–deflection curve. The flexural capacities ($M_u$) of slabs were obtained by utilizing the ultimate loads.

| Specimen ID | Corrosion Ratio (η (%)) | Yield Load ($F_Y$) (kN) | Percentage Increase of $F_Y$ (%) | Ultimate Load ($F_u$) (kN) | Flexural Capacity ($M_u$) (kN⋅m) | Percentage Increase of $M_u$ (%) |
|-------------|-------------------------|-------------------------|---------------------------------|-----------------------------|----------------------------------|---------------------------------|
| M-C         | 8.84                    | 31.8                    | –                               | 37.4                        | 13.09                            | –                               |
| M-P-3       | 7.68                    | 38.7                    | 21.70                           | 47.8                        | 16.73                            | 27.81                           |
| M-P-5       | 9.55                    | 43.3                    | 36.16                           | 53.8                        | 18.83                            | 43.85                           |
| M-M-3       | 8.37                    | 44.5                    | 39.94                           | 48.2                        | 16.87                            | 28.88                           |
| M-M-5       | 9.14                    | 47.7                    | 50.00                           | 60.5                        | 21.18                            | 61.80                           |
| S-C         | 16.25                   | 24.6                    | –                               | 28.5                        | 9.98                             | –                               |
| S-P-3       | 15.81                   | 25.0                    | 1.63                            | 33.2                        | 11.62                            | 16.43                           |
| S-P-5       | 14.94                   | 29.3                    | 19.11                           | 38.0                        | 13.30                            | 33.27                           |
| S-M-3       | 16.43                   | 25.7                    | 4.47                            | 31.8                        | 11.13                            | 11.52                           |
| S-M-5       | 14.27                   | 28.2                    | 14.63                           | 35.8                        | 12.53                            | 25.55                           |

The yields and ultimate loads of all strengthened slabs were generally higher than that of the control slabs. Under the moderate corrosion condition, the yield load of the strengthened slabs increased by 21.70%~50.0% and the flexural capacities increased by 27.81%~61.80% compared to that of the corresponding control slabs. Under the severe corrosion condition, these yield loads increased by 1.63%~19.11% and flexural capacities increased by 11.52%~33.27%.

Figure 10 illustrates the comparison of normalized flexural capacity of strengthened slabs in different corrosion states. It can be seen that the flexural capacity of the BTRM-strengthened slabs was higher than that of the BFRP-strengthened specimens under the moderate corrosion condition. The highest flexural capacity, namely 21.18 kN⋅m, was obtained from specimen M-M-5, which was strengthened by BTRM composites. Specimen M-M-5 was recorded as having a 12.48% higher flexural capacity increment when compared with that of specimen M-P-5. This abnormal phenomenon is likely due to the randomness of the sample. The strengthening technology and quality of M-M-5 was shown to be superior to the other specimens, while its curing time was longest because it was the last one to be tested. When the slabs were severely corroded, the capacity of the BTRM-strengthened specimens were slightly lower than that of the BFRP-strengthened specimens.

![Figure 10. Comparison of normalized flexural capacity of strengthened slabs.](image-url)
The initial corrosion ratio could significantly affect the flexural capacities of specimens. According to the test result in reference [44], the flexural capacity of uncorroded RC slab with the same parameters was 20.60 kN·m. When the initial corrosion ratio increased from 0% to 8.84% and finally to 16.25%, the flexural capacities of the control slabs dropped by 36.45% and 51.55%, respectively. From Figure 11, it can be concluded that the flexural capacities of strengthened specimens also decreased with the initial corrosion ratio increasing, and the rate of decrease was 0.8723.

Figure 11. Relationship between flexural capacity and corrosion ratio.

The influence of the number of strengthening layers on the slabs’ flexural capacity is presented in Figure 12. When the number of strengthening layers was increased from 0 to 3 and finally to 5, the normalized flexural capacities increased by 29% and 62% in the moderate corrosion state, respectively; these capacities increased by 12% and 26%, respectively, in severe corrosion state. The corresponding enhancement in the BFRP-strengthened slabs was similar, namely 28%, 44%, 16% and 33% in the two corrosion states, respectively.

