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Degradation model of bond performance between deteriorated concrete and corroded deformed steel bars

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HIGHLIGHTS

- Bond behaviour between the deteriorated concrete and corroded deformed steel bars are studied.
- The specimen failure mode is mainly splitting failure, but the mark of steel bars became unclear gradually.
- The relative thickness of protective concrete has great effect on the ultimate bond strength.
- Estimation model of bond strength between deteriorated concrete and corroded deformed steel bar was established.

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ABSTRACT

An experimental investigation was conducted to explore the degradation model of bond stress between concrete and deformed steel bars with four different diameters (12, 16, 20 and 25 mm) to determine the coupling effects on the bond between deteriorated concrete and corroded steel bars. The results indicated that the failure mode of the specimens was primarily splitting failure; however, the marks of the steel bars at the interface between the deteriorated concrete and corroded steel bars became unclear as the corrosion ratio increased. The ultimate bond strength of the two specimens with the smaller diameters increased with deterioration before a crack appeared and then it decreased, whereas that of the two specimens with the larger diameters continuously decreased due to smaller protective concrete thickness. A simplified calculation model of the bond strength between the cracked concrete and heavily corroded deformed steel bars was established based on the bonding theory of concrete and plane steel bars suing the following factors: the corrosion ratio of the steel bars, the compressive strength of concrete, the width of the cracks in the concrete and the friction factor between the deteriorated concrete and corroded steel bars.

1. Introduction

The bond strength between concrete and deformed steel bar depends on three aspects: chemical adhesion, friction and mechanical interlock [1]. Chemical adhesion increases with increasing concrete compressive strength, and friction is determined by the gripping force between concrete and the contact surface area. Mechanical interlock is the primary contributor to the bond strength between concrete and deformed steel bars, and is heavily influenced by the shear strength of the concrete along with the height and pitch of the ribs. Good bond strength is extremely important in the combined action of concrete and steel bars [2–4]. However, because concrete structures are affected by their service environment, concrete deteriorates and steel bars corrode; subsequently, the cracked concrete can even peel off, and the bond strength between the two materials can be eliminated [5–8].

Several studies have found that bond strength increases slightly as steel bar corrode and then decreases rapidly as the expansive cracks widen [9–12]. Therefore, the width of expansive cracks has been used as one parameter to evaluate bond strength [13,14]. However, the deterioration of concrete and corrosion of steel bars always exist together and are intercoupled [6,15]. Hence, the expansive width of cracks on the concrete surface increases linearly with the corrosion ratio of steel bars, and a deviation appears between the predicted and actual outcomes. Several other factors, such as the rate of the protective thickness of concrete (c), steel bar

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diameter (d) and the presence of a stirrup, affect the development of expansive cracks. Therefore, establishing a degradation model for the bond strength between deteriorated concrete and corroded deformed steel bars is complex.

A strain-softening model has often been adopted to establish the bond strength between corroded deformed steel bars and concrete, where the interface concrete is divided into two parts: the cracked inner part and the elastic outer part [8,16]. However, this model is difficult to use because of the unknown parameters. As the bond strength is strongly dependent on the relative rib area, an increase in the bond index from 0.04 to 0.10 increases the bond strength by up to 40% [17]. When deformed steel bars corrode and concrete heavily deteriorates, rust fills the inner cracks, and the rib height decreases. The bond performance between them is similar to that between plane steel bars and concrete, where friction is the primary factor [18], which could potentially be used to develop a new method to establish a calculation model of the bond strength between deteriorated concrete and heavily corroded deformed steel bars.

In this paper, experiments were conducted to explore the degradation of bond strength between deteriorated concrete and corroded deformed steel bars at different levels. The effects of the concrete compressive strength and the thickness of the relative protective concrete on the failure mode and ultimate bond strength are discussed. Then, a simplified calculation model of the bond strength between the cracked concrete and heavily corroded deformed steel bars is established based on the bonding theory of concrete and plane steel bars by considering the corrosion ratio of the steel bars, concrete compressive strength, concrete crack width and friction factor between the deteriorated concrete and the corroded steel bars.

2. Experimental program

2.1. Materials

Ordinary Portland cement of 42.5 grade with a specific gravity of 3.28 and a specific surface area of 350 m^2/kg was used in this investigation. The coarse aggregate was crushed stone with a maximum size of 31.5 mm. Natural river sand, which is medium sand with a fineness modulus of 2.42, was used as the fine aggregate. The sand ratio was fixed at 33%. The concrete mixtures were designed according to Chinese Standard JGJ55 [19]. The mixture proportion by weight was as follows: water: cement: fine aggregate: coarse aggregate = 195:362:608:1234. The concrete has a 28-day cubic compressive strength of 34.0 MPa, and the compressive strength was tested according to the Chinese standard GB/T 50152-2012 [20].

