Pergamon

Cement and Concrete Research 31 (2001) 669–671

CEMENT AND CONCRETE RESEARCH

Communication

Degradation of the bond between concrete and steel under cyclic shear loading, monitored by contact electrical resistance measurement

Jingyao Cao, D.D.L. Chung*

Composite Materials Research Laboratory, State University of New York at Buffalo, Buffalo, NY 14260-4400, USA

Received 10 October 2000; accepted 18 January 2001

Abstract

Degradation of the bond between steel reinforcing bar (rebar) and concrete 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. © 2001 Elsevier Science Ltd. All rights reserved.

Keywords: Bond strength; Electrical properties; Reinforcement; Degradation; Fatigue

1. Introduction

Steel-reinforced concrete is a widely used structural material. The effectiveness of the steel reinforcement depends on the bond between the steel reinforcing bar (rebar) and the concrete. Destructive measurement of the shear bond strength by pull-out, push-in, and related testing methods is commonly used to assess the quality of the bond [1–15]. Nondestructive methods of bond assessment are attractive for condition evaluation in the field. They include acoustic [16–18] and electrical [19] 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 [19]. 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.

* Corresponding author. Department of Mechanical and Aerospace Engineering, State University of New York at Buffalo, 608 Furnas Hall, Buffalo, NY 14260-4400, USA. Tel.: +1-716-645-2593 ext. 2243; fax: +1-716-645-3875.

E-mail address: ddlchung@acsu.buffalo.edu (D.D.L. Chung).

2. Experimental methods

The cement used was Portland cement (Type I) from Lafarge (Southfield, MI). Both fine and coarse aggregates were used. The fine aggregate was natural sand (99.9% SiO_2), 100% of which passed no. 8 U.S. sieve. The coarse aggregate was no. 57 (ASTM C33-84), 100% of which passed 25 mm (1 in.) standard sieve. The ratio of cement/fine aggregate/coarse aggregate was 1:1.5:2.5.

The water/cement ratio was 0.45. A water-reducing agent (TAMOL SN, Rohm and Hass, Philadelphis, PA; sodium salt of a condensed naphthalenesulfonic 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 no. 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

0008-8846/01/$ – see front matter
PII: S0008-8846(01)00462-8

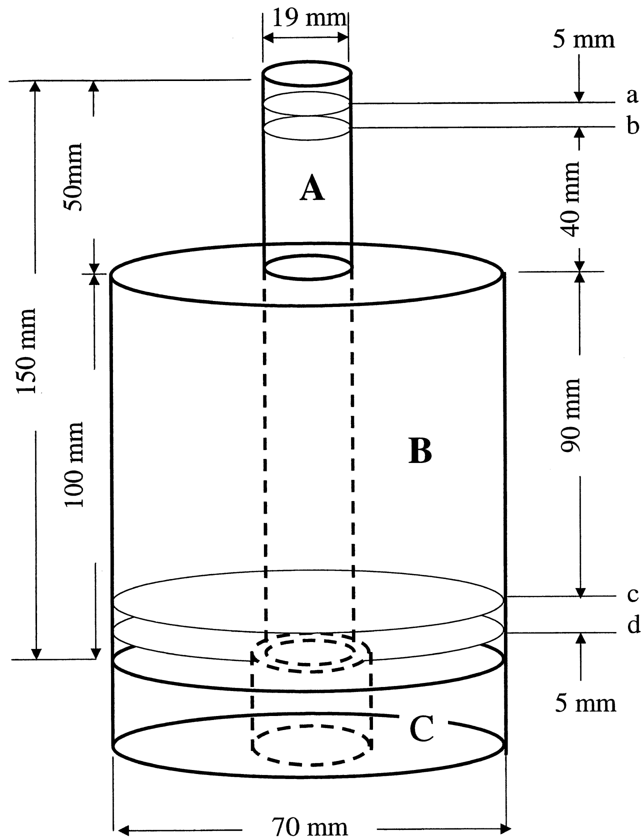

Fig. 1. Configuration for measuring contact electrical resistance during shear loading of joint between steel rebar (A) and concrete (B). Steel support (C).

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. 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 another two contacts were similarly applied around B, as shown in Fig. 1, in order to measure the DC 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 and 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 for the resistance measurement.

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

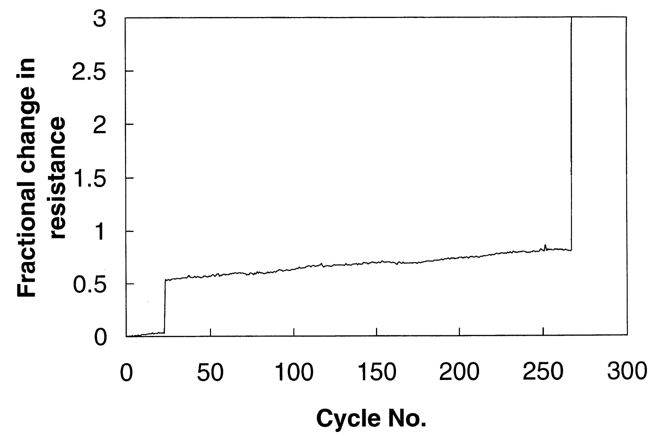

Fig. 2. Variation of the fractional contact resistance change ($\Delta R/R_o$) with cycle number during cyclic shear loading at a shear stress amplitude of 3.73 MPa up to bond failure.

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.

3. Results and discussion

Fig. 2 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.

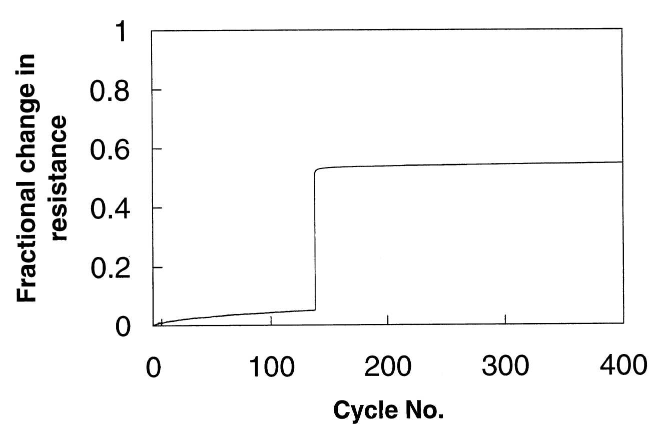

Fig. 3. Variation of the fractional contact resistance change ($\Delta R/R_o$) with cycle number during cyclic shear loading at a shear stress amplitude of 0.75 MPa. The test was stopped prior to bond failure.

The resistance did not change much upon stress cycling except for an abrupt increase after 8–31 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 cycles 220–270 (the particular cycle depending on the sample).

Fig. 3 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 ± 0.24 MPa; that after the abrupt increase (at the end of 400 cycles in Fig. 3) was 5.54 ± 0.43 MPa. Thus, even though the abrupt increase did not cause visually observable damage, bond degradation occurred.

Comparison of Figs. 2 and 3 shows that a higher stress amplitude caused bond degradation and bond failure to occur at lower numbers of cycles, as expected.

The abrupt increase in resistance due to bond degradation (not bond failure; Figs. 2 and 3) 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.

4. Conclusion

Degradation of the bond between steel rebar and concrete 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.

Acknowledgments

This work was supported in part by Department of Transportation, U.S.A. via D'Appolonia & Associates.

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