Structural Engineering and Mechanics, *Vol. 47, No. 6 (2013) 773-790*
DOI: http://dx.doi.org/10.12989/sem.2013.47.6.773

Bending characteristics of corroded reinforced concrete beam under repeated loading

Congqi Fang*[1], Shuai Yang[1] and Zhang Zhang[2]

[1]Department of Civil Engineering, Shanghai Jiaotong University, Shanghai 200240, China
[2]Shanghai Third Harbour Engineering Science & Technology Research Institute CO., LTD, Shanghai 200030, China

(Received September 2, 2012, Revised August 24, 2013, Accepted August 31, 2013)

Abstract. Bending behaviors of corroded reinforced concrete (RC) beams under repeated loading were investigated experimentally. A total of twenty test specimens, including four non-corrosion and sixteen corrosion reinforced concrete beams, were prepared and tested. A numerical model for flexural and cracking behaviors of the beam under repeated loading was also developed. Effects of steel corrosion on reinforced concrete beams regarding cracking, mid-span deflection, stiffness and bearing capacity of corroded beams were studied. The impact of corrosion on bond strength as the key factor was investigated to develop the computational model of flexural capacity. It was shown from the experimental results that the bond strength between reinforcement and concrete had increased for specimen of low corrosion levels, while this effect was changed when the corrosion level was higher. It was indicated that the bearing capacity of corrosion beam increased even at a corrosion level of about 5%.

Keywords: steel corrosion; concrete beam; repeated loading; cyclic loading; bending

1. Introduction

Reinforcement corrosion, with cracking and spalling of concrete, is one of the major durability problems of reinforced concrete structure and by far outweighs other forms of deterioration. Corrosion may affect: (a) the steel, due to the reduction of both the steel cross-section and mechanical properties; (b) the concrete, due to the expansion of corrosion products, cracking, splitting and even delaminating; (c) the bond strength between steel and concrete, due to the accumulation of corrosion products around the steel, deteriorating or completely loss.

A number of studies have investigated the effects of corrosion on the steel-concrete bond (Al-Sulaimani *et al*. 1990, Auyeung *et al*. 2000, Fang *et al*. 2004), and the effects on structural behavior (Castel *et al*. 1998, Mangat *et al*. 1999). The weakening of the bond between concrete and rebar together with the reduced cross-sectional area of steels is a significant factor affecting the serviceability and the flexural strength of concrete beams (Cabrera 1996, Stanish *et al*. 1999). The effect of loss of the reinforcing steel area on the flexural strength and the effect of corrosion on the bond between reinforcing steel and concrete have been studied by various researchers (Almusallam 2001, Berra *et al*. 2003). The residual confinement pressure given by the cracked

*Corresponding author, Associate Professor, E-mail: cqfang@sjtu.edu.cn

Copyright © 2013 Techno-Press, Ltd.
http://www.techno-press.org/?journal=sem&subpage=8 ISSN: 1225-4568 (Print), 1598-6217 (Online)

concrete around the rebar was given from the tests of Baldwin and Clark (1995) after the splitting of concrete cover. However, relatively little attention has been devoted to the load-deflections and serviceability failure, especially when considering a time-varying process. On the other hand, improving the understanding of the influence of the percentage of corrosion and service loading on the load-deflection curve would assist engineers insight into design, condition evaluation, and service-life prediction of corroded reinforced concrete (RC) beams under service loading.

Recently, a number of studies have investigated the deterioration of flexural strength of reinforced concrete beams, and the effect of corrosion on the load-bearing capacity of reinforced concrete beams under monotonic loading has been studied by various researchers. For example, Azad *et al*. (2010) improved further the accuracy of the analytical prediction method of residual flexural strength of corroded beams. The flexural behaviors of localized and uniform corrosion of rebar in beams have been investigated by Dekoster *et al*. (2003), and pitting corrosion of rebar in beams have been studied by Stewart (2009). In real structures, the corrosion takes place while the structure carries service loading and the two effects act synergistically to accelerate the deterioration of the structure such as bridges and oil production platforms. Here we may simplify service loading to repeating loading in experimental study. However, very few researchers have studied the effect of repeated loading on the corroded concrete beams. Al-Hammoud *et al*. (2010) investigated the fatigue bond behavior of corroded reinforced concrete beams under monotonic and fatigue loads. Balogh *et al*. (2008) found that the reduction in stiffness and in load capacity due to sustained loading was greater than that due to the cyclic loading. The results obtained by Ballim *et al*. (2003) show the importance of assessing the structural effects of reinforcement corrosion under simultaneous load and corrosion conditions and assessing the effects of reinforcement corrosion on serviceability deflections of RC beams. The appropriate assessment of the actual bearing capacity of the existing structure demands the development of calculation model, and the investigation of the effect of corrosion on deformation, cracking, stiffness, bearing and failure mode of the corroded RC beams.

Although considerable researches had been conducted on the bond behavior in reinforced concrete with significant damage due to reinforcement corrosion, little work has been done on both corroded reinforcement and repeated loading. Lack of experimental data on bond between corroded steel bars with concrete under repeated loading often leads to a conservative design of the structures. The objective of this study, therefore, was to assess the effect of corrosion on the bond strength degradation between corroded steel bar and concrete under repeated loading. Data were required to understand the relationship among the degree of reinforcement corrosion, repeated loading condition and the bond strength.

2 Experimental program

2.1 Specimens

Ordinary Portland cement, fine aggregate (medium-sized natural sand), and coarse aggregate (crushed limestone, 5~20 mm in diameter) with a water/cement ratio of 0.51 were used to prepare the concrete mixture. The geometry and reinforcement of specimens are indicated in Fig. 1. A total of twenty sample beams (120×180×1800 mm) were cast and cured for 28 days. Concrete cubes (100×100×100 mm) were also cast for the concrete compressive strength. The average concrete

