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Effect of corrosion on bond in reinforced concrete under cyclic loading

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Abstract

Cyclic loading can result in severe deterioration in the bond between reinforcing steel bar and the surrounding concrete, especially when the reinforcement is corroded. In this study, tests were carried out for bond stress–slip response of corroded reinforcement with concrete under cyclic loading. Parameters investigated include: corrosion level, confinement, bar type, and loading history. The results revealed that bond behaviour was significantly reduced under cyclic loading. Degradation in bond was significantly less for deformed bars than for smooth bars at the initial loading cycle, but the difference was diminished with loading. The bond reduction was more substantial for unconfined steel bars than for confined bars. The relatively high level of corrosion caused degradation primarily in the initial five cycles, the effect of corrosion being decreased with loading. It was also demonstrated that the cyclic bond stress–slip curves depended on loading history.

Keywords: Cyclic loading; Corrosion; Bond strength; Reinforcement; Concrete

1. Introduction

Structures located in seismic zones may be subjected to several reversals of loading. Reversed cyclic loading can cause deterioration in the bond between reinforcing steel and concrete [1–3]. Bond–slip results from corroded reinforcing steel bar with concrete under cyclic loading are valuable, since the amount of data in this area is very limited.

The interaction of reinforcing steel bars with concrete is a quite complex phenomenon that has important effects on the response characteristics of reinforced concrete elements and structures under static and dynamic loads [4]. A great number of research programmes have been carried out in the area of steel–concrete bond behaviour, such as those by Tepfers [5,6]. Also, many researchers have conducted studies for the mechanism of bond failure. For example, Cairns and Jones proposed an approach for the relationship between bond strength and splitting, in which bond strength of ribbed bars is composed of non-splitting and splitting components [7,8]; Balázs developed a theory for crack formation in reinforced concrete based on an analysis of slip, bond stresses and steel stresses [9].

The deterioration in bond is of particular concern for concrete structures when the embedded steel reinforcing bars are corroded. Many studies have been conducted for the influence of corrosion on bond between steel and concrete [10–19]. Although the corrosion product on the steel in some cases improves the bond between steel and concrete [20,21], corrosion is considered one of the main causes for the limited durability of steel-reinforced concrete. The volume increase due to corrosion causes splitting and leads to weakening of the bond, which directly affects the serviceability and ultimate strength of reinforced concrete members within a structure [17].

On the other hand, a number of studies have been carried out on bond strength of steel bars under cyclic loading. Tassios et al. conducted a comprehensive study of the bond–slip behaviour and proposed a model for bond–slip under cyclic loading [22]. Eligehausen et al. conducted integrated experimental and analytical investigations that permitted analytical prediction of the local bond stress–slip relationship of deformed reinforcing bars under both monotonic and cyclic loading [23]. Filippou et al. [2] proposed some modifications to the model originally proposed by Eligehausen et al. Yankelevsky et al. proposed a model to represent the typical cyclic bond stress–slip relationship [24]. Zuo and Darwin compared the load–slip behaviour of high relative rib area and conventional reinforcing bars subjected to cyclic loading [25]. Lundgren studied bond–slip of steel bar

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and splitting effects experimentally in both monotonic and cyclic loading [26]. Coronelli and Mulas investigated the effect of bond deterioration inside the joint cores on the response of an R/C plane frame subjected to a strong ground motion [27]. Alavi-Fard and Marzoulk examined the bond slip characteristics of high-strength concrete under cyclic loading [28]. They investigated cyclic bond of high-strength concrete under different parameters, including loading history, confining reinforcement, bar diameter, concrete strength, and the rate of pullout. Alavi-Fard and Marzoulk concluded that cyclic loading does not affect the bond strength of high-strength concrete as long as the cyclic slip is less than the maximum slip for monotonic loading.

However, a very limited number of research programmes [29] have been conducted for both corroded reinforcement and cyclic loading. Lack of experimental data on bond between corroded steel bars with concrete under cyclic loading often leads to a conservative design of the structures. In this study, the effect of corrosion on the bond strength between corroded steel bar and concrete under cyclic loading was investigated experimentally.

