Bond Behavior between Steel and Concrete in Low Level Corrosion of Reinforcing Steel

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SUMMARY:
Bond is one of the main keys to assess the performance of reinforced concrete (RC) structure against seismic load. In this paper, the bond behavior including bond stress, crack propagation, crack spacing and tension stiffening of tension RC members was experimentally investigated under certain levels of corrosion of reinforcing steel. Seven cylindrical specimens having 19mm bar diameter and 2.8 cover to bar diameter ratio were prepared and tested under simulated corrosive environment. The corrosion level was ranging from 0% to 4% in mass loss. In low level corrosion up to 1% of corrosion level, the bond stress increases causing a decrease in average crack spacing. For higher corrosion levels the decrease of average crack spacing is attributed to the decreasing of concrete tensile strength caused by cracks around corroded bar. A simple analytical formula has been proposed to predict the mean crack spacing of corroded tension members.

Keywords: Bond stress, Corrosion, Crack spacing, Stress distribution

1. INTRODUCTION

Bond between reinforcing steel and concrete is an important characteristic to assess the performance RC structure against seismic load. For example, the bond strength of longitudinal reinforcement, the tension of transverse reinforcement, and the compressive strut of concrete are known simultaneously required to transfer shear force by a truss mechanism in RC members. It was recognized that corrosion leads to the decreasing of structural performance in strength, serviceability and durability. Corrosion of reinforcing bar adversely influence the performance of structure in the form of cracking of concrete cover caused by expansion of corrosion product, decreasing of bond strength caused by changing in the concrete-steel interface, and reducing in the steel bar cross-section. The reduction in bond capacity for longitudinal steel bar in corroded RC members can lead to vanishing of truss mechanism to resist shear force due to earthquake load.

The bond behavior of corroded RC members has been previously investigated through experimental and analytical studies. Most of the studies were focused on corroded bar experiencing severe corrosion. Nevertheless, a little attention is given on the bond behavior in low level corrosion. Although, there is not significant change on steel cross-section, but high expansion pressure and cover cracking are known occur in early corrosion process of corroded RC members. From the experimental and analytical work conducted by Andrade et al. (1993) and Shinohara (2011), respectively, it shows that crack of concrete cover is generated by 20 to 30 micrometers of corrosion penetration (corrosion rate of 0.4-0.6%). Motivated by the need of comprehensive assessment at any level of corrosion in RC structures, this paper aims to explore the bond mechanism on tension members experienced corrosion on reinforcing steel.

A tensile test on the corroded reinforcing bar in various corrosion levels has been conducted by Amleh et al. (1999). However, most of the Amleh's specimens were experiencing severe corrosion (more than 4% of corrosion rate in mass) and having longitudinal cracks before the test. By performing tensile test, this experimental test is subjected to gain more knowledge on bond behavior of corroded steel bar in concrete especially in low level corrosion.

2. EXPERIMENTAL PROCEDURE

2.1. Preparation

Seven specimens were prepared for tension test. Each specimen, a deformed bar of 19 mm diameter was installed in the center of a 125 mm diameter concrete cylinder. A machine groove cutting of 3 mm in width, 3 mm in depth, and 840 mm in length was made in reinforcing bar for placing strain gauges (Fig. 1(a)). This grooving was made to avoid damage in strain gauges during accelerated corrosion process. Before and after cutting or grooving the bars, the weight of the bar was measured to determine the reduction rate of sectional area by grooving. At top and bottom of the concrete cylinder, a 50 mm of bond insulation was installed around steel bar to avoid cone damage (Fig. 1(c)). After attaching strain gauges on grooving, the gauge wires were passed through the grooving and taken out at the end of specimens. Then the grooving was filled by waterproofing material. Two types of displacement measurement were attached into specimens. To measure the strain distribution along reinforcing bar a number of strain gauge was attached at every 100mm on bar grooving (Fig. 1(c)) and the global elongation of specimens was measured using vertical jig attached to the specimen (Fig. 1(d)). The specified compressive concrete strength of 28 days was 48 N/mm^2. The concrete proportion/mixing was shown in Table 2.1. The average yield strength of reinforcing bar from tests was 435 N/mm^2, the tensile strength was 610N/mm^2, and the elastic modulus was 1.85×10^5 N/mm^2.