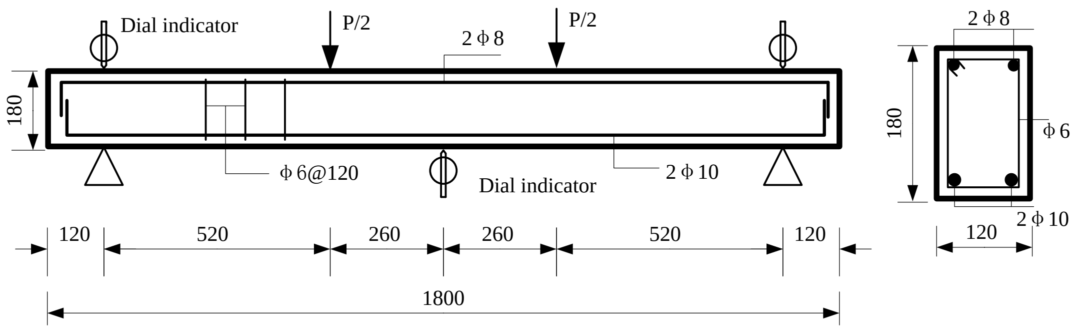


Fig. 1 Details of the experiment beam. All dimension in mm

strength was 23.56MPa. HRB335 deformed hot-rolled steel were used for longitudinal tensile steel bar and top steel bar. HPB235 smooth steel bar was used for stirrups. The concrete cover was 18mm. The bars were descaled and cleaned before being cast into the concrete beams.

2.2 Corrosion of sample RC beams

An electrolyte corrosion technique was used to accelerate steel corrosion. Only the main tensile steel bars at the bottom of the beam were corroded in this test. The stirrups were wrapped up in plastic and coated with insulating tape to prevent stirrups and compressive bars from corroding. The cured beams were fully immersed in a 5% NaCl solution in a plastic tank for 72 hours before direct current was applied to the steel bars, as shown in Fig. 2. Electric current and the duration of exposure were used for theoretical estimation of the mass loss of steel due to corrosion according to Faraday's Law. In Fig. 2, power supplies with adjustable voltage and direct current (from 0 to 2 A) were chosen for the electrolyte corrosion process. The direct current was impressed on the longitudinal main steel bars 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. It should be noted that the current was impressed on the two longitudinal main steel bars at the same time to ensure the electric potential at the bar ends were the same for the two main bars. The direction of the electric current was such that the reinforcing steel served as the anode while a stainless steel plate counter-electrode was positioned in the tank to act as a cathode. The stainless steel plate that served as the cathode consumed the excess electrons given off by the reinforcement during the corrosion process. After the power supply was switched on, the current was adjusted and fixed to a volume, and the voltage was adjusted automatically by the constant current supply. Constant current of a suitable value was chosen according to the actual range of the displayed voltages.

2.3 Loading

Before the normal loading of the corroded beam, cracking load of the corresponding non-corrosion beam was calculated for the loading steps. The load for each step was chosen according to the theoretical computed cracking load and the yielding strength of reinforcement. As showed in Fig. 1, deflection of the beam was measured by dial indicators. Data measured in the

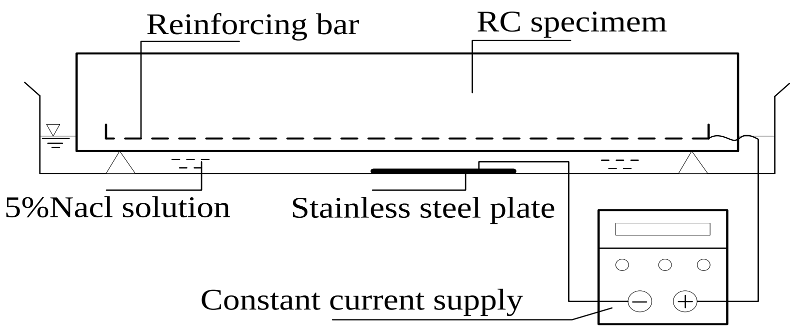


Fig. 2 Corrosion setup

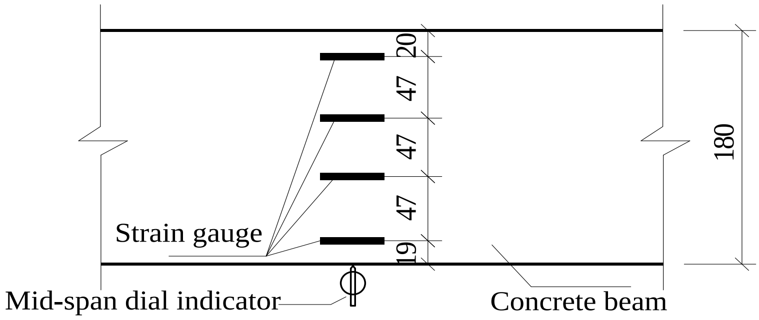


Fig. 3 Location of concrete strain

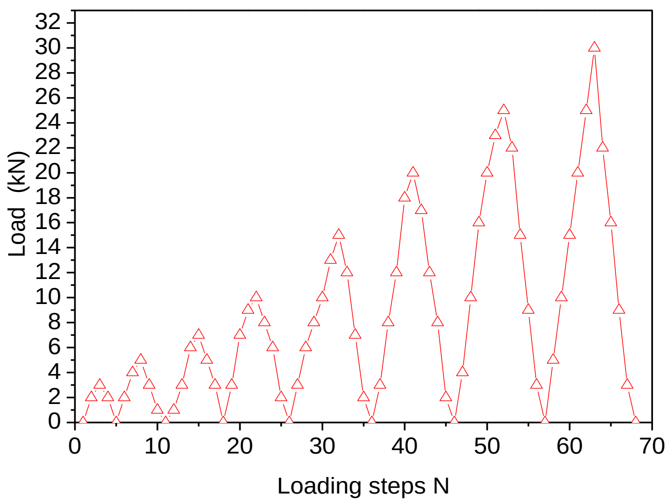


Fig. 4 Loading steps of reinforced concrete beams

strain gauges were imputed to strain measuring instruments and transferred to a computer. Loading sketch of the experiments is showed in Fig. 1. Concrete strains were installed at the 4 points arranged uniformly along the depth of the beam at the mid-span as showed in Fig. 3. Strain gauges (not shown) were installed on the tensile reinforcement to measure the actual strain of the reinforcement during loading. Dial indicators were used to measure the displacement of the bearings of the beam. An electronic displacement gauge was installed to measure the mid-span deflection of the beam.