Figure 12. Effect of number of strengthening layers (a) moderate corrosion; (b) severe corrosion.

3.4. Failure Patterns

The control slabs exhibited a typical flexural mode of failure that was due to yielding of the tensile reinforcements followed by concrete crushing at the compression zone.

All strengthened slabs failed in flexure after displaying flexural strength considerably higher compared to the corresponding control specimens, but the failure process was different from the control specimens. When the strengthened specimens were maintained within the elastic stage, the load–deflection relationship was linearly developed. Thereafter, the minor cracks propagated, increased and coalesced with the increase in the applied loads. Meanwhile, the fibers fractured gradually with the sound of cracking followed
by yielding of the tensile reinforcements. Subsequently, there was a loud snap and the strengthened layers damaged integrally, leading to a sudden load drop from the peak point. At the post-peak stage, the load almost remained constant while the width of flexural cracks and deflection of the slabs grew rapidly. Finally, the concrete in the compression zone was crushed and the specimens failed. The only exception was specimen M-P-5, which hardly had the post-peak stage because the strengthening layers and compressive concrete of that slab damaged almost simultaneously.

The details visual mapping of flexural cracks was shown in Figure 8. The flexural crack pattern of BFRP-strengthened slabs was dominated by the main cracks across the bottom surface of the slabs. As for the BTRM-strengthened specimens, a series of transverse cracks were observed during the whole test. In this respect, the BTRM composites seemed to present better performance at distributing stresses during the cracking process than the BFRP composites.

Four damage patterns of strengthening layers were observed in the strengthened slabs as shown in Figure 13. Strengthening layers of M-P-3, S-P-5, and S-M-5 failed due to fiber fracture at the mid-span region of the slabs (Figure 13a). The damage pattern for M-P-5 and S-P-3 was debonding of BFRP from the concrete surface accompanied with peeling off of the concrete cover (Figure 13b). However, the debonding of the BTRM composites of M-M-5 and S-M-3 occurred at the concrete–matrix interface (Figure 13c). In both debonding patterns, BFRP and BTRM composites, debonding initiated from the mid-span region and propagated toward the end of the slab. The BTRM composite of M-M-3 failed due to slippage of the fibers within the matrix accompanied by partial fracture of the fibers, at the maximum flexural moment region as shown in Figure 13d. Therefore, the load drop of M-M-3 was relatively gradual and smooth (refer to Figure 9).

![Figure 13. Damage patterns of specimen-strengthening layers: (a) fracture damage of strengthening layer; (b) debonding of BFRP layer (c) debonding of BTRM layer; (d) slippage damage of BTRM layer.](image-url)
3.5. Bending Deformation Analysis

The cracking loads of the strengthened slabs were approximately between 15 and 20 kN, which was slightly enhanced when compared with the control slabs. This increase indicated some activation of the strengthening layers in tension prior to concrete cracking [14]. The bending stiffness of specimens at different stages (initial, cracking and post-yielding) is shown in Table 6 where the data were calculated from the load deflection curves as the tangent stiffness of different stages. The table also indicated the enhancement of stiffness in cracking and post-yielding stages compared to the corresponding control slabs. It was noticed that the presence of the strengthening layers also increased the cracking and post-yielding stiffness of the corroded RC slabs, especially under the severe corrosion condition. This means that the lower the initial stiffness of the un-strengthened slabs, the higher the effect of strengthening [14].

According to the criterion recorded in Code for design of concrete structures (GB50010-2010) [39], the mid-span deflection of slabs at the serviceability limit state is limited to within \( l_0/200 \) mm when its span \( l_0 \) is less than 6 m. Due to the specimen span being 2.1 m, the limited deflection (\( D_a \)) for 10.5 mm could be calculated, and the service loads (\( F_a \)) could be extracted from the corresponding load–deflection curves.

