

Bond Deterioration of Corroded Steel in Two Different Concrete Mixes

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ABSTRACT

This paper investigated the degradation of bond performance between rebar and concrete due to steel rebar corrosion. Tests were carried out to evaluate the degradation of bond between reinforcing steel and concrete for different corrosion levels of reinforcing steel. A series of 120 specimens of two different concrete mixes (compressive strength of 20.7MPa and 44.4MPa) with various reinforcing steel corrosion levels were designed and manufactured. Each specimen was cast as a 200 mm concrete cube, and a steel rebar was centrally embedded with two stirrups around it. The steel rebar were corroded using an electrochemical accelerated corrosion technique. The corrosion crack opening width and length were recorded after the corrosion process. Then monotonic and cyclic pull-out loading tests were carried out on the specimens. The effects of steel corrosion with different concrete strength on crack opening, bond strength and corresponding slip value, slope of initial, residual bond stress, mechanical interaction stress, and energy dissipation, were discussed in detail. Especially, the mean value and coefficient of variance of these parameters were derived.

1. INTRODUCTION

It is well known that corrosion of steel rebar is a major problem influencing the long-term performance of reinforced concrete infrastructures ^[1]. Corrosion of reinforcement normally occurs due to attack by aggressive agents such as chloride ions from the marine environment. Therefore expansive stresses are induced around corroded steel bars, causing possible cracking, spalling of the concrete cover and loss of bonding between the steel and concrete ^[2]. Bonding behaviors primarily dependent on three factors: the compressive strength of concrete, confinement, and the surface of the steel (deformed or round) ^[3]. Previous studies have been undertaken to evaluate the effects of corrosion on the bond strength ^[4-8]. However, little information is available on the effects of different concrete strength on bond performance of corrosion reinforcing steel in concrete. In this study, an experimental study on bond behavior of corroded reinforcing steel in two different concrete mixes was carried out, the difference between the two concrete mixes was detail discussed. And another Parameter coefficient of variance was proposed in this experiment.

2. TEST SETUP AND PROCEDURE

2.1 Test specimen

The specimens consisted of deformed steel bars set in a concrete prism with two stirrups to provide confinement (Fig.1). 2 PVC pipes were used to limit the bonded length to 80mm. The bonded length was much less than the development length of the steel, so the bond stress along the steel was relatively uniform. The two closely-spaced stirrups provided confinement along the bonded length and helped to limit any end effects. 120 specimens were cast in total in this test: 60 specimens of concrete strength have designed 20MPa, and other 60 specimens of concrete strength have designed 40MPa.

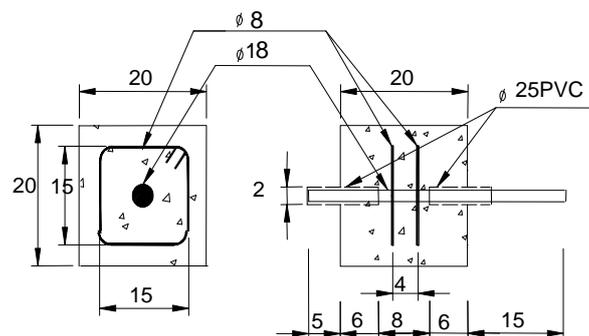


Fig.1 Specimen geometry (Unit: mm)

2.2 Material properties

The first concrete mix per cubic meter was: 336.02kg ordinary Portland cement, 201.61kg water, 725.81kg sand, and 1236.56kg stone. It was designed to have a compressive strength of about 20MPa with a w/c ratio of 0.6. The second concrete mix per cubic meter was: 418.06kg ordinary Portland cement, 200.67kg water, 677.26kg sand, and 1204.01kg stone. It was designed to have a compressive strength of about 40MPa with a w/c ratio of 0.48. Concrete cubes with dimensions of 100×100×100 mm³ were also cast for compressive strength testing. The two concrete mixes were found to have a 28 day average compressive strength of 20.7MPa and 44.4MPa.