Repeated loading is different from cyclic loading in that one cycle of loading consists of two phases: loading - unloading, i.e., loading phase followed by unloading phase. While one cycle of cyclic loading consists of four phases: loading - unloading - reversed loading - unloading. In the current study repeated loading were applied to a total of 20 specimens for bending characteristics of corroded beams. Preloading was conducted in order to make sure the test setup was in normal condition and then normal loading was applied to the beams. The applied loads for each loading step were showed in Fig. 4, where N is the number of loading steps. The period of one loading or unloading steps was about 10 minutes. The loading steps, load value, crack widths, readings from dial indicators as well as the failure modes of the beam were recorded manually. Once one or more of the following conditions occurred the tested beam was considered to be in a limit state or over limit state: the reinforcement yielded, the maximum deflection exceeded 1/50 of the beam span, maximum crack width at the tension reinforcement reached 1.5 mm, and concrete in compressive region crushed. When the limit state was reached, the applied load of the previous step was defined as the maximum limit state load, or the ultimate load carrying capacity of the beam.

3 Bond behavior of corroded RC beams under repeated loading

3.1 Degradation of flexural capacity

3.1.1 Load-deflection analysis of corroded RC beams under repeated loading

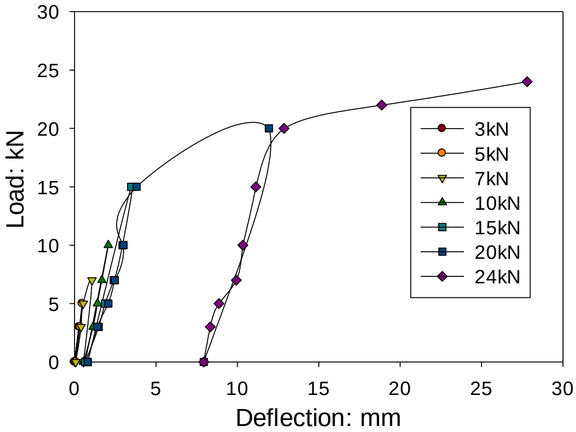
Some typical load-deflection curves of corroded RC beams under repeated loading are shown in Fig. 5. Figs. 5(a), 5(b), 5(c) and 5(d) are load-deflection curves for beams No. 1 (non-corrosion), No. 6 (corrosion level 1.77%), No. 9 (corrosion level 3.57%) and No. 19 (corrosion level 11.72%), respectively. Results from the tests showed linear load-deflection relationship when the load was low, and the deflection was returned to zero when the applied loads were totally released. In this stage when the applied loads were less than 6 kN, the deformation of the beam was in elastic state.

I. Comparative analysis of beam Nos. 1 and 6

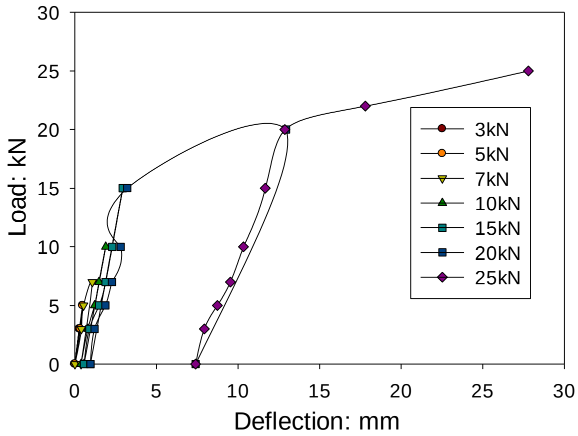
As the applied load was increased to 6 kN, concrete in the compression region began crushing, and a turning point on the load-deflection curve was observed. The deflection was increased rapidly thereafter with the increase of load. Residual deflection, although very small at this stage, existed even if the applied load was entirely released. It was demonstrated that the deformation of the beam was of elastic in nature when the applied load was less than 6 kN. However, as the applied load was increased, the deformation of the beam began to reach elastic-plastic state where plastic deformation existed. The load value corresponding to the turning point from elastic to plastic for non-corrosion beam was less than that for corrosion beam. It was demonstrated that cracking load for non-corrosion beam is less than that for corrosion beam.

As the applied load was increased to around 19kN, the second turning point on the load-deflection curve appeared. At this stage, the reinforcement steel began to yield and the deflection increased sharply. In addition, the deflection did not vanish after unloading. The load value corresponding to the turning point of non-corrosion beam was a little greater compared with that of corrosion one. This showed that non-corrosion tensile reinforcement yielded faster than the corrosion one. An explanation for this was the reduction of effective cross sectional area and the deterioration of mechanical property of a corroded steel bar, i.e., the yield load is lower than the non-corrosion one.

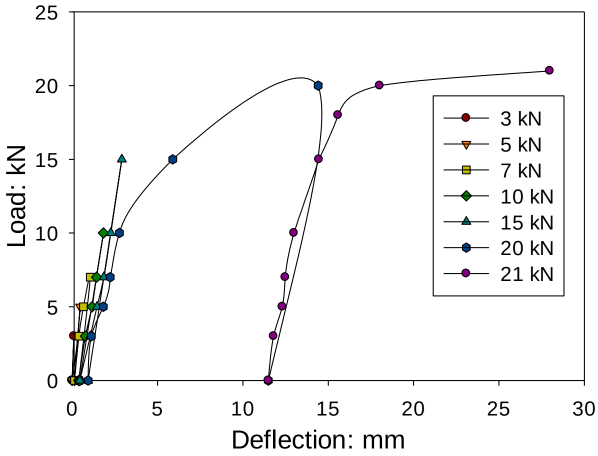
The failure loads of the non-corrosion beam and 1.77% corrosion beam were 24.6kN and 27.1kN, respectively. When the corrosion of reinforcement steel was at a low level, cross sectional areas of the steel bar was decreased to some degree, but the load bearing capacity was greater than those of non-corrosion reinforced concrete beams. This may be owing to the expansion of corrosion products that increased the bond between steel and the surrounding concrete, and the load-bearing capacity of the beam. The area of load-deflection curve of the beams increased gradually with corrosion level for specimen of low corrosion levels, suggesting that energy dissipation capacity and ductility of beams was enhanced.