Table 6. Comparison of stiffness at initial, cracking and post-yielding stage.

| Specimens | Initial Stiffness (kN/mm) | Cracking Stiffness (kN/mm) | Percentage Increase (%) | Post-Yielding Stiffness (kN/mm) | Percentage Increase (%) |
|-----------|---------------------------|-----------------------------|-------------------------|-------------------------------|-------------------------|
| M-C       | 5.39                      | 2.27                        | -                       | 0.73                          | -                       |
| M-P-3     | 7.04                      | 2.40                        | 5.73                    | 0.86                          | 17.81                   |
| M-P-5     | 7.01                      | 2.32                        | 2.20                    | 0.94                          | 28.77                   |
| M-M-3     | 8.46                      | 2.44                        | 7.49                    | 0.79                          | 4.20                    |
| M-M-5     | 8.74                      | 2.67                        | 17.62                   | 1.04                          | 42.47                   |
| S-C       | 5.32                      | 2.07                        | 5.92                    | 0.50                          | -                       |
| S-P-3     | 5.22                      | 2.46                        | 18.84                   | 0.71                          | 20.34                   |
| S-P-5     | 7.15                      | 3.02                        | 33.04                   | 0.83                          | 40.68                   |
| S-M-3     | 5.21                      | 2.34                        | 13.04                   | 0.67                          | 13.56                   |
| S-M-5     | 5.13                      | 2.58                        | 24.63                   | 0.86                          | 45.76                   |

Figure 14 shows that the service load of the specimens is significantly enhanced by strengthening with the BFRP/BTRM layers, and the performance of specimens was also enhanced with the increment of the strengthening layers. The promotion of load capacity was useful in cases where the serviceability limit state of deflection was the governing factor. When the slabs were strengthened by BFRP composite, the service load under the moderate corrosion condition was increased by 0.41% (3 layers) and 4.07% (five layers), respectively, and the enhancement ratio was 2.93% (three layers) and 13.66% (five layers) under the severe corrosion condition. As for the slabs strengthened with BTRM, the service load under the moderate corrosion condition was enhanced by 8.54% (three layers) and 28.05% (five layers), respectively, and enhancement under the severe corrosion condition was 1.95% (three layers) and 7.80% (five layers). It was therefore found that the strengthening effects of BTRM are better than that of BFRP, and the former actually enhanced the section height of the test slab, thereby controlling the deformation of the specimen.
The ductility performance of the specimens was evaluated by calculating their ductility index, which was the ratio of the slab deflection at the ultimate load to the deflection at the yield load [35]. The ductility index was expressed as Equation (2).

\[ k_d = \frac{D_u}{D_y} \]  

(2)

where \( k_d \) is the ductility index; \( D_u \) is the ultimate deflection of the slab; \( D_y \) is the yield deflection of the slab.

The yield and ultimate deflections of all specimens were presented in Table 7. To illustrate the influence of the strengthening composites on the deformation capacity, the normalized ductility indexes of the strengthened slabs are given in Figure 15. Compared with the corresponding control slabs, the ductility indexes of M-P-3, M-P-5, S-P-3 and S-P-5 increased by 1%, 6%, 18% and 19%, respectively; in addition, these indexes of M-M-5, S-M-3 and S-M-5 increased by 3%, 35% and 33%, respectively. Only the ductility index of specimen M-M-3 was 11% lower than that of the control specimen, which could be attributed to the slippage damage pattern of the strengthening layers. Generally, a higher ductility index means a higher ability of the slabs to redistribute moments and to exhibit large overall deformation. It was found that the deformation capacities of corroded RC slabs improved after they were enhanced by bonded BFRP or BTRM systems, especially in a severe corrosion state.