2.3 Accelerated corrosion program

The specimens were corroded using an electrochemical accelerated corrosion technique which involved impressing a direct current through the specimens to accelerate the oxidation process in a 5% NaCl solution^[8]. The level of corrosion was roughly estimated according to the mass loss of the steel to establish different corrosion levels for the steel. In this study, the current density was set as 300μA/cm², the corresponding current in the steel was 13.57mA per specimen. The accelerated corrosion of the steel was actually carried out for 3 specimens in series (Fig.2). The maximum required artificial corrosion process took approximately 92 days.

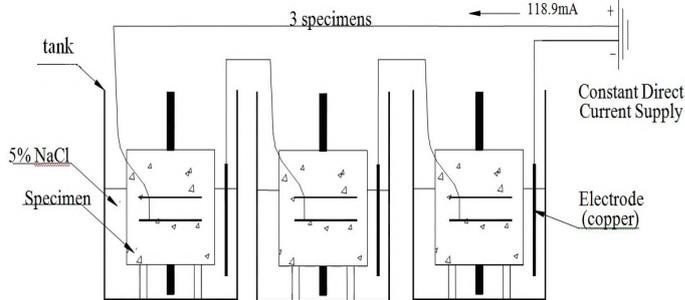


Fig.2 The electrochemical system

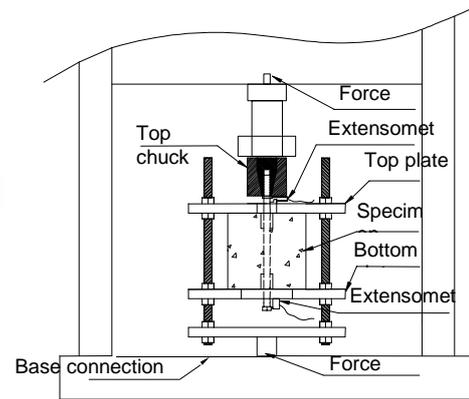


Fig.3 Loading and measuring system

2.4 Loading and measuring instrumentation

The corroded specimens were tested in MTS300 with a specially designed and fabricated loading frame. Fig.3 shows the schematic drawing of the loading system. Butter was used to lubricate the concrete and the steel plate surfaces to eliminate the effects of friction force. Both the loading-end and the free-end slips were measured using an extensometer with precision of ± 0.001 mm. The loading-end slip was limited to 3mm due to the space limitation. Three different loading schemes were used in this experiment. The first was monolithically increased slip loading until pull-out with the loading speed of 0.4 mm/min. The second was ± 0.1 mm cyclic slip loading repeated 10 times followed by pull-out. The third was a varied cyclic loading: first ± 5 kN loading for 3 cycles, then ± 0.1 mm~ ± 0.3 mm~ ± 0.6 mm slip loading, each amplitude loaded for 3 cycles, then pull-out. The loading speed was set at 0.4 mm/min for the cyclic loading and 2 mm/min for pull-out for the 2nd and 3rd loading cases.

3. TEST RESULTS

3.1 Monolithic pull-out loading

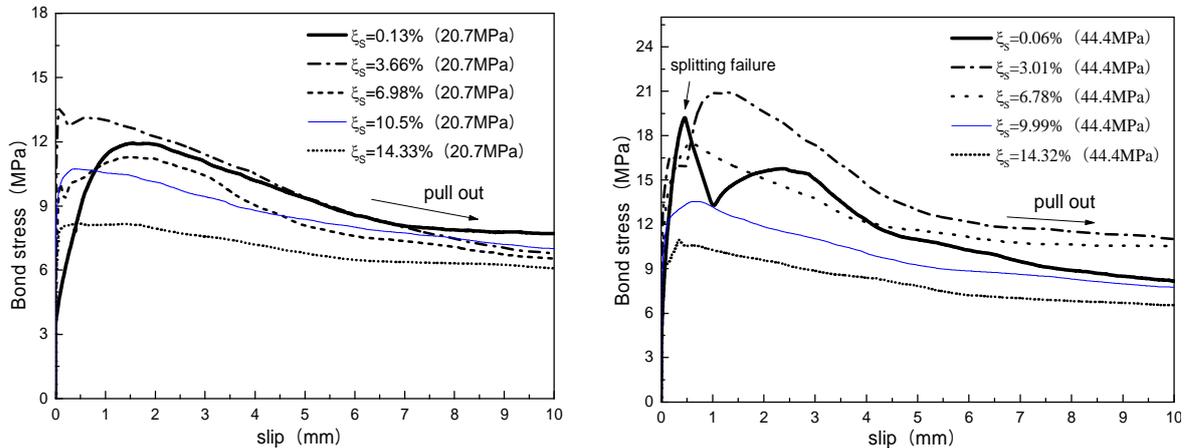
Fig.4 (a) shows five typical bond stress vs. free-end slip curves, the concrete strength is 20.7MPa. The bond stress was derived from the measured load force by the following equation:

$$\tau = \frac{P}{\pi dl} \quad (1)$$

where l is the bond length, d is the diameter of steel, and P is the measured pullout load force.

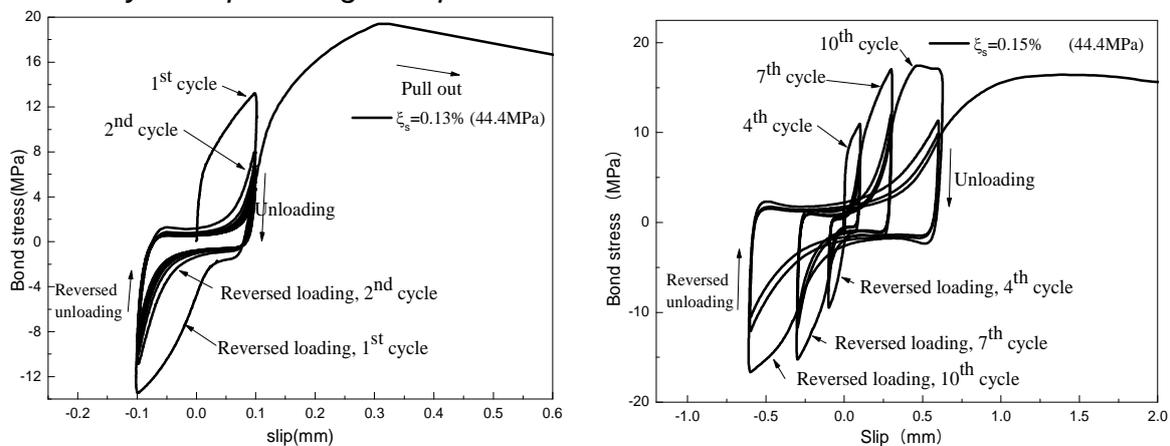
The curve could be divided into five stages for intact specimen and specimen with 10.5% corrosion level. The first stage of the curve corresponded to tiny slip value, bond stress increases very fast at the initial loading slip in the μm range, and the corresponding slope of curve is much larger than that of the following stages. In the second stage, bond stress increased steadily as the slip value increasing, until it reached to a kink. The bond stress increased slowly after this kink point until it reached the bond strength (maximum bond stress) which corresponded to bond failure [3], and

this is the third stage. After the maximum bond stress, the curve came to the fourth stage, which showed a stage of progressively diminishing bond stress until slip value reached to about 8.0mm. Then bond stress became steady and came to the residual bond stress corresponding to 10mm free-end slip. The curve of un-corroded specimen and specimen with 10.5% corrosion level actually showed the pull-out bond failure mode.