2. Experimental programme

2.1. Test specimens

The mechanical properties of the steel bars are shown in Table 1. Some specimens contained two closed stirrups, as shown in Fig. 1. The bars were descaled and cleaned before being cast into the concrete specimens. The embedment length chosen was 4 times the bar diameter, e.g., 80 mm. This short embedment length was selected to avoid yielding of the steel bar under loading.

Ordinary (ASTM C150 Type I) Portland cement, fine aggregate (medium-sized natural sand), and coarse aggregate (crushed limestone) in the ratio of cement/sand/coarse aggregate = 410:658:1120 with a water/cement ratio of 0.44 were used to prepare the concrete mixture. Concrete cubes with dimensions of $100 \times 100 \times 100$ mm^3 were also cast for the concrete compressive strength.

After casting, the concrete specimens were kept in the moulds and covered with damp polyethylene for 3 days. They were then demoulded and kept in a curing room maintained at 20 ± 3 °C and 90% or greater relative humidity. The 28-day cured specimens had an average measured strength of 56.2 MPa.

2.2. Accelerated corrosion

Electrolytic methods were used to accelerate steel corrosion. In the procedure of corrosion, electric current and the duration were used as the indications for the theoretically calculated corrosion levels. Direct electric current was impressed on the main steel bar embedded in the concrete, using an integrated system incorporating a rectifier with a built-in ammeter to monitor the current, and a potentiometer to control the current intensity. The direction of the electric current was such that the reinforced steel bar served as the anode while a stainless steel plate counter-electrode was positioned in the tank to act as a cathode. Fig. 2 shows schematically the electrolytic corrosion system used. The stainless steel plate that served as the cathode consumed the excess electrons given off by the reinforcement during the corrosion process. Power supplies with adjustable voltage and current (from 0 to 2 A) were chosen for the electrolytic corrosion process. To ensure that only the bonded zone would be corroded, one end of the steel bar was insulated during the corrosion. The degree of non-uniformity of corrosion along the embedded reinforcing steel bar was greatly reduced by adding a small amount of NaCl to the concrete mixture before casting.

To establish different corrosion levels for steel bars, tables showing durations of transmitted current and the corresponding degrees of corrosion were prepared according to Faraday's Law. The theoretically calculated amount of corrosion was estimated according to the corresponding degree of corrosion and the weight of the steel bar along the bond length, based on the unit length of the bar. The theoretically calculated amount

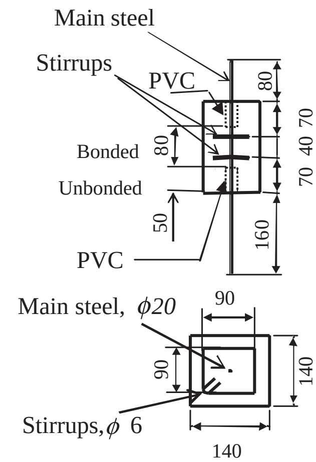

Fig. 1. Geometry of the specimens. All dimensions in mm.

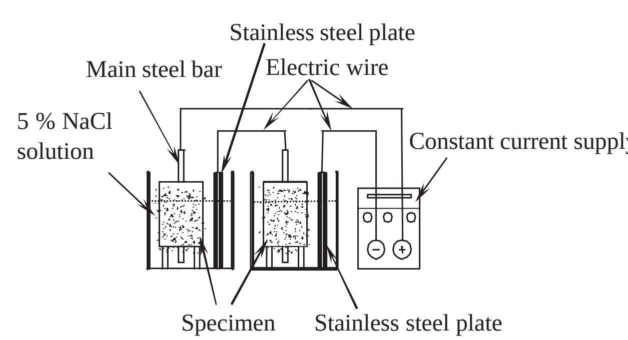

Fig. 2. Electrochemical corrosion system.