Table 7. The deflections and ductility index of specimens.

| Specimen ID | Corrosion Ratio \( \eta \) (%) | Yield Deflection \( D_y \) (mm) | Ultimate Deflection \( D_u \) (mm) | Ductility Index \( k_d \) | Maximum Deflection in Post-Peak Stage \( D_{\text{max}} \) (mm) |
|-------------|----------------------------|-------------------------------|-------------------------------|----------------|--------------------------|
| M-C         | 8.84                       | 17.751                        | 34.057                        | 1.908          | 34.057                   |
| M-P-3       | 7.68                       | 21.783                        | 41.850                        | 1.921          | 61.624                   |
| M-P-5       | 9.55                       | 25.248                        | 50.931                        | 2.017          | 50.931                   |
| M-M-3       | 8.37                       | 23.719                        | 40.361                        | 1.702          | 57.837                   |
| M-M-5       | 9.14                       | 22.096                        | 42.774                        | 1.973          | 59.771                   |
| S-C         | 16.25                      | 14.954                        | 27.082                        | 1.811          | 27.082                   |
| S-P-3       | 15.81                      | 15.362                        | 32.900                        | 2.141          | 71.730                   |
| S-P-5       | 14.94                      | 16.414                        | 35.268                        | 2.149          | 69.532                   |
| S-M-3       | 16.43                      | 15.306                        | 37.550                        | 2.453          | 62.030                   |
| S-M-5       | 14.27                      | 17.677                        | 42.461                        | 2.402          | 72.997                   |

Figure 15. Deformation capacities of slabs.

Table 7 lists the maximum deflections of the strengthened slabs at the post-peak stage. The control specimens and M-P-5 failed in a brittle manner after reaching the peak load.
The rest of the strengthened slabs sustained the load after reaching the peak point and underwent a large elongation before failure.

3.6. Distribution of Cross-Sectional Strain

The strain distribution along the height of the section is illustrated for representative slabs strengthened with five layers as shown in Figure 16. It can be seen from the figure that the neutral axis of the specimens moved up gradually with the increase in the applied loads, and the height of the compressive zone decreased correspondingly. It was noted that the strains on the cross-section at different heights presented linear variety approximately before specimens reached the peak load under the moderate corrosion condition. Then the strain distribution of slabs under the severe corrosion state showed non-linear characteristics more obviously. That phenomenon could be ascribed to the deterioration of the bond at the steel-to-concrete interface caused by corrosion, and to some extent to the reduction in the cross-sectional area of the tensile reinforcing bars. The strain distribution of the corroded slabs strengthened with three layers was similar to that of slabs strengthened with five layers. It was regarded that the deformation of the specimens was satisfied with the plane section assumption after corrosion and reinforcement.

Figure 16. The strain distribution along the height of section: (a) M-P-5; (b) M-M-5; (c) S-P-5; (d) S-M-5.
4. Flexural Capacity Analysis

4.1. Theory Assumptions

From the test results presented in Section 3, it can be seen that the tensile reinforcement yielded when specimens reached the ultimate load, with subsequent damage to the strengthening materials. In this study, the obvious debonding phenomenon of specimens mainly occurred at the post-peak stage. This indicates that the corroded RC slabs and the strengthening materials. In this study, the obvious debonding phenomenon of specimens yielded when specimens reached the ultimate load, with subsequent damage to the strengthening layers were both satisfied with the plane section assumption. Therefore, the simplified computing model and formula based on that assumption could be obtained as shown in Figure 17 and Equation (3).

![Figure 17. Simplified computation model for structural flexural capacity calculation.](image)

\[
\frac{\epsilon_{c,t}}{\xi h_0} = \frac{\epsilon_s}{(1 - \xi) h_0} = \frac{\epsilon_r}{\xi h_r} \tag{3}
\]

where \(\epsilon_{c,t}\) denotes the compressive strain of concrete at the top of specimen compression zone, \(\epsilon_s\) denotes the tensile strain of steel reinforcements, \(\epsilon_r\) denotes the tensile strain of strengthening layer, \(\xi\) denotes the relative depth of compression zone, \(h_0\) denotes the effective height of test specimen, and \(h_r\) denotes the distance between the strengthening layer center and the compression zone edge.