Table 2.1. Concrete Mixing (unit: kg/m^3)

w/c	Water	Cement	Fine Aggregate	Coarse Aggregate	Air entraining and water reducing
0.50	175	350	780	968	0.80%

2.2. Accelerated Corrosion and Loading Method

After 4 weeks of concrete curing, an accelerated corrosion using an electrochemical process was performed to the specimens. The set up was arranged so that the reinforcing bar acted as anode and the copper plate acted as cathode. During electrochemical process specimens were placed in the specific tank and filled with 3 percent NaCl solution. Furthermore, a 200mA of current was applied and monitored using data logger. A monotonic loading of tensile test was performed using a 2000kN of Amsler Universal Testing Machine. The applied load and the displacement were continuously recorded by an automatic data acquisition system.

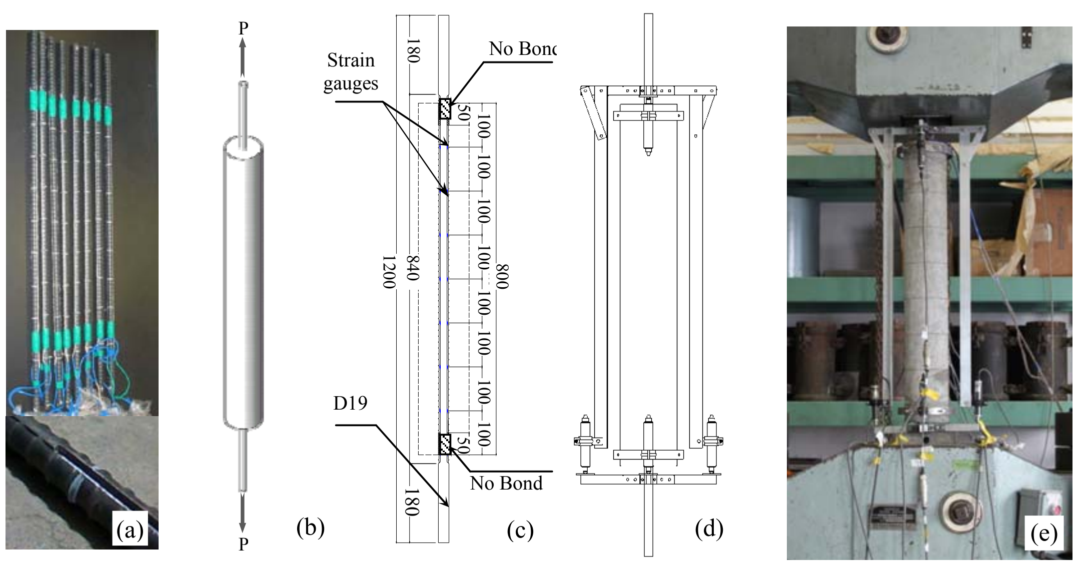

Figure 1. (a) Typical bar grooving (b) Typical specimens (c) Strain gauges location (d) Jig for vertical (e) Test Setup

3. EXPERIMENTAL RESULTS AND DISCUSSION

3.1. Characteristic of Corrosion

The corrosion of the reinforcing bar was classified into seven corrosion levels (Table 3.1). The degree of corrosion was measured as loss in weight of the reinforcing bars divided by original bar weight. To measure the corrosion weight, after the completion of loading test, a corrosion section of 700mm was taken out to calculate the corrosion rate by a mass measurement. To determine corrosion level during accelerated corrosion process, the prediction from relationship between the duration of the impressed current and the corresponding degree of corrosion was determined using Faraday's law. As expected, the corrosion rate calculated by Faraday's law shows an overestimate result. The difference between Faraday's law and weight measurement considerably varies from 30% to 60%, except for specimen No.2.