Table 1
Mechanical properties of test steel bars

	Materials	Yield stress (N/mm^2)	Tensile strength (N/mm^2)	Elastic modulus (N/mm^2)	Elongation (%)
Smooth bar	A3	289.6	440.0	210 000	30.3
Deformed bar	20MnSi	350.9	521.2	200 000	34.0

of corrosion products, in terms of the mass loss of steel due to corrosion, can be expressed by the following equation [14]:

$$m_t = \frac{t \times \bar{I} \times 55.847}{2 \times 96487} \quad (1)$$

where t is the duration of exposure and $\bar{I}$ the average magnitude of electrical current.

It should be noted that the theoretically calculated corrosion levels as well as the corresponding amounts of corrosion (Eq. (1)) were used only to roughly adjust the electric current and the duration in the corrosion procedure. The actual degree of corrosion, or corrosion level, was accordingly measured as the loss in weight of the steel bar relative to the weight of the bond length before corroding, thereby representing an average corrosion level along the bond length; see Eq. (2). Thus, the actual corrosion level was determined with the following equation:

$$C_R = \frac{G_0 - G}{g_0 l} \times 100\% \quad (2)$$

where G_0 is the initial weight of the steel bar before corroding, G the final weight of the steel bar after removal of the corrosion products, g_0 the weight per unit length of the steel bar, and l the bond length.

2.3. Cyclic loading

The tests were performed using a servo-controlled hydraulic test machine to apply the cyclic loading. A specially designed and fabricated loading frame was used that was fixed to the base of the machine during the loading. Since the length of steel bar in contact with concrete is short, the recorded bond stress can be considered as representative of the bond stress. In order to measure the slip more accurately, one crack opening displacement (COD) gauge was used during loading where the slip was less than 2.0 mm, while a linear variable differential transducer (LVDT) was used all the time during loading. The setup for COD is shown in Fig. 3. All the loads were applied through the computer, using displacement-control. The loading speed was 0.004 mm/s during cyclic loading, and 0.02 mm/s thereafter in the final procedure of pulling out the bar from concrete after 10 cycles of cyclic loading. For all the tests, the strain in the steel bars was very low and yielding of the steel bars was not observed. Table 2 shows the number of test specimens of each type tested.

Based on the results of previous pullout tests [16], different displacements with low and high values were selected to illustrate the effect of load history on steel–concrete bond. Load history was applied through a computer program. Some specimens were subjected to a constant displacement of 0.3 mm, 1.0 mm or 1.5 mm. Some others were loaded first at a small displacement of ±0.1 mm for several cycles, and subsequently at ±0.3 mm, ±0.6 mm or ±1.0 mm.

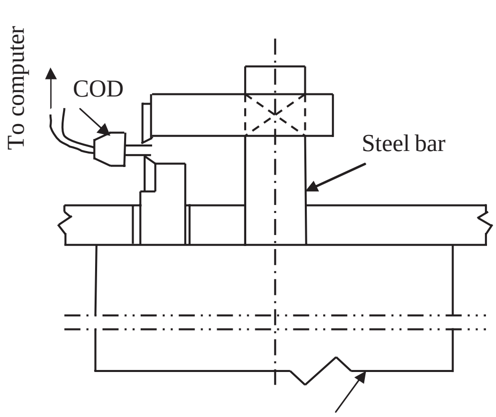

Fig. 3. Schematic representation of crack opening displacement gauge setup.

Table 2
Test specimens

Bar type	Type of loading displacement	No. of specimens tested, n
S1	Fixed displacement	4
S1	0.1∼0.3∼1.0 mm	3[a]
S2	Fixed displacement	4
D1	Fixed displacement	4
D1	0.1∼0.3∼0.6 mm	3[a]
D2	Fixed displacement	6

S1 – smooth without stirrups; S2 – smooth with stirrups; D1 – deformed without stirrups; D2 – deformed with stirrups.
[a] One specimen in each of these groups failed in splitting during cyclic loading.