Deformed steel bars with four different diameters were selected for the investigation, including 12, 16, 20 and 25 mm, as illustrated in Table 1. A plane steel bar with a diameter of 6 mm was selected as the stirrup.

2.2. Test design

Twenty groups of short anchor specimens with physical dimensions of 240 mm (length) × 150 mm(width) × 160 mm(height) were created for this investigation, with 5 groups of each deformed steel bar and 2 blocks in each group. As shown in Fig. 1, the smallest thickness of protective concrete was 25 mm, and the lengths of the free end and load end of the steel bar were 20 and 320 mm, respectively. Three parameters were considered in the design of the specimens: (a) a small bond length (100 mm) was adopted to obtain an even bond stress; (b) non-bond sections were designed at both ends of the steel bar, which was protected by a PVC tube, and the length was taken as 5 times the diameter of the steel bar; and (c) one piece of stirrup was placed in the specimen to simulate reinforced concrete structures in service.

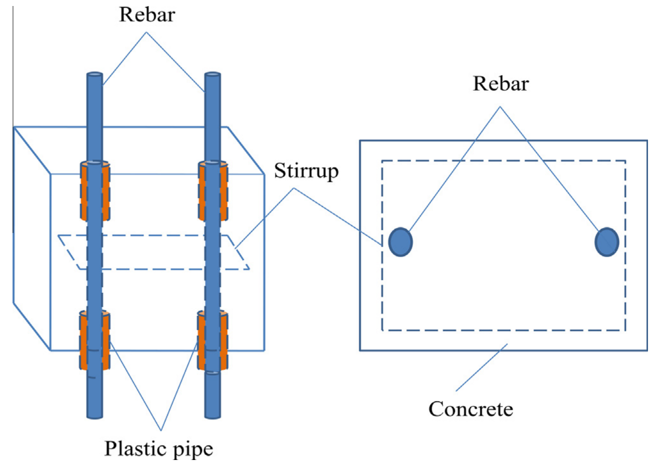

Fig. 1. Sketch of specimen.

Table 2
Design of simulation environment.

NaCl (mass ratio)	$MgCl_2$ (mass ratio)	Na_2SO_4 (Mass ratio)	CO_2 (mg/m^3)	HCl (mg/m^3)	Temperature (°C)	Humidity (%)
0.25%	2.71%	10%	5778	128	50	75

To accelerate the investigation, the simulation environment was designed according to in-situ data and parameters. The following assumptions were made based on the main characteristics of a colliery environment [6]. (a) the HCl and Cl_2 products are dissolved in water vapour or liquid rain and deteriorate the concrete by producing H^+ and reacting with OH−. Then these products can be replaced by HCl according to the concentration of H+. Due to lower solubility and weak acidity, carbonization was achieved by spraying carbon dioxide; (b) Because the main corrosive medium in the liquid wassodium, potassium, magnesium, chloride or sulphate, the SO_4^{2-}, Cl^-, Na^+, K^+ and Mg^{2+} in the liquid can be converted to NaCl, $MgCl_2$ and Na_2SO_4 according their concentration. All Na+ was assumed to be contained in Na_2SO_4·KCl was assumed to be replaced by NaCl, and $CaCl_2$ was assumed to be replaced by $MgCl_2$. The mixture was sprayed into the test equipment. (c) The solids were given no special considerations because they were mainly the products of the above reactions, although some contained dust. (d) The wetting-drying cycle was neglected as temporary. To accelerate the investigation, the parameters were amplified to 50 times the test results, as illustrated in Table 2. The exposed bar was brushed with epoxy resin to resist corrosion and the end of the PVC tube was sealed with a silicone polymer before the specimens were placed in the simulation environment.

The deterioration behaviours of the specimens were detected at 90-day intervals, and the total time of the investigation was 360 days. After each cycle, the specimens were taken out of the simulation environment and naturally dried. Then they were loaded into an electro-hydraulic servo testing machine, according to GB/T 50152-2012. The setup of the test and instruments is shown in Fig. 2. LVDTs were placed at both the free end and load end of the specimen to measure the relative slip between the steel bar and concrete. The entire procedure from the start of loading to the specimen destruction was noted. Data acquisition instruments were used to collect the test data.

Table 1
Performance of steel bars.

Type	Diameter/mm	Yield strength/MPa	Ultimate strength/MPa	Elongation/%	Linear density/kg m^{-1}
ϕ6	6	371	528	26.1	0.253
ϕ12	12	373	553	30.7	0.782
ϕ16	16	404	570	31.0	1.469
ϕ20	20	386	593	28.0	2.351
ϕ25	25	372	573	30.7	3.656