

# Cementitious bond degradation during cyclic shear loading, studied by contact electrical resistance measurement

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Degradation of the cementitious bond between steel rebar and concrete and that between old and new mortar under cyclic shear loading was observed nondestructively by measuring the contact electrical resistance of the joint. Degradation, which caused a decrease in bond strength but no visual damage, was indicated by an abrupt increase in the resistance at a small fraction of the fatigue life. Bond failure was also accompanied by an abrupt increase in resistance. © 2001 Kluwer Academic Publishers

## 1. Introduction

Cementitious bonding is involved in the joint between a steel reinforcing bar (rebar) and concrete and that between old concrete and new concrete (as encountered in the repair of concrete structures [1–7]). Partly due to the drying shrinkage of the concrete (i.e., new concrete), the quality of the bond is limited. The quality of the bond is critical to the performance of the steel reinforcement and of the concrete repair. Destructive measurement of the shear bond strength by pull-out, push-in and related testing methods is commonly used to assess the quality of a bond [8–22]. However, the bond may degrade at stresses below the shear bond strength, even though the degradation may not be visible. This degradation may occur during static or cyclic loading. In particular, cyclic loading may lead to fatigue.

Nondestructive methods of bond assessment are attractive for condition evaluation in the field. They include acoustic [23–25] and electrical [26] methods. In particular, measurement of the contact electrical resistivity of the bond interface has recently been used to investigate the effects of admixtures, water/cement ratio, curing age, rebar surface treatment and corrosion on the steel-concrete bond [26]. Degradation causes the contact resistance to increase. This paper uses this electrical method to monitor in real time the degradation of the bond during cyclic shear loading. Cyclic loading may lead to fatigue and the damage evolution is of scientific and technological interest.

## 2. Experimental methods

Concrete (with fine and coarse aggregates) was used for studying the bond between steel and concrete, whereas mortar (with fine aggregate only) was used for studying the bond between old mortar and new mortar.

### 2.1. Bond between steel and concrete

The cement used was portland cement (Type I) from Lafarge Corp. (Southfield, MI). Both fine and coarse aggregates were used. The fine aggregate was natural sand (99.9% SiO<sub>2</sub>), 100% of which passed #8 U.S. sieve. The coarse aggregate was #57 (ASTM C33-84), 100% of which passed 25 mm (1 in) standard sieve. The ratio of cement to fine aggregate to coarse aggregate was 1 : 1.5 : 2.5.

The water-cement ratio was 0.45. A water-reducing agent (TAMOL SN, Rohm and Hass Co., Philadelphia, PA; sodium salt of a condensed naphthalenesulphonic acid) was used in the amount 2% of the cement mass.

All ingredients except water were mixed in a concrete mixer at a low speed for 1 min. After that, water was added and then mixing was conducted at a high speed for 5 min. After this, the concrete mix was poured into oiled molds. A vibrator was used to facilitate compaction and decrease the amount of air bubbles.

The mild steel rebar was of size #6, length 150 mm, and diameter 19 mm, and had 90° crossed spiral surface deformations of pitch 26 mm and protruded height 1 mm.

A cylindrical piece of concrete labeled B (Fig. 1) was poured concentrically around a steel rebar A, such that the top flat surface of A protruded out of that of B, and the bottom flat surface of A was flush with that of B. The A-B joint was subjected to shear when B had been cured for 28 days. Shear stress was imposed by applying a downward load on the top flat surface of A, while the bottom flat surface of B was supported by a steel annular ring C with a central circular hole slightly larger than the cross-section of A. In this way, A went through the hole of C upon complete debonding at the A-B joint. The C was electrically insulated from A and B by using a paper lining.

Two electrical contacts in the form of silver paint in conjunction with copper wire strands were applied circumferentially around the protruded part of A and