3. Test results and discussion

3.1. General observations

A typical cyclic bond stress–slip curve for confined specimens with constant displacement is shown in Fig. 4. As the specimens were well confined, the bond stress–slip curve under cyclic loading can be characterized by a monotonic bond–slip envelope and curves of reduced resistance. The

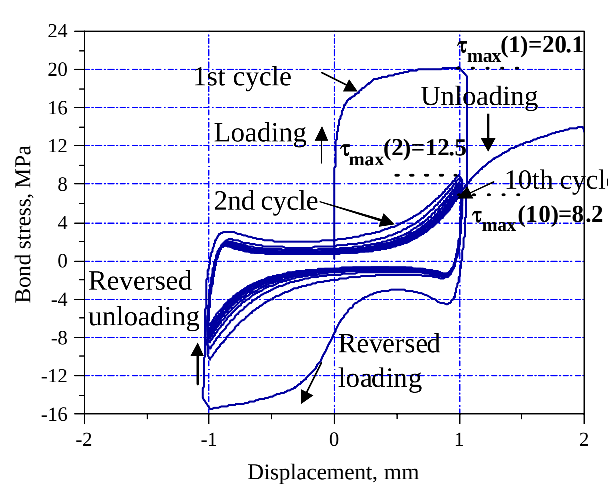

Fig. 4. A typical bond stress–slip under constant displacement. $\tau_{max}(N)$ designates the maximum bond stress at the Nth cycle.

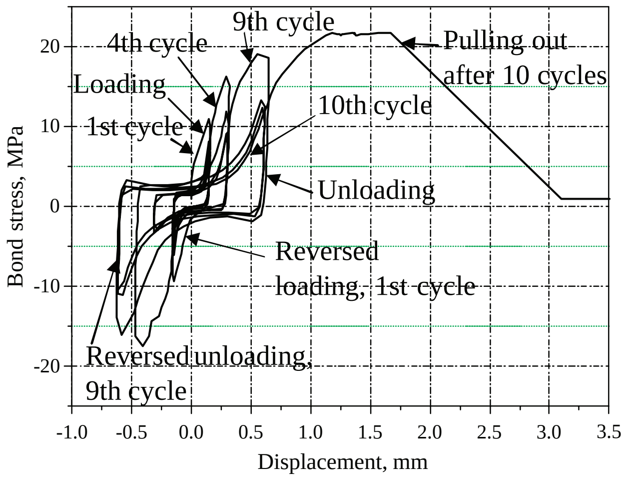

Fig. 5. D1 control specimen (0% corrosion) under 0.1∼0.3∼0.6 mm loading.

typical bond stress–slip curve starts at a slip value of zero, and ascends toward a peak value of the bond stress. Bond decreased rapidly at the first cycle, and slowed thereafter; see Fig. 4. For all the specimens, the larger part of the reduction in bond strength took place in the initial cycle. The bond stresses of unloading branches were about 25% lower than that of the loading branch of the same cycle.

A recorded typical bond stress–displacement response for a load history of 0.1∼0.3∼0.6 mm is plotted in Fig. 5. Cracking did not occur at the smallest displacement, 0.1 mm. In all figures, bar slip in the direction of pulling out is defined as positive, while that in the opposite direction is defined as negative. Fig. 5 indicates that, at all three displacement levels, 0.1 mm, 0.3 mm and 0.6 mm, the maximum bond capacity was not reached. Bond stress reached its maximum value of 21.47 MPa at the pulling-out phase with a corresponding slip of 1.67 mm. At each assigned displacement, as the number of cycles was increased, more reduced envelopes were formed. These observations illustrated the degradation in bond between reinforcing steel bar and concrete under cyclic loading. The bond strength of the unloading branch was lower than that of the loading branches of the same cycle. This indicates that the unloading and reloading branches of the curves as well as the reduced envelopes depended on the load history [28]. In Fig. 5, the rapid drop in the curve after the maximum bond stress indicated a splitting failure.