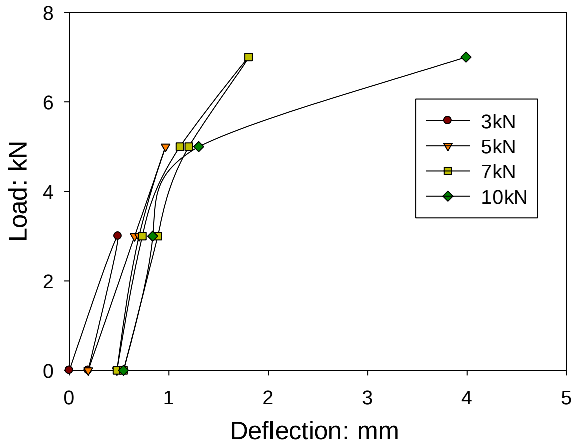
(a) Beam 1



(b) Beam 6



(c) Beam 9



(d) Beam 19

Fig. 5 Load-deflection curves of RC beams

II. Comparative analysis of beams Nos. 6 and 9

As the applied load was increased to about 6kN, the turning point in the load-deflection curve for No. 9 beam (corrosion level 3.57%) was seen. Concrete in the compressive region cracked at a load lower than that of No. 6 beam with 1.77% corrosion level. A significant weakening of deformation recovery capacity was noticed after completion of the 20kN loading - unloading test. It was seen that the residual deformation of No. 9 beam was greater than that of No. 6 beam. Compared with No. 6 beam, No. 9 beam cracked earlier again under 19kN loading. It is shown that the yield strength of steel in No. 9 beam was lower than that of No. 6 beam.

When corrosion level increased to a certain level, both of cross sectional areas of steel bar decreased and mechanical behavior deteriorated. This process was accompanied by bond stress decrease and weakening of cooperative work between steel bar and concrete, and then resulted in bearing capacity decrease of beams. The viewpoint was verified by results of the experiment: The failure load of No. 9 beam was 23.4kN, which was less than that of No. 6 beam.

III. Discussion of No. 19 beam

The failure process of No. 19 beam (corrosion level 11.72%) is shown in Fig. 5(d). The deflection of the beam increased drastically, and concrete in the tensile zone cracked as the applied load increased. Meanwhile, most part of deformation in this stage could not be recovered. After the load reached 7kN, the deflection increased drastically, and not tensile concrete but reinforcement trended to yield first before the final failure of the beam. The failure load was only 9.5kN.

Corrosion products occupy a larger volume so that their continued production applies tensile stresses on the surrounding concrete, which causes corrosive cracking along the length of steel bar and eventually cracking of the cover concrete. It was observed in test that there was a larger amount of corrosion products on cracks than on no cracks, and corrosion was severe and uneven greatly. Severe non-uniform corrosion could cause brittle fracture of steel bar. Therefore, it is difficult to give formula for the bearing capacity for beams of higher corrosion levels currently.

The failure mode for beams of higher corrosion levels was of brittle in nature for longitudinal reinforcements in shear-moment regions of the beam. Because of the low thickness of concrete cover, stirrups hooped outside of longitudinal reinforcements were corroded deeply and then lead to shear failure of RC beams.

IV. The mechanism of bond deterioration

In the initial stages of corrosion, the friction between steel reinforcement and concrete increases because the corrosion products improve the friction coefficient at the interface. The second reason is that accumulating corrosion products expand and occupy a larger volume, and then could carry greater grip of surrounding concrete. But at higher levels of corrosion, a weak layer of corrosion products created that will make the adhesion between the concrete and the corroded steel bars be destroyed and friction decrease. In addition, the deterioration of the ribs of the deformed bars causes a significant reduction of the interlocking forces between the bars and the surrounding concrete. This reduction weakens the primary mechanism of the bond strength between the deformed bars and concrete, and hence, the bond strength decreases significantly. On the other hand, corrosion of reinforcing bars may cause cracking, spalling of the concrete cover. Therefore, restraint of bars given by the concrete cover may be reduced by partial loss of cover. The test results illustrated that both the bond strength and bond stiffness decreases considerably when corrosion reaches high levels.

Table 1 Failure load of specimens

Beam Number	Corrosion level (%)	Failure load (KN)
1	0	24.6
2	0	26.3
3	0	25.7
4	0	25.2
5	1.23	26.8
6	1.77	27.1
7	2.43	27.3
8	2.92	26.9
9	3.57	23.4
10	5.08	24.7
11	5.94	24.4
12	6.72	24.9
13	6.91	25.1
14	7.87	23.8
15	8.56	23.1
16	9.78	19.3
17	9.97	10.4
18	10.3	15.2
19	11.72	9.5
20	12.19	9.2

3.1.2 Effects of corrosion on bearing capacity

The failure loads of the 20 tested RC beams with different corrosion levels were obtained from the experiment, as shown in Table 1. The relationship between the bearing capacity of beam and the corrosion level of steel bar was established based on regression analysis with corrosion level reduction coefficient.

The flexural capacity is presented as a reduction coefficient η which is defined as the ratio of flexural capacity at the corrosion level to the original flexural capacity for non-corrosion specimen. Having average value of the bearing capacities of 4 tested non-corrosion RC beams as a benchmark and corrosion level as independent variable, subsection function of corrosion level reduction coefficient was established. By introducing a simple quadratic function and using a sectional fitting method for test data fitting processing, the corrosion level reduction coefficient of corroded RC beams was calculated as Eq. (1).

$$\eta = \begin{cases} 0.0513\rho + 1 & \rho \leq 2 \\ 0.0140\rho^2 - 0.1404\rho + 1.3212 & 2 < \rho \leq 7 \end{cases} \tag{1}$$

where η is the reduction coefficient, and ρ the corrosion level in %. Here η is fitted by descent stage of quadratic function for the corrosion levels greater than 2%, and the effect data in Eq. (1) ranges up to 7% corrosion.

The flexural capacity of corroded RC beams is calculated as Eq. (2).

$$M_u = \eta M_{u0} \tag{2}$$

Table 2 Comparison between test and calculated bending strength

Test beams	Corrosion level (%)	Test value (kN·m)	Calculated value (kN·m)
BD1	0.47	10.27	9.9
BD2	0.54	9.53	9.9
BD3	1.21	9.92	10.51
BD4	1.24	9.32	10.53
BD5	1.24	10.27	10.53
BD6	1.27	9.53	10.54
BD7	2.15	9.53	10.73
BD8	2.82	9.07	10.26
BD9	2.83	9.53	10.25
BD10	2.88	8.69	10.22
BD11	3.45	9.53	9.93
BD12	4.14	8.2	9.70
BD13	5.2	8.69	9.60
BD14	6.05	7.89	9.74

where M_u and M_{u0} is the bearing capacity of corroded beams and non-corrosion reinforced concrete beams respectively.