(a) Bond stress-slip curve (20.7MPa) (b) Bond stress-slip curve (44.4MPa)
 Fig.4 Bond stress-slip curves under Monolithic pull-out loading

3.2 Cyclic slip loading then pull out



(a) ± 0.1 mm slip cyclic loading (b) ± 5 kN~ ± 0.1 ~ ± 0.3 ~ ± 0.6 mm slip cyclic loading

Fig.5 Bond stress-slip curves under cyclic loading then pull out

Fig.5 (a) shows the measured bond stress-slip curves of specimens with 44.4MPa concrete strength under 10 cycles of 0.1mm slip loading then pull-out. The bond stress-slip curve starts at a slip value, and ascends toward a peak value of the bond stress. Bond strength decreased rapidly after the first cycle of both loading and reversed loading, and slowed thereafter. The maximum bond stress of first cycle was about 40.21% higher than that of the second cycle.

Fig.5 (b) shows the tested bond stress-slip curve of specimen with 44.4MPa concrete strength under ± 5 kN~ ± 0.1 ~ ± 0.3 ~ ± 0.6 mm cycled slip loading followed by

pull-out. The first 3 cycles of $\pm 5\text{kN}$ loading cannot be clearly observed, as the stress-slip curve is almost linear during the loading and unloading processes, which confirmed that little damage occurred during the $\pm 5\text{kN}$ loading process. The maximum bond stress had appeared the 10th cycle of the bond slip curve, which the specimen steel corrosion ratio is 0.15%.

3.3 Bond strength

Fig. 6 shows the bond strength vs. steel corrosion ratio. The mean value and the coefficient of variance ^[9] are both shown near the green star in a bracket and separated by a comma, respectively. It clearly shows an increment of coefficient of variance for specimens with corroded steels compared to those with un-corroded steels. Two batches of specimens clearly showed an increment in the bond strength for 3.6% corrosion level compared to that of the intact steel. After 3.6% corrosion level, a decreasing trend for the mean bond strength could be observed for higher steel corrosion level. It was observed that slight corrosion increased the bond strength. Meanwhile, the increasing amplitude of bond strength of the first batch of specimens was larger than the second batch of specimens. The coefficient of variance shows an increment for specimens of two kinds of concrete strength.

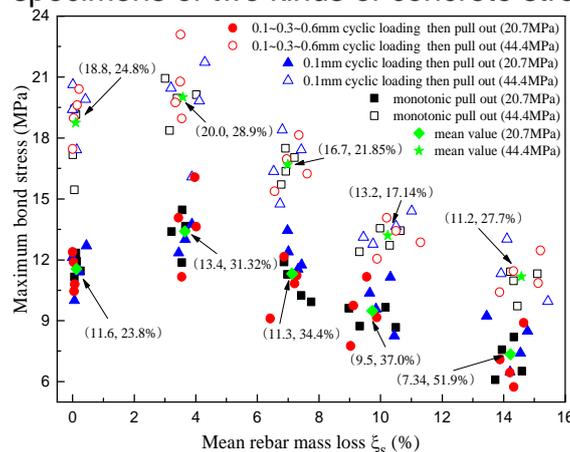


Fig.6 Bond strength vs. steel corrosion ratio

3.4 Slope of initial

The slope of initial was derived by linear fitting of the bond-slip curve for the bond force between 0 to 10.0kN. Fig.7 shows the slope of initial for each tested specimen. The mean slope of initial of two batches of specimens corresponding to 3.64% steel corrosion level was increased compared to that of intact steel bar. As the steel corrosion level continued increasing, the mean slope of initial decreases. It can be noted that increasing amplitude of specimen with 20.7MPa concrete strength was larger than the second batch of specimens. And the coefficient of variation of corroded specimens was obviously larger than that of specimens with intact steel.