3.2. Confinement

Table 3 shows bond degradations of some specimens under constant-displacement cyclic loading. Comparing confined smooth bars with unconfined smooth bars shows that both reached their highest bond capacity at the first loading cycle. Bond stress–slip results from constant-displacement cyclic loading presented a behaviour characterized by a typical monotonic bond–slip envelope and curves of reduced resistance, which are cycle- (N) and amplitude- (S_{max}) dependent [24]. If a degradation parameter is used to indicate the reduction of the maximum bond stress at the Nth cycle, the degradation in bond during the Nth cycle can be expressed as:

$$\lambda_N = 1 - \kappa = 1 - \frac{\tau_{max}(N)}{\tau_{max}(1)} \quad (3)$$

where λ_N is the degradation parameter of the Nth cycle, $\tau_{max}(1)$ the maximum bond stress of the first cycle, $\tau_{max}(N)$ the maximum bond stress of the Nth cycle, and κ the ratio of the maximum bond stress to that of the first cycle. The smaller the degradation parameter is, the less the reduction in bond capacity.

The test results show that the degradation parameters for confined steel bars are consistently smaller than those for unconfined ones. For unconfined corroded deformed bars, splitting failure was observed after the bond stress reached its maximum value, followed by a sharp drop of bond stress to a very low level. For confined specimens, a pullout type of failure was observed. It was demonstrated that bond loss was less for confined specimens than for unconfined specimens. Besides, stirrups contributed to a small crack width when bond failed, about 0.2 mm. As a comparison, specimens without stirrups generally split at a very small slip, often with a crack width up to 1.5 mm or even larger, as shown in Fig. 6(A) and (B). In areas of seismic risk, the effect of confinement reinforcement to reduce bond deterioration under cyclic loading enhances the energy absorption and dissipation capabilities and, consequently, improves the chances that the structure or structural components will survive earthquake loading [30].

3.3. Bar type

As shown in Table 3, for the confined deformed bar with a corrosion level of 4.7%, the degradation parameters at the 2nd and the 10th cycles were 0.384 and 0.616, respectively. As a

Table 3
Bond degradations of some test specimens under constant displacement cyclic loading

Specimen type	Corrosion (%)	Loading displacement	$\tau_{max}(1)$ (MPa)	$\tau_{max}(2)$ (MPa)	$\tau_{max}(10)$ (MPa)	$\lambda_2 = 1 - \frac{\tau_{max}(2)}{\tau_{max}(1)}$	$\lambda_{10} = 1 - \frac{\tau_{max}(10)}{\tau_{max}(1)}$
S2	2.9	0.3 mm, constant	14.7	7.1	6.0	0.517	0.592
S1	1.7	0.3 mm, constant	14.1	4.9	3.5	0.653	0.752
D1	0.6	1.0 mm, constant	Splitting failed during loading				
D2	0.0	1.0 mm, constant	20.2	7.6	4.6	0.624	0.772
D2	1.0	1.0 mm, constant	20.1	12.5	8.2	0.378	0.592
D2	4.7	1.5 mm, constant	23.2	14.3	8.9	0.384	0.616
D2	5.1	1.0 mm, constant	20.6	12.5	8.2	0.392	0.602
D2	7.6	1.0 mm, constant	20.8	12.1	7.1	0.418	0.659