The characteristic point of corrosion effects on bearing capacity was emphasized by subsection function given by Eq. (1). The peak value of reduction coefficient means the flexural capacity of beams reaches the maximum when corrosion level is about 2%.

When the corrosion level is low (less than 2%), the flexural capacity of corroded beam is increased compared with non-corrosion beam. This may be owing to the bond between corroded steel and the surrounding concrete was increased with a small increase of corrosion product for a mild corroded steel bar. The contribution of increase of load-bearing capacity offset and exceeded the decrease of capacity due to decrease of cross sectional areas and mechanical behavior deterioration. However, with the continued increase of the corrosion level (more than 2%) the effect of favorable factors continue to weaken and bond stress, reinforcing steel area and flexural capacity decrease.

3.1.3 Comparative analysis of calculative formula

Based on the comparative study between computational formula and test results from Jin and Zhao (2001)，the flexural capacity of tested beams with different corrosion level were calculated using Eqs. (1) and (2), as shown in Table 2. The average of flexural capacity of tested beams BD1 and BD2, with corrosion levels less than 1%, was seen as the value of non-corrosion beams.

The calculated values are slightly larger than test values since the tested beams BD1 and BD2 were calculated as non-corrosion beams. The results show the tendency of changing of calculated values is consistent with experiment results. .

3.2 Deterioration of cracking moment

The cracking coefficient was introduced to demonstrate the relationship between the cracking moment and the corrosion level. The cracking moment of non-corrosion RC beam with the same

Table 3 Corrosion level and cracking coefficient of RC beams

Beam Number	Corrosion level (%)	Cracking load (KN)	Cracking coefficient
1	0.00	5.70	1
2	0.00	5.67	1
3	0.00	5.72	1
4	0.00	5.62	1
5	1.23	5.69	1.002201673
6	1.77	5.71	1.005724351
7	2.43	5.56	0.979304271
8	2.92	5.57	0.981065610
9	3.57	5.43	0.956406869
10	5.08	5.26	0.926464113
11	5.94	5.25	0.924702774
12	6.72	5.27	0.928225451
13	6.91	5.21	0.917657420
14	7.87	5.22	0.919418758
15	8.56	5.16	0.908850727
16	9.78	5.12	0.901805372
17	9.97	5.15	0.907089388
18	10.30	5.13	0.903566711
19	11.72	5.12	0.901805372
20	12.19	5.11	0.900044033

size, reinforcement and loading conditions was used as standard value, cracking moment of corroded RC beam can be estimated using Eq. (3).

$$M_{cr} = \mu M_{cr}^{0} \tag{3}$$

where M_{cr}^{0} is the cracking moment of non-corrosion beam, and μ the cracking coefficient.

The accurate cracking load was difficult to obtain in experiments, because cracking variations were too tiny to be observed. In this test the cracking load was adopted when the maximum crack width is 0.1mm. The average of cracking load of 4 non-corrosion RC beams was used as a benchmark and the cracking coefficient of non-corrosion RC beams was set as 1. The experimental cracking coefficients of different corrosion levels are shown in Table 3.

Based on experimental data in Table 3, the cracking coefficient was fitted into a function as Eq. (4).

$$\mu = \begin{cases} 0.0028\rho + 1 & \rho \le 2 \\ 0.0015\rho^2 - 0.0289\rho + 1.0463 & 2 < \rho \le 10 \end{cases} \tag{4}$$

where μ is the cracking coefficient, and ρ the corrosion level in %. Here formula is applied in a specific range due to the characteristics of quadratic function.

Proposed numerical models given in Eqs. (3) and (4) provide reasonable estimation for cracking moment. When the corrosion is in the range of 0-2%, cracking moment increases with corrosion level, and the low corrosion level results in improvement of beam stiffness. The sectional area is monotonically decreased with the increase of corrosion level, and that has adverse effect on property of beams. When corrosion level is low the space between steel and the surrounding concrete is filled with the expanded of corrosion products and the bond stress is thus increased while cooperative work capacity between steel and concrete improved. This is beneficial to bearing capacity and stiffness of beam. But the increase in bond stress is limited with corrosion level increasing, and the negative influence of decrease of the sectional area on bearing capacity and stiffness of beam will play a leading role. As corrosion level increases further, expansive pressures from accumulating corrosion products is exerted on the surrounding concrete. The corrosion products lead to cracking of concrete along the steel, which weakens the anchorage of the steel, creating a weak layer of corrosion product that will break off under relatively low stress levels.

3.3 Stiffness degradation

3.3.1 Effects of reduction in reinforcement cross-sectional area

The steel corrosion products are mainly Fe_2O_3 and Fe_3O_4, etc. They are all brittle materials and their tensile strength can be considered as zero. The iron element outside rebar does not work once corroded.

RC beam is in elastic stage before cracking. The whole cross-section of the beam is at work and concrete uniformly deforms together with steel in tensile region. Transformed area of the tension steel is expressed as nA_s, here A_s is cross-sectional area of tension steel, elastic modular ratio n is equal to E_s/E_0, E_s and E_0 are the elastic modulus of tension reinforcement and concrete, respectively.

For a single-side reinforced rectangular beam, section stiffness B_0 under elasticity stage was calculated by inertia moment method using Eqs. (5) and (6) (Guo and Shi 2003).

$$B_0 = E_0 I_0 \tag{5}$$

$$I_0 = \frac{b}{3}[x_0^{\ 3} + (h - x_0)^3] + (n-1)A_s(h_0 - x_0)^2 \tag{6}$$

where x_0 is the height of concrete compression zone, being depended on the condition that the area moment of compression zone to neutral axis is equal to the tension zone, and can be expressed in Eq. (7).

$$x_0 = \frac{\frac{1}{2}bh^2 + (n-1)A_s h_0}{bh + (n-1)A_s} \tag{7}$$

During the whole process of the beam failure under loading, the influence of the reduction steel cross sections to the flexural rigidity of corrosion beam is constant. Therefore, a degeneration factor λ_1 is defined which is related to corrosion level. The degeneration factor is expressed as the impact of steel cross sectional reduction to flexural rigidity calculated by Eq. (8).

$$\lambda_1 = 1 - \rho \tag{8}$$

where ρ is the corrosion level of tensile reinforcement calculated by steel weight loss. A_s in Eq. (6) and Eq. (7) is replaced by the remaining cross-sectional area of corroded steel $\lambda_1 A_s$, and degeneration factor β_1 is the ratio of B_c to B_0, where B_c is the stiffness affected by reinforcement cross-sectional area, see Eq. (9).

$$\beta_1 = B_c / B_0 \tag{9}$$

3.3.2 Effects of bonding degradation

Degradation of bond between steel and concrete is one of the main factors for the degradation of flexural rigidity of corroded RC beam. Once the beam cracks, part of concrete in the tensile region stops working immediately while the neutral axis moves up quickly, and the height of compression zonedecreases rapidly. Meanwhile, there is a non-bonded area in the position of cracking, which leads to different deformation ratio for steel and concrete, and therefore bond stress losses seriously and stiffness of beam decreases sharply.