Table 3.1. Comparison of Corrosion Rate by Faraday's law and Weight Measurements

Designation of reinforcement	Before corroding (g/700mm)	Faraday's law		Weight Measurement	
		Corroded metal (g)	Reduction rate (%)	After corroding (g)	Reduction rate (%)
No.1	1335.5	—	0	—	0
No.2	1331.8	9.54	0.72	1322.2	0.72
No.3	1334.1	30.175	2.26	1322.4	0.88
No.4	1336.8	39.738	2.97	1322.8	1.05
No.5	1335.6	50.36	3.77	1320.7	1.12
No.6	1335.3	73.37	5.49	1299.0	2.72
No.7	1340.8	90.94	6.78	1287.9	3.95

3.2. Crack Propagation

Table 3.2 shows the development of cracking loads related to transverse cracks. Specimen No.1 produced slightly lower first cracking load compared to Specimen No.2 to No.4 which have higher corrosion rate. The highest first cracking load was produced by Specimen No. 4 (corrosion rate 1.05%). Higher first cracking load can indicate higher contribution of concrete or tension stiffening. For higher corrosion rate of specimen No.5, No.6, and No.7 produce slightly lower first cracking load than Specimen No.1 to No.4. It may be caused by the increasing of radial cracks around the bar due to corrosion product expansion. For specimen No.6 and No.7, the longitudinal crack appears in concrete surface during accelerated corrosion before tensile test as shown in Fig. 3(f) and Fig. 3(g). This longitudinal crack also contributes to the reduction of concrete confinement around the bar and reducing the contact area between steel and concrete.

Table 3.2 Transverse Cracking Load

Designation of specimen	First crack (kN)	Second crack (kN)	Third crack (kN)	Fourth crack (kN)
No.1 (0%)	31.0	54.5	—	—
No.2 (0.72%)	33.4	43.7	79.9	—
No.3 (0.88%)	33.6	32.8	67.4	—
No.4 (1.05%)	35.8	40.6	41.7	—
No.5 (1.12%)	29.3	37.2	39.0	52.9
No.6 (2.72%)	26.3	28.5	47.8	87.0
No.7 (3.95%)	27.0	31.5	33.7	50.6

Fig. 3 shows the specimen's crack pattern after steel bar yielding and steel bar strain distribution. The number inside the circle in Fig. 3 indicates the order of cracking occurrence. Because more than a half of attached strain gauges were damage for Specimen No. 1 and No.6, therefore the strain distribution cannot be presented. From this figure, it obviously shows that the strain of reinforcing bar varies along the bar. Clearly, when crack is formed in the specimen, the steel strain at crack location becomes large. It means that reinforcing steel carried most of the applied load.