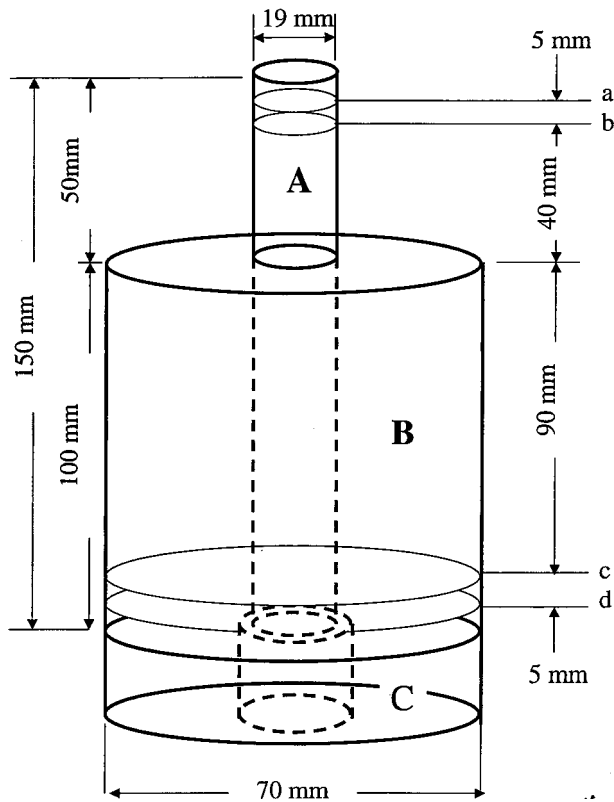


Figure 1 Configuration for measuring contact electrical resistance during shear loading of joint between steel rebar (A) and concrete (B). C was the steel support.

another two contacts were similarly applied around B, as shown in Fig. 1, in order to measure the contact electrical resistance of the A-B joint during shear. The measured resistance was actually the sum of the contact resistance of the steel-concrete interface, the volume resistance of the steel and the volume resistance of the concrete. However, the two volume resistances were small and did not change during loading, so they were neglected. The four-probe method was used. The outer two contacts (a and d in Fig. 1) were for passing current. The inner two contacts (b and c in Fig. 1) were for voltage measurement. A Keithley 2002 multimeter was used.

Shear stress and contact electrical resistance were simultaneously measured during cyclic loading at different shear stress amplitudes (0.75 and 3.73 MPa). The time for each cycle was 20 s. The curve of stress vs. time within a cycle was a triangle. Six samples were tested at each of the two stress amplitudes.

In order to confirm that an abrupt increase in the contact electrical resistance during cyclic shear is due to degradation of the bond between steel and concrete, the shear bond strength was destructively measured before and after the first abrupt resistance increase during cyclic shear at a stress amplitude of 0.75 MPa. Six samples were tested before the abrupt resistance increase (actually before any cyclic shear) and six samples were tested after the abrupt increase. The shear bond strength was measured during static loading up to failure, using the testing configuration in Fig. 1.

## 2.2. Bond between old and new mortar

The cement used was portland cement (Type I) from Lafarge Corp. (Southfield, MI). The sand used was natural sand (100% passing 2.36 mm sieve, 99.9% SiO<sub>2</sub>). The sand/cement ratio was 1.0. The water/cement ratio was 0.35. A water reducing agent (WR) was used in the amount of 1.0% by weight of cement. The WR was TAMOL SN (Rohm and Haas, Philadelphia, PA) which contained 93–96% sodium salt of a condensed naphthalene sulfonic acid. No coarse aggregate was used. A Hobart mixer with a flat beater was used for mixing, which was conducted for 5 min. After that, the mix was poured into oiled molds. A vibrator was used to facilitate compaction and decrease the amount of air bubbles.

A cylindrical piece of mortar labeled A (poured first) was concentrically surrounded by a cylindrical piece of mortar labeled B (poured 28 days after pouring A), such that the top flat surface of A protruded out of that of B, and the bottom flat surface of A was flush with that of B (Fig. 2). The A-B joint was subjected to shear when B had been cured for 28 days. Shear stress was imposed by applying a downward load on the top flat surface of A, while the bottom flat surface of B was supported by a steel annular ring C with a central circular hole slightly larger than the cross-section of A. In this way, A went through the hole of C upon complete debonding at the A-B joint. The C was electrically insulated from A and B by using a paper lining.

Electrical contacts in the form of silver paint in conjunction with copper wire strands were applied circumferentially around the protruded part of A and around B, as shown in Fig. 2, in order to measure the contact electrical resistance of the A-B joint during shear. The two-probe method rather than the four-probe method was used because of the very high values of the A-B joint contact resistance (of the order of 10 MΩ), which overshadowed the volume resistances of A and B, as well as the contact resistance of the silver paint contacts. A Keithley 2002 multimeter was used.