The degradation of bond between concrete and reinforcing steel due to rebar corrosion has been experimentally studied previously. An increase of experimental bond strength values has been observed with corrosion level up to about 1-4%, and significant reduction has been observed for the corrosion level beyond 4% (Al-Sulaimani *et al*. 1990, Amleh and Mirza 1999, Auyeung *et al*. 2000, Fang *et al*. 2004). The bonding strength ratio (the ratio of bonding strength corrosion component to non-corrosion component) proposed to reflect corrosion level effect on bond strength of RC beam under monotonic loading (Bhargava *et al*. 2007), which can be given by Eq. (10).

$$R = \begin{cases} 1 & \rho \le 1.5\% \\ 1.346e^{-19.8\rho} & \rho \ge 1.5\% \end{cases} \tag{10}$$

3.3.3 Stiffnss of corrosion beam

Cross sectional reduction of steel and bond stress degradation are main factors in flexural rigidity degradation of corrosion RC beam. The geometric dimension and reinforcement ratio are both inherent property of beam. Based on the above discussion, the effect of steel cross sectional reduction is fixed, so once steel corrosion level is determined the effect of steel cross sectional area reduction on the flexural rigidity of corrosion beam can also be determined. Bonding strength ratio in Eq. (10) is only related to steel corrosion level. Consequently, while flexural rigidity degradation calculated by Eqs. (9) and (10), stiffness of corrosion beam is determined by corrosion level. Bonding strength ratio R means the ratio of bonding strength of corrosion beam to non-corrosion one. That means bonding strength accords the linear relation with flexural rigidity which influenced on bond strength.

Degeneration factor λ_1 and bonding strength ratio R are both less than or equal to 1. Therefore the flexural rigidity of corrosion beam under monotone loading can be given as Eq. (11).

$$B_{cor} = \beta_1 R B_0 \tag{11}$$

4 Bearing capacity of corrosion RC beam under cyclic loading

It is well known that the bond between reinforcment steel and the surrounding concrete decreases under repeated loading or cyclic loading. However, the reduction of bond stress under repeated loading or cyclic loading is different from that under static loading. This phenomenon is caused by the difference of loading mode and loading path. The bond strength of corroded RC beam under cyclic loading was calculated based on test results of repeated loading in this paper.

4.1 Influence of bond stress on bearing capacity under repeated loading

Bond stress between reinforcement and concrete is related to loading mode, cyclic number of loading and the control value of loading as well as surface roughness and corrosion level of rebar. In repeated loading, the influence of number of repeating on bearing capacity of beam is very small and can be neglected. However, in cyclic loading the number of loading is significant since the direction of loading and reversed loading is changing, the bond stress is alternately redistributed along the length of beams, and the internal damage is accumulated while the bond strength and stiffness deteriorated with cyclic number.

Bearing capacity of corroded beams is calculated using Eqs. (1) and (2). Neglecting the effect of bond stress, the bearing capacity reduction factor caused by loss of steel area is defined as η'. Considering the influence of loss of steel area and reduction of yield strength on bearing capacity, the values of non-corrosion beam and 100% corrosion beam are taken as $\eta'=1$ and $\eta'=0$, respectively. Concrete being neglected in the calculation of bearing capacity, reduction factor η' follows the elastic distribution in Eq. (12) for the whole range $0\leq \rho \leq 100$

$$\eta' = 1 - 0.01\rho \qquad 0 \leq \rho \leq 100 \tag{12}$$

The bearing capacity of beams under repeated loading can be given as Eq. (13)

$$M_u = \eta M_{u0} = M_{u0} - [(1-\eta')M_{u0} + (1-\eta'')M_{u0}] \tag{13}$$

where η'' is the bearing capacity reduction factor caused by bond stress.
It shoud be noted that the major factors effecting bearing capacity of corroded reinforced concrete beam are loss of corss-sectional area of steel and bond strength. With Eqs. (1), (12) and (13), the bearing capacity reduction factor η'' for different corrosion levels is calculated as Eq. (14)

$$\eta'' = 1 + \eta - \eta' = \begin{cases} 1 + 0.0613\rho & \rho \leq 2 \\ 0.0140\rho^2 - 0.1304\rho + 1.3212 & 2 < \rho \leq 7 \end{cases} \tag{14}$$

In Eq. (14) it is demonstrated that the bearing capacity is imporved with corrosion of a low percentage, i.e., up to 4.6% copmarred with non-corrosion beam. The explanation for this is that bond stress and cooperative work capacity between steel and concrete was increased due to expansion of corrosion products of lower corrosion level steel. Here the range of the corrosion level ρ is obtained from Eqs. (1) and (12), while the corrosion level higher than 7% is not discussed currently.

4.2 Influence of bond stress on bearing capacity under cyclic loading

Based on results from previous pull-out tests, Fang *et al*. (2006) came to conclusion that under constant amplitude cyclic loading, the relationship between bond stress and slip is related to cycle number N and slip displacement S. The ratio of the maximum bond stress in the N th cycle to that in the first cycle is expressed as degree of deterioration of bond strength. The corrosion influence on bond strength is mainly embodied in the initial cycles of loading, especially in the first five cycles. The bond stress decreases rapidly in the initial cyclics, and the reduction weakens with the number of loading cycles. It was concluded that bond stress in the unloading stage is about 3/4 of that in the loading stage of the same cycle.