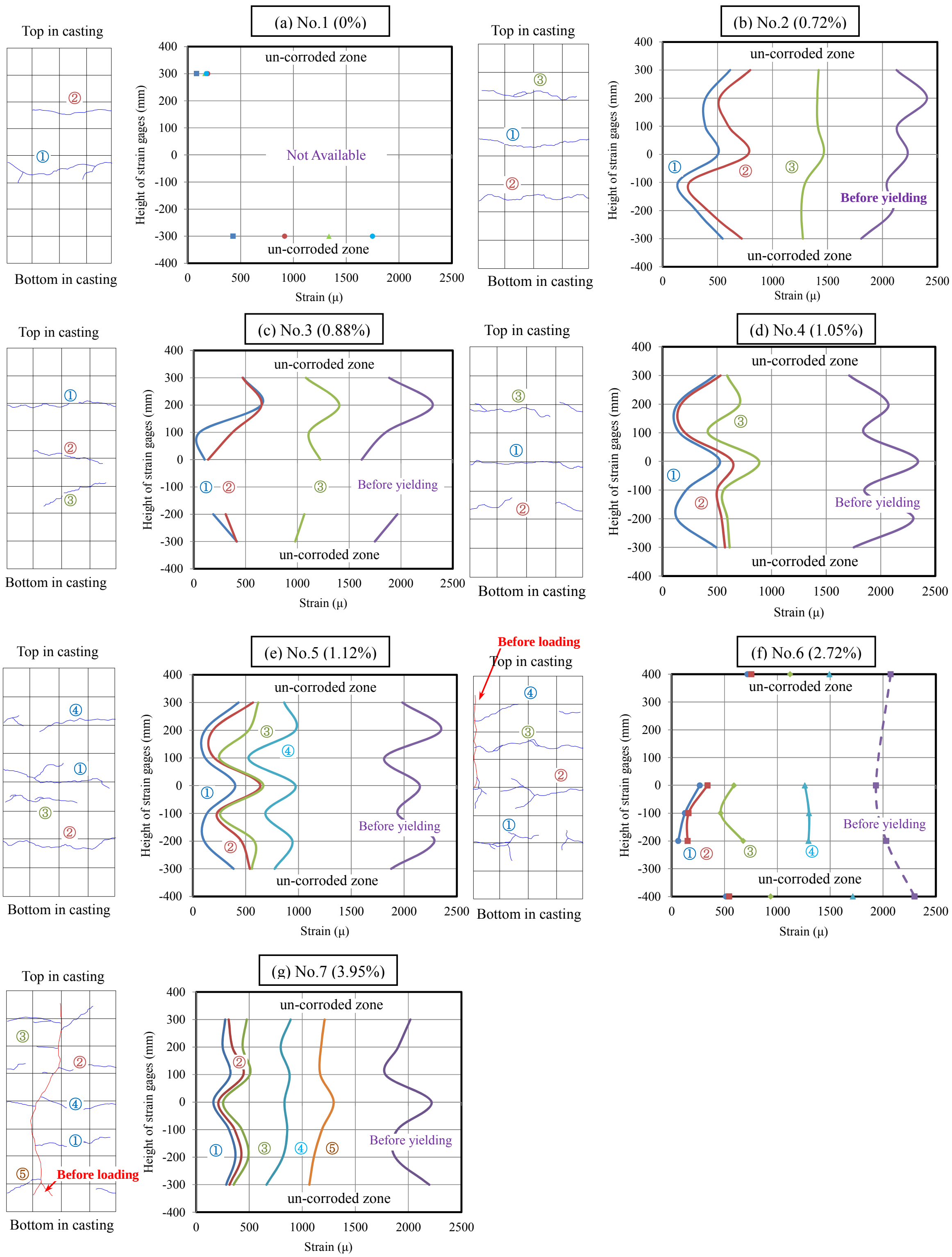

Figure 3. Specimen's Crack Pattern and Steel Strain Distribution (Cont'd)

3.3. Load-Strain Relationship

Fig. 4 shows the load-strain relationship for specimens No.1 to No.7 compared to bare bar under tensile loading. The red line curve in Fig. 4 shows the load-strain relationship of each specimen calculated from the global elongation. The actual global elongation is measured over the gauge length of 840 mm. However, in Fig. 4, the average strain is determined from the effective bond length of 700 mm which is obtained from the actual global elongation deducted by strain elongation of 140 mm of unbonded or bare bar at both ends of specimens. As shown in the Fig. 4 tensile force of specimens No.1 to No.7 before steel bar yielding produced higher force value, tension stiffening, than bare bar. If specimen No.2 and No.3 are compared to healthy specimen, higher tension stiffening is produced as shown in Fig. 4(b)-(c). Meanwhile, Specimen No.4 and No.5, produce almost similar tension stiffening with specimen No.1 (Fig. 4(d)-(e)). This indicates that corrosion influences the bond and the force transfer from steel to surrounding concrete. In other hand, as shown in Fig. 4(f)-(g), Specimen No.6 and No.7, where the longitudinal corrosion cracking formed before the load test, exhibit lower tension stiffening compared to Specimen No.1. It shows that longitudinal corrosion cracking results in bond tension stiffening deterioration. In addition, a small reduction of specimen's yielding load of compared to yielding load of bare bar indicates a small effect of applied corrosion into the yield strength of RC members in low level corrosion.