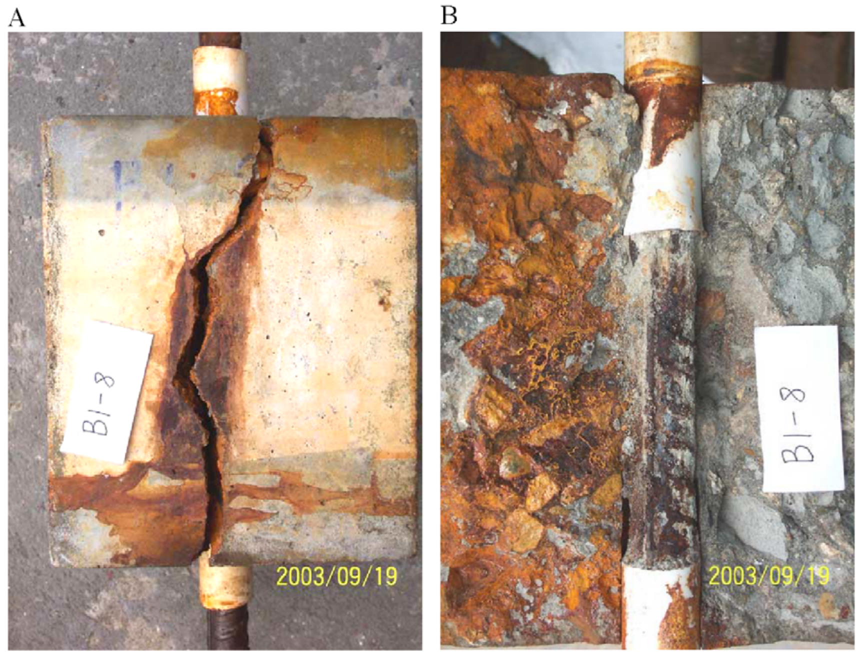

Fig. 6. Splitting of corroded D2 specimen: (A) wide crack after cyclic test; (B) after breaking of the specimen.

comparison, for the confined smooth bar with a corrosion level of 2.9%, the degradation parameters at the 2nd and 10th cycles were 0.517 and 0.592, respectively. The comparison shows that the degradation in bond during the first loading cycle was substantially less for the deformed bars than for the smooth bars; however, the difference was significantly lower after 10 loading cycles. As a comparison, results from the previous pullout tests for specimens of the same profile also showed that the maximum bond strength for deformed bars was 3.5–6.9 times as high as for the smooth bars [16].

Bond stress–slip curve of unconfined specimens indicated that when varying displacement was assigned with the largest displacement applied first and the smallest last, the maximum bond capacities were realized at the loading phase of the first cycle; see Figs. 7 and 8. Unlike the bond–slip curves for deformed bars shown in Fig. 5, where the peak bond stress increased with the assigned displacement, the peak bond stress decreased as the displacement increased, as shown in Figs. 7 and 8. This indicates that the bond strength was lowered during the cyclic loading.

It was noted that for unconfined bars, the bond capacity of deformed bars was much higher than that of the smooth bars, as shown in Fig. 9. For unconfined bars, after 10 cycles of loading, the highest bond stress level of deformed bars was 5 to 10 times as high as that of smooth bars. A possible reason might be the resistance due to the contact interlock between the steel bar lugs and the surrounding concrete in the deformed bar specimens [31].

3.4. Corrosion level

The corrosion levels for the confined deformed bars tested under 1.0 mm constant displacement loading were 1.0%,

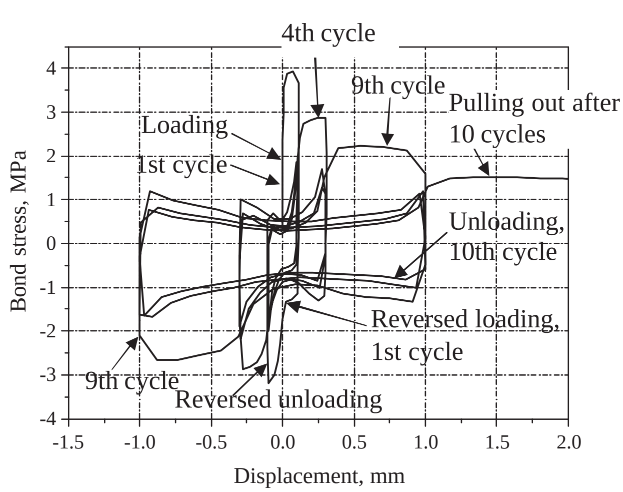

Fig. 7. S1 control specimen, under 0.1∼0.3∼1.0 mm loading.

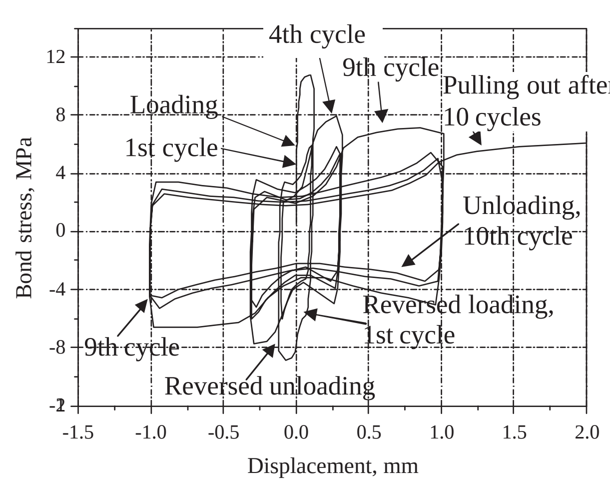

Fig. 8. S1 corrosion level 1.0% under 0.1∼0.3∼1.0 mm loading.