According to the load–displacement relationship of the test specimens, it can be regarded that after the tensile reinforcing bars yielded at the peak loads, the concrete compression zone was reached at the elastic-plastic stage, and the strain at the top of the compression zone varied within \(\epsilon_0 \leq \epsilon_{c,t} \leq \epsilon_{cu}\). The corresponding stress information can be obtained by using the equations suggested by Wu et al. [45] as follows:

\[
\sigma_c = f_c \tag{4}
\]
\[
\sigma_s = f_y \tag{5}
\]
\[
\sigma_r = E_r \epsilon_r = E_r \frac{1 - \xi}{\xi h_0} \epsilon_{c,t} h_r \tag{6}
\]

where \(\sigma_c\) denotes the concrete compressive stress under compressive strain \(\epsilon_c\), \(f_c\) denotes the axial compressive strength, \(\epsilon_0\) and \(\epsilon_{cu}\) denotes the peak and ultimate compressive strain of concrete, which is generally defined as the values for 0.002 and 0.0033, respectively, \(\sigma_s\) denotes the tensile stress under tensile strain \(\epsilon_s\), \(f_y\) denotes the yield strength of steel reinforcement, and \(\sigma_r\) and \(E_r\) are the tensile stress and elastic modulus of the strengthening layer, respectively.

According to the calculation model (Figure 17), the mechanical equilibrium equation in the horizontal direction and the bending moment equilibrium equation around the strengthening layer center could be obtained by using the equations suggested by Jin et al. [46] as follows:

\[
f_c b_2 h_0 \left(1 - \frac{\epsilon_0}{3 \epsilon_{c,t}}\right) - f_y A_s - E_r \frac{1 - \xi}{\xi h_0} \epsilon_{c,t} h_r A_r = 0 \tag{7}
\]
\[ M_{u,\text{cal}} = f_c b \xi h_0 \left( 1 - \frac{e_0}{3 \varepsilon_{c,t}} \right) \left\{ h - \xi h_0 \left[ 1 - \frac{1}{12} \left( \frac{e_0}{3 \varepsilon_{c,t}} \right)^2 \right] \right\} - f_y A_s a_s \]  

(8)

where \( M_{u,\text{cal}} \) denotes the calculated flexural capacity of test specimen, \( A_s \) denotes the total cross-sectional area of tensile reinforcing bars, \( A_c \) denotes the cross-sectional area of strengthening layers, and \( a \) denotes the distance between the strengthening layer center and the centroid of steel bar cross section. Generally, strain \( \varepsilon_{c,t} \) can be calculated with the measured data based on the plane section assumption, and elasticity modulus \( E_r \) can be measured by the tensile test for BFRP and BTRM, respectively (see Table 2 for the elasticity modulus of BFRP and BTRM composites).

### 4.2. Parameters Calibration

Due to corrosion of the steel reinforcements and the fact that the cracks were investigated at the bottom of the test specimens, the mechanical performance of the tensile reinforcement was reduced. It was therefore essential to calibrate the nominal yield strength of corroded steel bars and the bonding performance of concrete (the nominal yield strength of a corroded steel bar is defined as the tensile yield load divided by the section area which is calculated by the nominal diameter of the bar). According to Jin et al. [46], a certain relationship exists between the nominal yield strength of a corroded bar and the corresponding corrosion ratio, which can be expressed as follows:

\[ \frac{f_{y,c}}{f_{y,n}} = 1 - a \eta \]  

(9)

where \( f_{y,c} \) denotes the nominal yield strength of the corroded steel bar, \( f_{y,n} \) denoted the yield strength of a normal steel bar, \( \eta \) denotes the corrosion ratio of a steel bar, and \( a \) denotes the regression coefficient. In this study, coefficient value \( a \) was adopted as 1.75 through the test described in reference [44]. However, to consider the challenge of describing the bonding performance of concrete to corroded reinforcement, the effective cross-sectional area of the steel bar was selected as a reduction factor in the following equations proposed by Caurns et al. [47]:

\[ A_{k,se} = k_s \beta_s A_s \]  

(10)

\[ k_s = -0.2722 \omega + 1.0438 \]  

(11)

\[ \beta_s = 1 - \eta \]  

(12)

where \( A_{k,se} \) denotes the effective cross-sectional area of corroded steel bars, \( k_s \) denotes the coordination coefficient, \( \beta_s \) denotes the reduction coefficient to cross-sectional area, and \( \omega \) denotes the crack width of test specimens caused by expansion.