3.4. Bond Stress

In a typical tension member as shown in Fig. 5, at crack location, all tensile load is carried entirely by steel bar $f_s = P/A_s$. Between adjacent cracks, a portion of tensile force is transmitted to surrounding concrete by bond over the transmission length L_t and causing the stress distribution on steel bar and concrete. If the bond stress distribution along the bar between two adjacent cracks is defined as steel stress variation at certain length. The relationship between bond stress and steel stress variation at certain length can be expressed as follows:

$$\tau_b = \frac{(\sigma_2 - \sigma_1)A_s}{\Delta x \phi_s} = \frac{(\sigma_2 - \sigma_1)}{\Delta x}\frac{d}{4} \tag{3.1}$$

where τ_b is local bond stress over the specified length of Δx; σ_1 and σ_2 are steel stress between Δx; A_s is effective area of steel bar; ϕ_s is perimeter of the steel bar, d is bar diameter and Δx is specified length along the bar or in this case interval length of strain gauges.

The tension force carried by concrete which is transmitted by bond τ_m along the transmission length L_t can be described by

$$F_\tau = \tau_m \phi_s L_t \tag{3.2}$$

where in this case the bond stress τ_m is assumed to be constant and assumed as the maximum local bond stress along the transmission length or the stiffest strain curve, and the transmission length L_t is defined as length over which slip between steel and concrete occurs. The maximum tensile strength of concrete to provoke cracking is given by

$$F_c = f_{ct} A_c \tag{3.3}$$

where f_{ct} is mean value of concrete tensile strength when crack appeared in concrete and A_c is effective area of concrete. When $F_\tau = F_c$ a transverse crack occurs on tension member. Therefore, from Eqn. 3.2 and Eqn. 3.3, it can be derived for the transmission length