Considering slip influence on bond stress, the ratio of the actual slip to the maximum value is defined as slip ratio using Eq. (15)

$$k = \frac{S}{S_{\max}} \tag{15}$$

where $S_{\max}$ is related to concrete strength, bar diameter, section shape and corrosion level. The influence of corrosion level on bond stress was considered in Eq. (14), and thus $S_{\max}$ can be determined through pull-out test of non-corrosion specimens. The slip ratio also reflects the level of loading, and can be given as Eq. (16)

$$k = \frac{S}{S_{\max}} = \frac{\sigma}{\sigma_{\max}} \tag{16}$$

where σ is the actual tensile stress of reinforcement, and $\sigma_{\max}$ the maximum tensile stresses of specimen in pull-out test before failure.

A coefficient γ is defined as the loass of bond stress in a single cycle of loading. The characteristic value of γ is 1 and 3/4, with corresponding k as 0 and 1 respectively. The coefficient γ that ranges from 1 to 3/4 is changes linearly with slip ratio k, as given in Eq. (17)

$$\gamma = -\frac{1}{4}k + 1 \tag{17}$$

The paths of bond strength loss of repeated loading and cyclic loading are shown in Figs. 6 and 7 respectively. Repeated loading is different from cyclic loading in that one cycle of repeated loading consists of loading - unloading, while one cycle of cyclic loading consists of loading - unloading - reversed loading - unloading.

The loss of bond stress is mainly due to disaccord of deformation of reinforcement and concrete and then dislocation between them. Fig. 8 shows deformation characteristics of reinforcing steel bars and concretes adherent to the surrounding of bars under positive loading and reversed loading. In pure bending area of beam, different effects on bond stress loss between

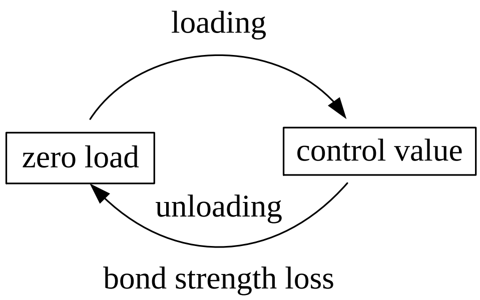


Fig. 6 Bond strength loss path under repeated loading

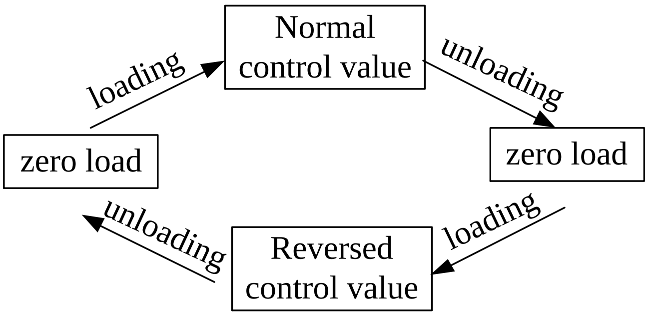


Fig. 7 Bond strength loss path under cyclic loading

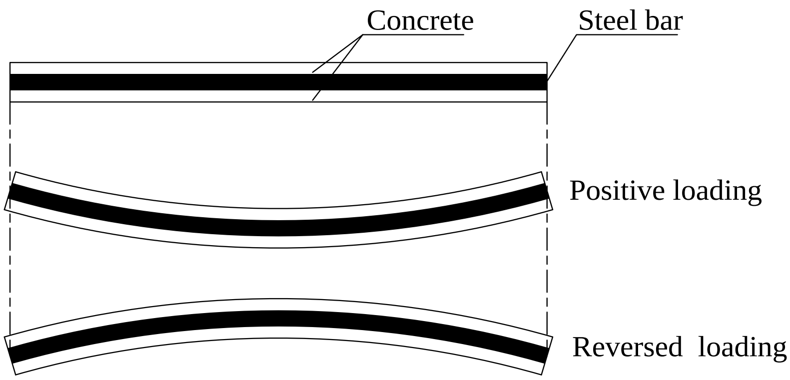


Fig. 8 Deformation characteristics of concrete and reinforcement

compression and tension of steel bars are neglected. Then the loss of bond stress between reinforcement and concrete under positive or reversed loading is the same when the load control value is the same.

Now, considering effects of cyclic number of loading and unloading, the single-cycle loss coefficient of bond stress γ is used, so the total loss coefficient of bond stress under repeated loading ς is given as Eq. (18).

$$\zeta = \gamma^N = (-\frac{1}{4}k + 1)^N \tag{18}$$

where N is the cyclic number of loading and unloading. Considering the effect of corrosion level ρ in Eq. (16), here ς relates to loading value and cyclic number, but unrelated to ρ.

Therefore, reduction factor of bearing capacity caused by bonding stress diversification under repeated loading can be given as Eq. (19).

$$\lambda = \eta'' \zeta \tag{19}$$

where λ is reduction factor of bearing capacity considering the effect of loading value and cyclic number on bond stress loss under repeated loading.

4.3 The bearing capacity formula of corroded reinforced concrete beams under repeated loading

Considering reduction of steel area and bond stress under repeated loading, the bearing capacity of corroded reinforced concrete beams under repeated loading M_u can be given as Eq. (20).

$$M_u = M_{u0} - [(1-\lambda)M_{u0} + (1-\eta')M_{u0}] = (\lambda + \eta' - 1)M_{u0} \tag{20}$$

5. Conclusions

Bending behaviors of corroded reinforced concrete beams under repeated loading were studied based on results from experiments. Bearing capacity of corroded beams under repeated loading and cyclic loading was investigated. The following findings and conclusions are obtained:

(1) A numerical model for bending bearing capacity of corrosion beams under repeated loading was proposed, depending on experimental results of twenty sample beams. Effect of steel corrosion on bearing capacity of corroded beams showed that bond stress was increased and this was beneficial to bearing capacity of beams when corrosion level was less than 2%; bearing capacity was decreased as corrosion level when the corrosion level was higher than 2%.

(2) A numerical model for cracking bending moment of corrosion beams under repeated loading was proposed. Bond stress between reinforcement and concrete was increased in low corrosion levels, which was beneficial to the cracking properties of the beam. When the corrosion level was higher, the beneficial effects gradually disappeared and the further increase of corrosion level adversely affecting the cracking. Once the reinforced concrete beam was cracking under repeated loading, bond stress should lose seriously and flexural stiffness should decline sharply.