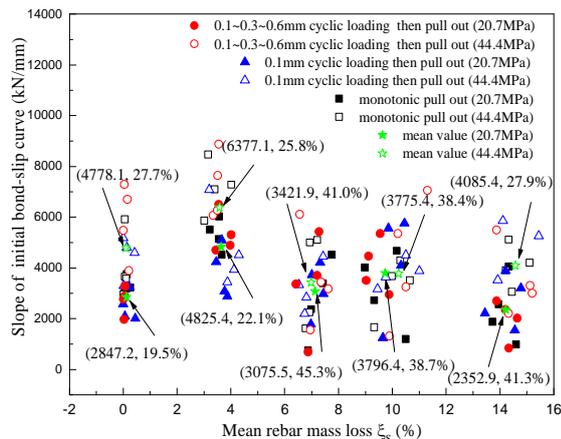


Fig.7 Slope of initial vs. steel corrosion ratio

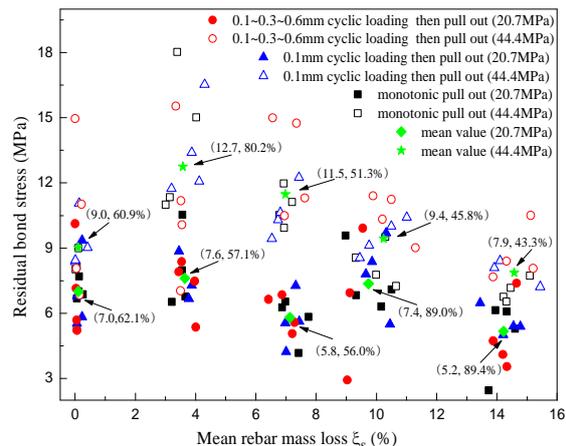


Figure.8 Residual bond stress vs. steel corrosion ratio

3.5 Residual bond stress

Residual bond stress is the friction between steel and concrete. Fig.8 shows the residual bond stress of each tested curve corresponding to 10mm free-end slip. The mean residual bond stress of the first batch of specimens was increased from 7.0MPa of intact steel bar to 7.6MPa of 3.6% corrosion level, and the second batch of specimens increased from 9.0MPa of intact steel bar to 12.7MPa of 3.58% corrosion level. As corrosion level increased, the mean residual bond stress of two batches of specimens decrease gradually. It can be seen that the higher the concrete strength, the faster the residual bond stress increases when steel corrosion level is small. And the coefficient of variation of corroded specimens was also larger than that of specimens with intact steel.

3.6 Mechanical interaction stress

The definition of mechanical interaction stress is the difference between maximum bond stress and residual bond stress of 10mm slip. Fig.9 shows the mechanical interaction stress of each tested specimen. It is clearly indicated that two batches of specimens showed an increment in mechanical interaction stress for 3.6% corrosion level compared to that of the intact steel bar. As corrosion level increased, the mean residual bond stress decreased gradually. And the reducing amplitude of the second batch of specimens was larger than the first. Moreover, the coefficient of variation of corroded specimens was also larger than that of specimens with intact steel bar.

3.7 Energy dissipation

3.7.1 Monolithic pull out loading energy dissipation

The energy dissipation of bond slip curves was derived by calculating the area of the bond-force-slip curves. Fig.10 shows the mean value and coefficient of variance of energy dissipation; For the 3.55% steel bar corrosion level, both the energy dissipation of 44.4MPa concrete mix and the energy dissipation of 20.7MPa concrete mix slightly increased compared to that of intact steel bar. For higher steel bar corrosion levels, a diminishing of energy dissipation could be observed for both of them. The reducing

amplitude of the second batch of specimens was larger. Besides this, Fig.13 also showed an increased coefficient of variance for corroded members than that of un-corroded specimens for this test.