$$L_t = \frac{f_{ct} A_c}{\tau_m \phi_s} \tag{3.4}$$

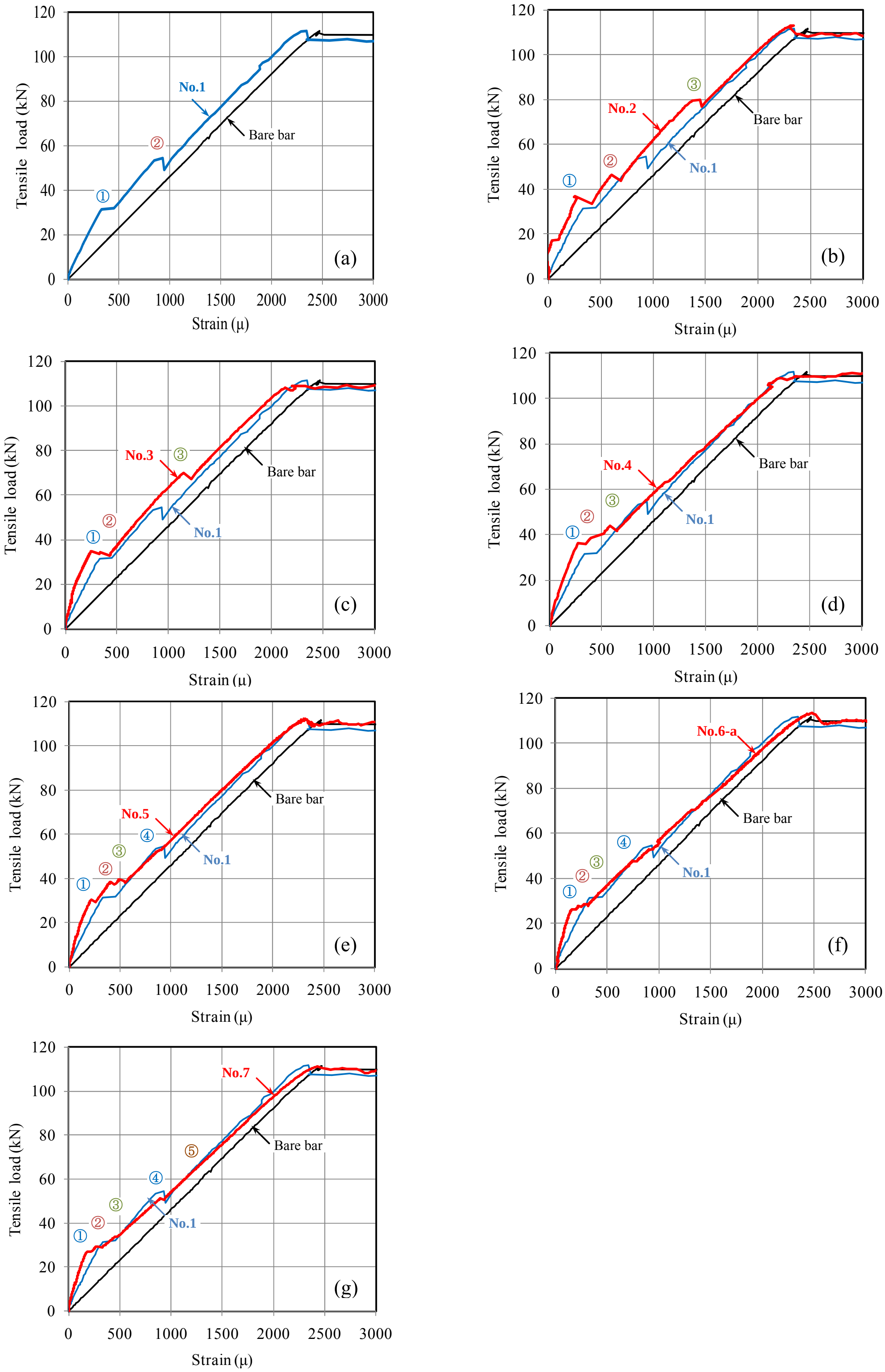

Figure 4. Load-Strain Relationship

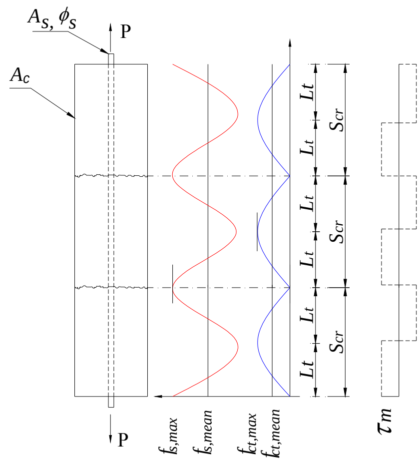

Figure 5. Stress Distribution on Tension Member

From Eqn. 3.4 it shows that the transmission length L_t will decrease if the bond stress τ_m increases or concrete tensile strength f_{ct} decreases. By using Eqn. 3.1 and experimental data as shown in Fig. 3, the maximum local bond stress is obtained, and the concrete tensile strength is obtained using Eqn. 3.4. The results are summarized in Table 3.3. From this table, it shows that for corrosion level up to 1 %, with the increasing of corrosion level, the local bond stress is increasing. One of the possible reasons of increasing bond stress is that the micro space on the interface between concrete and reinforcing steel is filled with corrosion product and increase the mechanical bond properties of interface.