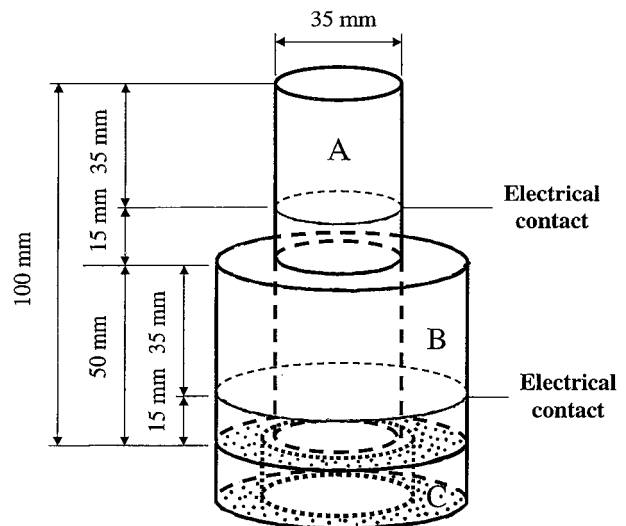


Figure 2 Configuration for measuring contact electrical resistance during shear loading of joint between old mortar (A) and new mortar (B). C was the steel support.

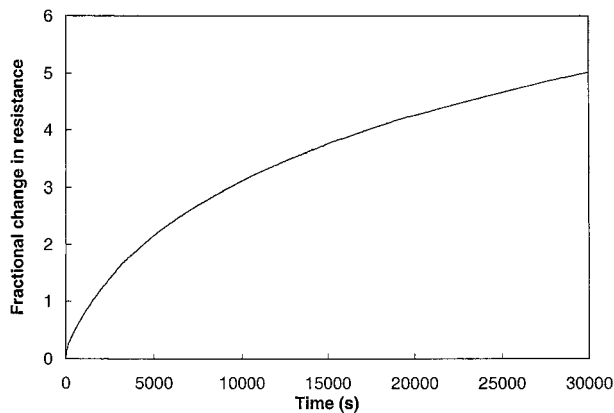


Figure 3 Variation of the fractional contact resistance change with time during continuous resistance measurement.

Shear stress and contact electrical resistance were simultaneously measured during static loading up to failure and during cyclic loading at different shear stress amplitudes (0.81, 0.97 and 1.21 MPa). The time for each cycle was 20 s. The curve of stress vs. time within a cycle was an isosceles triangle (Fig. 6). Six samples were tested for each loading condition. The loading rate during static loading was 0.027 MPa/s.

Due to the voltage present during electrical resistance measurement, electric polarization occurs as the resistance measurement is made continuously. The polarization results in an increase in the measured resistance [27]. This effect contributed negligibly to the observed resistance changes unless the time of continuous resistance measurement was long, as in the case of cyclic loading for more than 100 cycles. (This effect is even smaller when the bond involves steel as a member (Section 2.1), since steel is highly conducting. As a result, polarization was not considered in Section 2.1.) Nevertheless, the polarization-induced resistance increase, as separately measured as a function of the time of resistance measurement in the absence of stress (Fig. 3), was subtracted from the measured resistance change obtained during cyclic loading in order to correct for the effect of polarization.

In order to confirm that an abrupt increase in the contact electrical resistance during cyclic shear is due to degradation of the bond between old and new mortar, the shear bond strength was destructively measured before and after the first abrupt resistance increase during cyclic shear at a stress amplitude of 1.21 MPa. Six samples were tested before the abrupt resistance increase and six samples were tested after the abrupt increase. The shear bond strength was measured during static loading up to failure, using the testing configuration in Fig. 2.

### 3. Results and discussion

#### 3.1. Bond between steel and concrete

Fig. 4 shows the fractional change in contact electrical resistance of the joint between steel and concrete during cyclic shear loading at a shear stress amplitude of 3.73 MPa. The resistance did not change much upon stress cycling except for an abrupt increase after 8–31

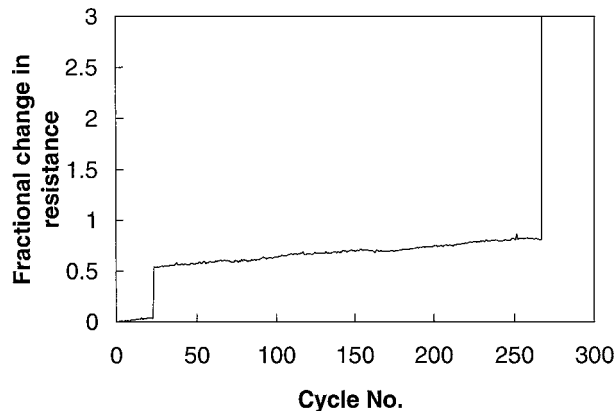


Figure 4 Variation of the fractional contact resistance change ( $\Delta R/R_0$ ) with cycle No. during cyclic shear loading of a steel-concrete joint at a shear stress amplitude of 3.73 MPa up to bond failure.

cycles (the particular cycle depending on the sample), when there was no visual sign of damage, and another abrupt increase at bond failure (steel pull-out), which occurred at cycle 220–270 (the particular cycle depending on the sample).