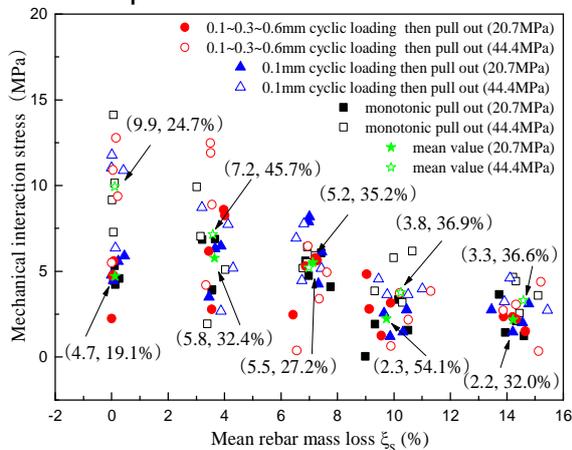


Figure.9 Mechanical interaction stress

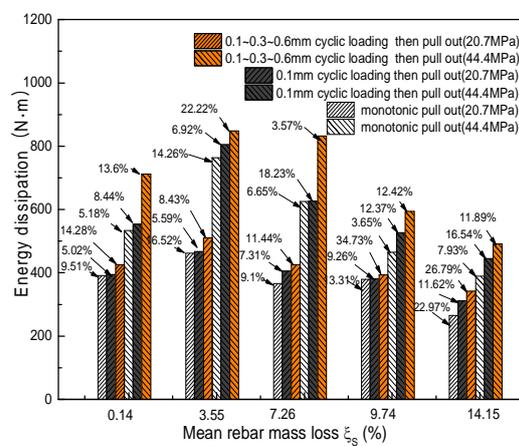


Figure.10 Energy dissipation

3.7.2 Cyclic slip loading energy dissipation

Fig.10 showed the energy dissipation of cycle stage under 0.1mm cyclic loading. It clearly shows both the energy dissipation of two batches of specimens corresponding to 3.55% steel corrosion level was increased compared to that of intact steel bar. As the corrosion level continued increasing, the mean energy dissipation of the first batch of specimens for 7.26%, 9.74%, 14.15% steel bar mass loss were smaller than that of intact steel bar in this test. Such as the mean energy dissipation of 14.15% corrosion level was decreased 18.87% compared to that of intact steel bar. The trend of the mean energy dissipation of the second batch of specimens was similar to the first.

Fig.10 showed the energy dissipation of cycle stage under $\pm 5\text{kN} \sim \pm 0.1\text{mm} \sim \pm 0.3\text{mm} \sim \pm 0.6\text{mm}$ cyclic loading. The energy dissipation of first three cycles are not calculation due to the stress-slip curve of first three cycles is almost linear during the loading and unloading processes. For the first batch of specimens, it clearly shows the mean energy dissipation corresponding to 3.55% corrosion level was increased compared to that of intact steel bar. As the steel bar corrosion level continued increasing, the mean energy dissipation decreases. The mean energy dissipation of specimens with 44.4MPa concrete strength is similar to the 20.7MPa. It is also showed an increased coefficient of variance for corroded members than that of un-corroded members for this test both of two batches of specimens.

4. CONCLUSIONS

This paper reported a detailed experimental study of steel corrosion effects on bond performance of reinforcing steel in two different concrete mixes. It was found that:

(1) Slight steel corrosion may increase bond strength and slope of initial. The increasing amplitude of them of the first batch of specimens was larger than the second batch of specimens. The reducing amplitude of mean bond strength of two batches of specimens was almost the same.

(2) Slight steel corrosion may also increase the residual bond stress. However, the mean residual bond stress of two batches of specimens decreased gradually with the increasing of steel corrosion level. The increasing of pressure between concrete and steel with the appearing of corrosion products may contribute to the increasing of the residual bond stress when corrosion level was slight.

(3) The specimens with 20.7MPa concrete strength shows an increment in mechanical interaction stress for 3.6% corrosion level compared to that of the intact steel bar, but the specimen with 44.4MPa concrete strength has been decreased.

(4) Energy dissipation also increased when steel bar corrosion is slight. With the corrosion level increasing, a diminishing of energy dissipation could be observed for both of them. However, the amplitude of increasing or decreasing of specimens with 44.4MPa concrete strength was larger than specimens with 20.7MPa concrete strength.

(5) The coefficient of variance for these indexes of corroded members was obviously larger than that of un-corroded members. The reason behind this phenomenon was attributed to the complex and random nature of concrete material, together with reinforcement corrosion initiation and its consequence. It may be an important basis for the evaluation of performance of using structure.

This study shows the importance and complexity of reinforcement corrosion to bond performance. But detailed investigations and further developing a numerical bond-slip model considering uncertainty to quantify these effects will be carried out in the near future.

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