The increasing of bond strength has also been reported by Al-Sulaimani et al. (1990) and Al-Musallam et al. (1996) from the experimentally pull-out test on embedded corroded bar approximately up to 1% of corrosion rate. The change of roughness in the interface of concrete and steel due to corrosion leading to the increasing of mechanical interlocking or friction is mentioned as mainly cause of increasing of bond strength. From Table 3.3, in a larger corrosion level above 1%, the bond stress and average concrete tensile strength decrease gradually. The reduction in local bond stress can be mainly caused by increasing local cracking around bar surface and initial longitudinal crack resulting from corrosion expansion product.

Table 3.3 Maximum Local Bond stress and Concrete Tensile Strength

Specimens	Loss of Weight (%)	Maximum Local Bond Stress (N/mm^2)	Concrete Tensile Strength f_{ct} (N/mm^2)
No.1	0	N/A*	2.30**
No.2	0.72	4.15	2.04
No.3	0.88	4.65	2.33
No.4	1.05	3.64	1.73
No.5	1.12	3.44	1.72
No.6	2.72	N/A*	N/A*
No.7	3.95	1.89	1.34

Note: * Specimen No.1 and No.6 is not available (N/A) caused by the damage of attached strain gauges
** Estimated by AIJ Code

3.5. Crack Spacing

Fig. 6(a) presents the maximum, minimum and mean crack spacing observed from the test. From the figure, the ratio of maximum crack spacing to mean crack spacing ranges from 1.2-1.8 and the ratio minimum to mean crack spacing is 0.4-0.8. It also shows that as the level corrosion increases, the mean crack spacing decreases. This seems a contradiction with the result reported by Amleh et al. (1999) in which the increasing corrosion level leads to decreasing of bond stress and increasing of mean crack spacing. Regarding to Eqn. 3.4, this phenomenon can be an indication of increasing bond stress for low level corrosion resulting in decreasing of transmission length, and followed by decreasing of crack spacing. On the other hand, for higher corrosion level as shown in Table 3.3, the mean concrete tensile strength of specimens seems to be decrease due to the radial cracks around bar during the corrosion process and it also decreasing of transmission length.

The mean crack spacing has been commonly assumed as the average of the minimum (L_t) and maximum ($2L_t$) of transmission length

$$S_{cr\,mean} = 1.5\,L_t \tag{3.5}$$

The transmission length described in Eqn 3.4 is obtained from the equilibrium and the assumption that the distribution of bond stress along the bar between cracks can be taken as a function of maximum bond stress. As mentioned in CEB-FIP Model Code (1990) for non-corroded steel bar, bond stress is assumed proportional to concrete tensile strength f_{ct}. In case of corroded steel, the correlation of bond stress and concrete tensile strength need to be modified because the decreasing of bond stress due to corrosion may be not linearly proportional to the decreasing of tensile concrete strength. Therefore, in this case Eqn 3.4 can be rewritten as

$$L_{t(corr)} = \frac{1}{\alpha}\frac{Ac}{\phi_s}, \text{ where } \alpha = \frac{\tau_{m(corr)}}{f_{ct(corr)}} \tag{3.6}$$

Using available data presented in Table 3.3, the ratio of bond to tensile strength is obtained and the linear equation is made as shown in Fig. 6(b).