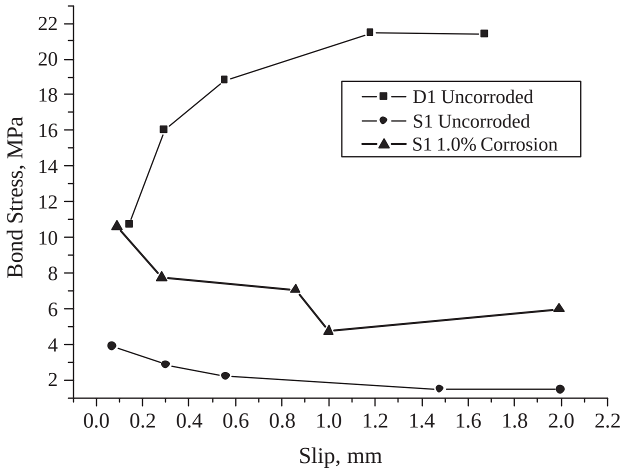

Fig. 9. Peak bond stresses at each displacement after 10 loading cycles, for different specimens.

5.1%, 7.6% and 0% (uncorroded control specimen), respectively. For these specimens, the maximum bond stresses at the first cycle were about the same. The degradations in bond at the 2nd cycle were respectively 0.378, 0.392, 0.418 and 0.624, as shown in Table 3. The curves of degradation parameter versus cycles for confined deformed bars of different corrosion levels are shown in Fig. 10. It was demonstrated that during the first loading cycle the bond was reduced most for the control specimen without corrosion. In the initial cycles the degradation decreased with corrosion level within the range between 0% and 5.1%, but increased with corrosion level when the corrosion level continued to increase thereafter. This indicated that the bond strength increases with corrosion level up to a maximum, after which increasing corrosion causes a significant reduction of bond strength. Similar findings were reported by Almusallum et al. [10] and by Amleh and Mirza [11]. For corroded reinforcement with a low to medium level of corrosion, corrosion products might have increased the pressure between reinforcement and the surrounding concrete, and therefore lowered the bond loss during the initial loading cycles. In these tests, the bond degradation was smallest for corrosion levels around 5%. For the specimens with a higher corrosion level, 7.6% for example, as the number of cycles increased, bond degraded drastically up to the 5th cycle, and thereafter degraded gradually up to the 10th cycle. All the degradation parameters at the 10th cycle for specimens with a corrosion level between 1.0% and 7.6% were more or less around 0.6, while the corresponding value for the control specimen, 0.772, was significantly higher. It was thus concluded that the relatively high corrosion level degraded the bond primarily in the initial five loading cycles, and this effect was lowered during subsequent loading cycles.

3.5. Further discussion

It was observed that the corrosion levels measured as the weight loss of the steel bars were generally lower than the corresponding ones theoretically calculated from the applied current. This difference may indicate that the permeability of concrete played a role in the corrosion procedure. Cairns and Melville made tests of different types of electrical measurement. They concluded that surface treatments have effects on the measurement of corrosion [32]. In the current study, however, the permeability of the concrete was not included in the theoretical calculation of corrosion level. Another factor that may have affected the corrosion is the heterogeneity of the coarse aggregate within the specimen. The results showed that, generally, the longer the specimens were immersed in the NaCl solution, the less were the differences between the measured and the theoretically calculated corrosion levels. Also, the test results demonstrated that the rate of corrosion increased after the cracking of concrete. Thus, cracking had significant influence on the measured corrosion levels. According to research by Capozucca, the commencement and propagation of corrosion in reinforced concrete structures depend on many factors, including aggressiveness and availability of reactants [33].