Through the above improved Equations (7)–(12), the tested and calculated flexural capacity of the specimens are compared in Table 8. It can be seen that the test results were relatively close to the corresponding calculated results, the mean value of \( M_{u,\text{exp}}/M_{u,\text{cal}} \) was equal to 0.9514, and the corresponding standard deviation was 0.0953.

**Table 8.** Comparison of test and calculated results of specimen flexural capacity.

| Specimen ID | Corrosion Ratio \( \eta \)(%) | \( M_{u,\text{exp}} \)(kN·m) | \( M_{u,\text{cal}} \)(kN·m) | \( M_{u,\text{exp}}/M_{u,\text{cal}} \) |
|-------------|-------------------------------|-----------------|-----------------|-----------------|
| M-J-3       | 7.68                          | 16.73           | 15.76           | 1.062           |
| M-J-5       | 9.55                          | 18.83           | 19.06           | 0.988           |
| M-M-3       | 8.37                          | 16.87           | 18.01           | 0.957           |
| M-M-5       | 9.14                          | 21.18           | 19.02           | 1.114           |
| H-J-3       | 15.81                         | 11.62           | 12.34           | 0.942           |
| H-J-5       | 14.94                         | 13.30           | 14.60           | 0.911           |
| H-M-3       | 16.43                         | 11.13           | 13.70           | 0.812           |
| H-M-5       | 14.27                         | 12.53           | 14.71           | 0.852           |
5. Conclusions

This paper reports the results of an experimental study designed to investigate the feasibility and effectiveness of using externally bonded BFRP and BTRM composites to enhance the flexural performance of RC slabs in different corrosion states. A total of ten slabs were tested under four-point bending until failure. The studied parameters included initial corrosion rate, number of fiber mesh cloth layers, and strengthening materials (BFRP versus BTRM composites). Although the research involved a limited number of specimens, the main conclusion can be drawn as follows.

1) The corroded RC slabs strengthened by BFRP or BTRM showed better flexural performance when compared with the un-strengthened ones in the serviceability limit state. Compared with the control slabs, the service loads of the strengthened corroded RC slabs improved by 0.41% to 28.05% in the serviceability limit state \((D_s = l_0/200 \text{ mm})\). Moreover, the service load of BTRM-strengthened slabs increased more significantly than that of BFRP-strengthened slabs, which shows that BTRM composites are more effective in terms of improving the flexural performance of corroded slabs in the serviceability limit state.

2) Both BFRP and BTRM composites can be fully used to improve the flexural capacity of corroded RC slabs in the ultimate limit state. The improvement effect of flexural capacity by BFRP and BTRM composites has a certain relationship with the initial corrosion ratio. The flexural capacities were increased by 27.81%~61.85% under the moderate corrosion condition and 11.52%~33.27% under the severe corrosion condition, respectively. Under the same strengthening condition, the flexural capacities of moderately corroded RC slabs improved more significantly than that of the severely corroded slabs. The flexural capacity improvement also increased with the number of strengthening layers. By increasing the number of textile layers from three to five, the increments of the flexural capacity of the strengthened slabs almost doubled under different corrosion conditions.

3) The experiments showed that both BFRP and BTRM can improve the deformation performance of corroded RC slabs, especially under severe corrosion conditions. Compared with the control slabs, the ductility indexes of strengthened slabs increased slightly under the moderate corrosion condition, even a little lower than that of the control slabs. However, these indexes increased by 18%~35% under the severe corrosion condition. It was observed from the test that all strengthened slabs, except for M-P-5, undergo a long ductile deformation process at the post-peak stage.

4) Based on the damage characteristics of the specimens and the plane section assumption, this study proposed a scheme for calculating the flexural capacity of strengthened corroded RC slabs, which included consideration of the reduction in tensile and bonding performance of corroded steel bars. By comparison, the calculated flexural capacity values of corroded RC slabs strengthened with BFRP and BTRM are in a good agreement with the test results.

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