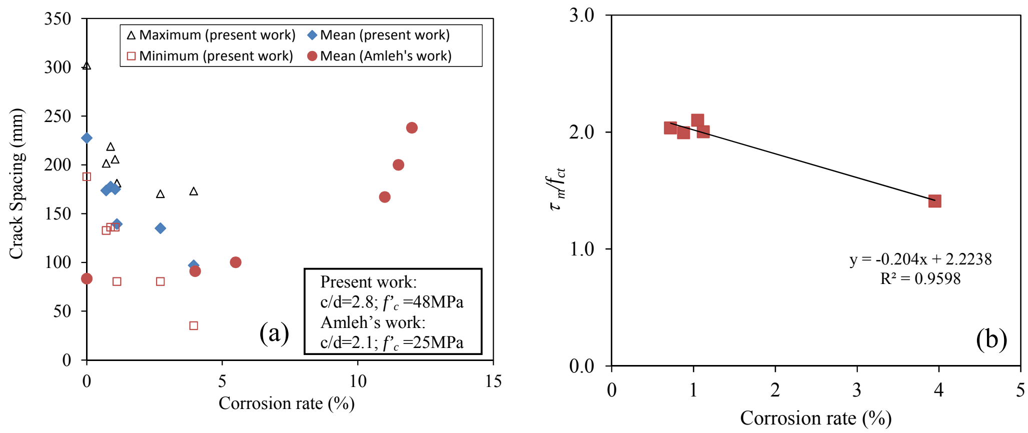

Figure 6. (a) Average Crack Spacing (b) Reduction Factor of Concrete Tensile Strength

Using Eqn 3.5 to 3.6 and the relation of bond to tensile strength ratio as shown in Fig. 6(b), the predicted mean crack spacing was calculated and plotted in Fig 7(a). As shown in Fig. 7(a) the predicted mean crack spacing give reasonable agreement to experimental results, but it still have a large deviation particularly for healthy specimen and specimen No.7. The large difference between predicted and experiment can be attributed to uneven concrete strength, non-uniform corrosion along specimen, or three dimensional effect of corrosion cracking causing cracks distribute unevenly or not full cracking occurs. Using a similar procedure and equation for bond to tensile strength ratio as shown Fig. 6(b), the mean crack spacing for Amleh's experiment is estimated and then it is plotted in Fig 7(b). A good agreement is obtained for low level corrosion up to 5% of corrosion level, but large deviation is generated for higher corrosion level. This is because the available data to obtain the relation of bond to tensile strength in this experiment is limited with maximum corrosion level is only 4%. Considering the large variability regarding to corrosion phenomena such as non-uniformity on corrosion, uneven concrete properties, three-dimensional effect of corrosion, and present of ties, further analysis also need to be considered. However, this empirical prediction is useful on predicting average crack spacing of corroded tension RC members.

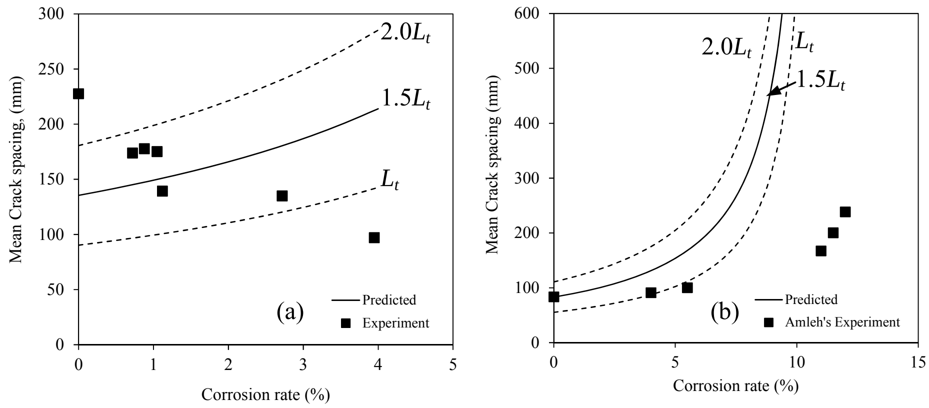

Figure 7. Predicted Average Crack Spacing

4. CONCLUSIONS

Corrosion of reinforcing bar in RC members influences bond stress, crack spacing and tension stiffening. The experimental result of corroded RC tension members indicate that in low level corrosion it shows an increasing of local bond stress. It also shows a decreasing of average crack spacing by increasing of corrosion levels. This decreasing of average crack spacing is mainly attributed to increasing of bond stress in low corrosion up to 1% and the decreasing of concrete tensile strength caused by cracks around corroded bar for higher corrosion. The longitudinal/splitting crack also plays a major rule in the decrease of bond stress and tension stiffening. From the test, as expected there is marginal reduction in yield strength of corroded RC members in low level corrosion.

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