(3) The bearing capacity reduction coefficient of corrosion beams were calculated for the effects of bond stress under repeated loading and cyclic loading, respectively. Comparing with non-corrosion reinforced concrete beam, the bending bearing capacity of corroded beams impacted by the bond stress still increases even at a corrosion level up to 5%, while this effect was changed when the corrosion level was higher.

(4) In mild reinforcement corrosion, the bearing capacity of corrosion beam grows in a linear fashion as the bond stress increases.

Acknowledgments

The authors gratefully acknowledge the support provided by the National Natural Science Foundation of China (Granted No. 51048001 & No. 51178264) and the Key Project of Technology Research and Development Program of the National Railway Ministry of China (Grant No. J2011G003).

Reference

Al-Hammoud, R., Soudki, K. and Topper, T.H. (2010), “Bond analysis of corroded reinforced concrete beams under monotonic and fatigue loads", *Cem. Concr. Compos*., **32**, 194-203.

Almusallam, A.A. (2001), "Effect of degree of corrosion on the properties of reinforcing steel bars", *Constr. Build. Mater*., **15**, 361-368.

Almusallam A.A. Al-Gahtani A.S., Aziz A.R. and Rasheeduzzafart (1996), "Effect of reinforcement corrosion on bond strength", *Constr. Build. Mater*., **10**(2), 123-129.

Al-sulaimani, G.J., Kaleemullah, M., Basunbul, I.A. and Rasheeduzzafar. (1990), "Influence of corrosion and cracking on bond behaviour and strength of reinforced concrete member", *ACI Struct. J*., **87**(2), 220-231.

Amleh, L. and Mirza, S. (1999), "Corrosion influence on bond between steel and concrete", *ACI Struct. J*., **96**(3), 415-423.

Auyeung, Y.B., Balaguru, P. and Chung, L. (2000), "Bond behavior of corroded reinforcement bars", *ACI Mater. J*., **97**(2), 214-220.

Azad, A.K., Ahmad, S. and Al-Gohi, B.H.A. (2010), "Flexural strength of corroded reinforced concrete beams", *Mag. Concr. Res*., **62**(6), 405-414.

Azad, A.K., Ahmad, S. and Azher, S.A. (2007), "Residual strength of corrosion-damaged reinforced concrete beams", *ACI Mater. J*., **104**(1), 40-47.

Baldwin, M.I. and Clark, L.A. (1995), "The assessment of reinforcing bars with inadequate anchorage", *Mag. Concr. Res*., **47**(171), 95-102.

Ballim, Y. and Reid, J.C. (2003), "Reinforcement corrosion and the deflection of RC beams-an experimental critique of current test methods", *Cem. Concr. Compos*., **25**, 625-632.

Balogh, J., Fragiacomo, M., Gutkowski, R.M. and Fast, R.S. (2008), "Influence of repeated and sustained loading on the performance of layered wood- concrete composite beams", *J. Struct. Eng*., **134**(3), 430-439.

Berra, M., Castellani, A., Coronelli, D., Zanni, S. and Zhang, G. (2003), "Steel-concrete bond deterioration due to corrosion: finite-element analysis for different confinement levels", *Mag. Concr. Res*., **55**(3), 237-247.

Bhargava, K., Ghosh, A.K., Mori, Y. and Ramanujam, S. (2007), "Corrosion induced bond strength degradation in reinforced concrete-Analytical and empirical models", *Nucl. Eng. Des*., **237**, 1140-1157.

Cabrera, J.G. (1996), "Deterioration of concrete due to reinforcement steel corrosion", *Cem. Concr. Compos*., **18**(1), 47-59.

Castel, A., François, R. and Arliguie, G. (1998), "Mechanical behaviour of corroded reinforced concrete beams—Part 1: Experimental study of corroded beams", *Mater. Struct*., **33**, 539-544.

Chen, W.F. and Saleeb, A.F. (1982), *Constitutive Equations for Engineering Materials: Elasticity and Modelling v. 1*, John Wiley & Sons Inc, London.

Coronelli, D. and Gambarova, P. (2004), "Structural assessment of corroded reinforced concrete beams: Modeling guidelines", *J. Struct. Eng*., **130**(8), 1214-1224.

Dekoster, M., Buyle-Bodin, F., Maurel, O. and Delmas, Y. (2003), "Modeling of the flexural behaviour of RC beams subjected to localized and uniform corrosion", *Eng. Struct*., 25, 1333-1341.

Fang, C., Lundgren, K., Chen, L. and Zhu, C. (2004), "Effect of corrosion on bond in reinforced concrete", *Cem. Concr. Res*., **34**(11), 2159-2167.

Fang, C., Gylltoft, K., Lundgren, K. and Plos, M. (2006), "Effect of corrosion on bond in reinforced concrete under cyclic loading", *Cem. Concr. Res*., **36**, 548-555.

Guo, Z.H. and Shi, X.D. (2003), *Reinforced Concrete Theory and Analyse*, Tsinghua University Press, Beijing.

Jason, L., Huerta, A., Pijaudier-Cabot, G. and Ghavamian, S. (2006), "An elastic plastic damage formulation for concrete: Application to elementary tests and comparison with an isotropic damage model", *Comput. Meth. Appl. Mech. Eng*., **195**, 7077-7092.

Jin, W.L. and Zhao, Y.X. (2001), "Effect of corrosion on bond behavior and bending strength of reinforced concrete beams", *J Zhejiang Univ-Sc*., **2**(3), 298-308.

Mangat, P.S. and Elgarf, M.S. (1999), "Flexural strength of concrete beams with corroding reinforcement", *ACI Struct. J*., **96**(1), 149-159.

Stewart, M.G. (2009), “Mechanical behaviour of pitting corrosion of flexural and shear reinforcement and its effect on structural reliability of corroding RC beams”, *Struct. Saf.*, **31**, 19-30.
Suzuki, H. and Chen, W. (1983), “Elastic-plastic fracture analysis of concrete structures”, *Comput. Struct.*, **16**(6), 697-705.
Stanish, K., Hooton, R.D. and Pantazopoulou, S.J. (1999), “Corrosion effects on bond strength in reinforced concrete”, *ACI Struct. J.*, **96**(6), 915-922.
Tracy, R.G. and Klein, R. (1994), “Corrosion-induced slab liftoff”, *Concr. Int.*, **16**(3), 35-38.