Fig. 5 shows the fractional change in contact electrical resistance during cyclic shear loading at a shear stress amplitude of 0.75 MPa. The resistance abruptly increased after 150–210 cycles (depending on the sample), due to bond degradation, which was not visually observable. Bond failure did not occur up to 400 cycles, at which testing was stopped. The bond strength before any cyclic shear was  $6.68 \pm 0.24$  MPa; that after the abrupt increase (at the end of 400 cycles in Fig. 5) was  $5.54 \pm 0.43$  MPa. Thus, even though the abrupt increase did not cause visually observable damage, bond degradation occurred.

Comparison of Figs 4 and 5 shows that a higher stress amplitude caused bond degradation and bond failure to occur at lower numbers of cycles, as expected.

The focus of this paper is bond degradation rather than bond failure. Bond failure is a relatively drastic process that is affected by the surface deformation on the steel rebar and is dependent on the failure mode. However, bond degradation is a relatively subtle

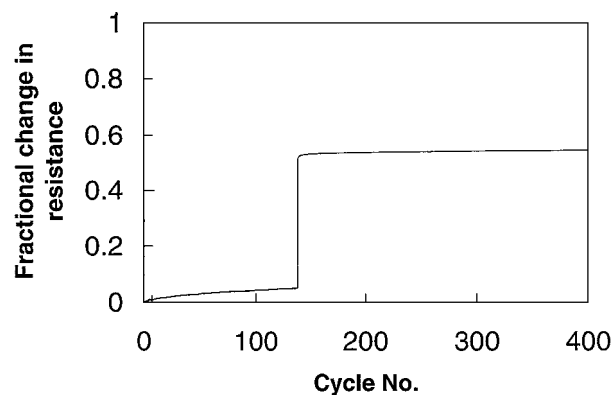


Figure 5 Variation of the fractional contact resistance change ( $\Delta R/R_0$ ) with cycle No. during cyclic shear loading of a steel-concrete joint at a shear stress amplitude of 0.75 MPa. The test was stopped prior to bond failure.

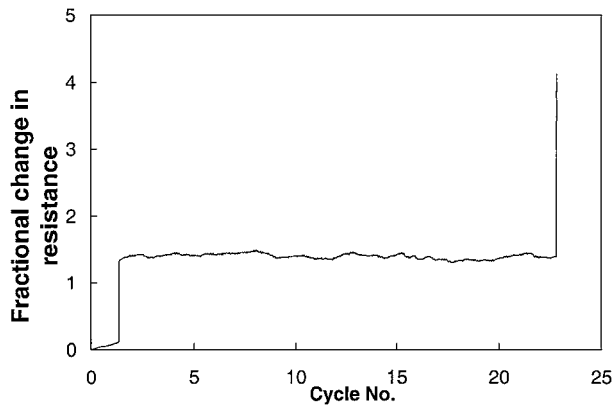


Figure 6 Variation of the fractional contact resistance change with cycle No. during cyclic shear loading of a joint between old mortar and new mortar at a shear stress amplitude of 1.21 MPa up to bond failure. Thick curve: fractional change in contact resistance. Thin curve: shear stress.

process that is expected to be less dependent on the rebar surface deformation or the failure mode.

The abrupt increase in resistance due to bond degradation (not bond failure) (Figs. 4 and 5) provides a method of monitoring bond quality nondestructively in real time during dynamic loading. In contrast, bond strength measurement by mechanical testing is destructive. The bond degradation is attributed to fatigue.

### 3.2. Bond between old and new mortar

Fig. 6 shows the fractional change in contact electrical resistance of the joint between old and new mortar during cyclic shear loading at a shear stress amplitude of 1.21 MPa. The resistance did not change upon stress cycling except for an abrupt increase after 1–6 cycles (the particular cycle depending on the sample), when there was no visual sign of damage, and another abrupt increase at bond failure, which occurred at cycle 18–27 (the particular cycle depending on the sample).

The bond strength before the first abrupt increase was  $2.87 \pm 0.18$  MPa; that after the first abrupt increase was  $2.38 \pm 0.22$  MPa. Thus, even though the first abrupt increase did not cause visually observable damage, bond degradation occurred.