In the current study, the maximum difference in the measured corrosion levels between specimens of the same type was around 20%. Despite the data scatter, a reasonable reproducibility of the measured corrosion level as well as bond–slip performance for each type of specimen was achieved. It should be noted that the differences between the theoretically calculated and measured corrosion levels had no influence on the conclusions, since the theoretically calculated corrosion levels were not used for bond–slip but for the duration of the electrolytic corrosion.

The results of this study provide further evidence that a limited degree of reinforcement corrosion improves the bond between reinforcing steel and the surrounding concrete. The maximum corrosion which was observed to have improved the bond was around 5%. This is close to that from pullout tests conducted by Almusallam et al. [10] and Fang et al. [16]. The change of the effect of corrosion on bond can be attributed to the mechanical pressure on the surrounding concrete due to the expansion of the corrosion products compared to the steel [10].

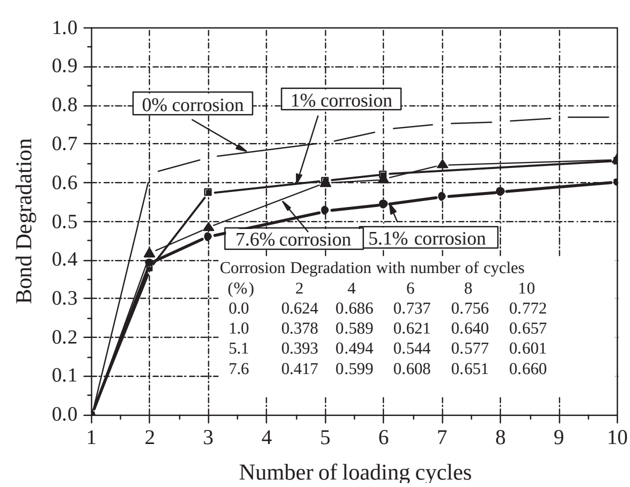

Fig. 10. Degradation parameter vs. number of cycles for confined deformed bars of different corrosion levels.

It has been observed by some researchers that for a small degree of corrosion, significant pressure is exerted on the concrete before cracking occurs [5,34].

Increasing the corrosion to higher levels led to an increasing reduction in bond strength. For all specimens, uncorroded control specimens and corroded specimens of all levels, the bond behaviour is significantly reduced under cyclic loading. As much as 38–63% of bond strength was lost during the first cycle for different specimens. After 10 cycles of loading, the bond strength loss was 59–77%. It was shown that the effect of a high level of corrosion on bond under cyclic loading substantially took place within the initial 5 cycles, and the effect due to corrosion was being decreased in the successive cycles. Confinement reinforcement was found to effectively reduce bond loss under cyclic loading. The study by Alavi-Fard and Marzouk for the uncorroded steel under cyclic loading showed that specimens with steel confining reinforcement sustained a greater number of cycles before indicating a bond failure [28]. In the current study, however, no similar finding was obtained because cyclic loading was applied to the specimen for only 10 cycles before the bars were pulled out from the concrete.

Several other factors also have effects on bond behaviour under cyclic loading. These factors include concrete strength, cover thickness, diameter of the reinforcing steel bar, and the loading rate. They were given fixed values in the current study. These aspects are to be included in future research programs.

4. Conclusions

(1) Bond behaviour of both deformed and smooth bars, confined or unconfined, is significantly reduced under cyclic loading. It was shown that substantial degradation in the bond capacity took place during cyclic loading, and that an increase in cyclic displacement will lead to more severe bond deterioration.
(2) The test results showed that severe corrosion will cause significant reduction in bond capacity under cyclic loading. This bond reduction primarily took place within the initial 5 cycles. However, corrosion of a limited low level (in these tests around 5%) increased the bond capacity.
(3) Confinement reinforcement was found to effectively reduce bond degradation under cyclic loading.
(4) More substantial bond reduction was observed for smooth bars than for deformed bars. For the case of confined specimens, the maximum bond stress of deformed bars after 10 cycles of loading was 5 to 10 times as high as that of smooth bars.

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