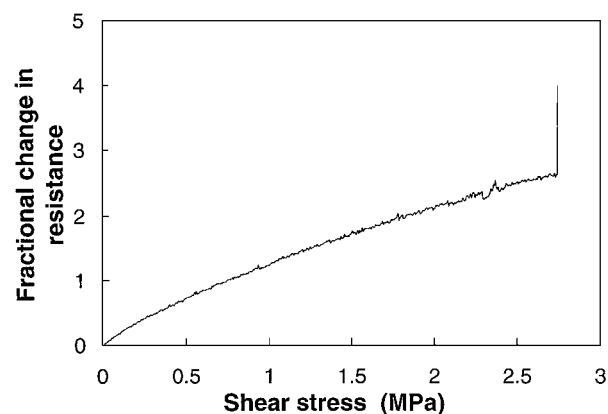


Figure 7 Variation of the fractional contact resistance change with shear stress during static shear loading of a joint between old mortar and new mortar up to failure.

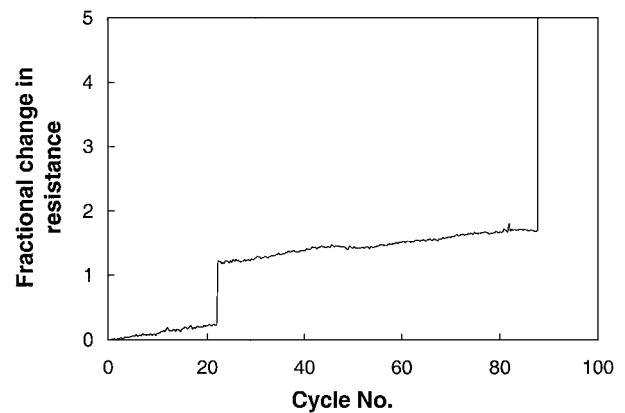


Figure 8 Variation of the fractional contact resistance change with cycle No. during cyclic shear loading of a joint between old mortar and new mortar at a shear stress amplitude of 0.97 MPa up to bond failure.

During static loading, the contact resistance increased monotonically with increasing shear stress and abruptly increased at bond failure, as shown in Fig. 7 for the case of a specimen which had not been loaded prior to the measurement. No abrupt increase in resistance was observed during static loading prior to failure, in contrast to the observation of an abrupt increase prior to fatigue failure (Fig. 6).

Fig. 8 shows the fractional change in contact electrical resistance during cyclic shear loading at a shear stress amplitude of 0.97 MPa (lower than that of Fig. 6). The resistance showed the first abrupt increase after 22–48 cycles (the particular cycle depending on the sample), and another abrupt increase at bond failure, which occurred after 69–92 cycles (the particular cycle depending on the sample).

Fig. 9 shows the fractional change in contact electrical resistance during cyclic shear loading at a shear stress amplitude of 0.81 MPa (lower than that of Fig. 8). The resistance abruptly increased after 557–690 cycles (depending on the sample), due to bond degradation, which was not visually observable. Bond failure did not occur up to 1300 cycles, at which testing was stopped.

Comparison of Figs 6, 8 and 9 shows that a higher stress amplitude caused bond degradation and bond failure to occur at lower numbers of cycles, as expected.

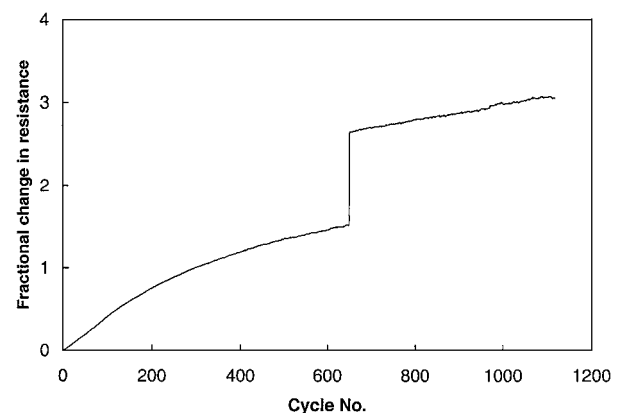


Figure 9 Variation of the fractional contact resistance change with cycle No. during cyclic shear loading of a joint between old mortar and new mortar at a shear stress amplitude of 0.81 MPa. The test was stopped prior to bond failure.

The abrupt increase in resistance due to bond degradation (not bond failure) (Figs 6, 8 and 9) provides a method of monitoring bond quality nondestructively in real time during dynamic loading. In contrast, bond strength measurement by mechanical testing is destructive. The bond degradation is attributed to fatigue. This interpretation is consistent with the absence of an abrupt resistance increase during static loading prior to failure.

#### 4. Conclusion

Degradation of the bond between steel rebar and concrete and that between old and new mortar under cyclic shear loading was observed nondestructively by measuring the contact electrical resistance of the joint. Degradation due to fatigue and causing decrease in bond strength, though causing no visually observable damage, was indicated by an abrupt increase in the resistance. It occurred at a small fraction of the fatigue life. Bond failure was also accompanied by an abrupt increase in resistance.

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