Prediction of Corrosion-Induced Crack Width of Corroded Reinforced Concrete Structures

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ABSTRACT

This paper presents an experimental investigation on the crack width induced by the corrosion of reinforcing steel bars in concrete. Reinforcing steel bars with different diameters were corroded using an accelerated test in three types of concrete members, including 24 beams with corroded stirrup and 20 beams with corroded longitudinal reinforcing steel bars. The corrosion patterns and rusts characteristics of the corroded steel bars were examined and compared with those observed in real RC structures. According to the experimental result, the corrosion-induced crack width were analyzed and showed a linear relationship with the corrosion loss of reinforcing steel bar. Finally, an analytical prediction model of crack width was established after considering concrete cover depth and corrosion penetration depth.

Keywords: structural concrete, crack width, corrosion, reinforcing steel bar, multienvironments

1. INTRODUCTION

Corrosion of reinforcing steel in concrete is the main cause of deterioration of reinforced concrete (RC) structures, and it is also an important issue that needs to be considered when evaluating and rehabilitating RC structures. Random distribution of pore spaces suggests that aggressive substances, such as chloride, carbon dioxide, oxygen, and moisture, may penetrate through the weak points of the concrete cover (Allam, Maslehuddin, Sariciment, & Al Mana, 1994), break down the passive, trigger the corrosion of reinforcing steel bars in concrete (Richardson, 2002), induce the cracking of the concrete (Williamson & Clark, 2000), and finally deteriorate the load-bearing capacity of RC structure.

Often, corrosion of steel rebar is the most important input parameter to study the residual performance of corroded RC structure. To this purpose, researchers have been trying to use nondestructive techniques to obtain the corrosion rate of steel rebar embedded in concrete (Monteiro, Morrison, & Frangos, 1998). However, these methods cannot provide us precise information about the corrosion-induced steel rebar section loss until now (Song & Saraswathy, 2007). Visual inspection is an accessible and effective alternative method to evaluate the state of corrosion of in situ corroded structural members (Stewart, 2010). As corrosion-induced crack width is the common visual indicator of corrosion presence, corrosion crack widths have been shown to always effectively increase with the steel corrosion level (Suo & Stewart, 2009).

Therefore, considerable experiments and models have been devoted to study the relation between corrosion

crack widths and corrosion loss of steel rebar. In experimental investigations on corrosion-induced crack width in concrete, most of the corrosion process (Aligizaki, 1999; Alonso, Andrade, Rodriguez, & Diez, 1998; Andrade, Alonso, & Molina, 1993; Rodriguez, Ortega, Casal, & Diez, 1996a; Vu, Stewart, & Mullard, 2005) is accelerated by applying a current, so that the concrete cracking can be achieved in a relatively short time. However, the corrosion process caused by electrically accelerated tests is very different from natural corrosion and generates the link between corrosion crack widths and corrosion level, which may not be representative of the real on site situation. Vidal, Castel, and Francois (2004) and Zhang, Castel, and François (2010) studied three beams corroded in an aggressive saline environment and subjected to wetting–drying cycles and sustained loading for about 14, 17, and 23 years and found the wetting–drying cycle-induced corrosion was very much closer to natural chloride-induced corrosion. Although this method can provide a result much closer to the real on site corrosion, it involves a very time consuming process. Xia, Jin, and Li (2011) and Malumbela, Moyo, and Alexander (2011) proposed a wetting–drying–current experimental program trying to combine the advantages of the above-mentioned two methods and found that the accelerated wetting–drying–current method can also produce a corrosion distribution, corrosion type, and oxides, which were very much closer to those of natural chloride-induced corrosion (Xia, Jin, Zhao, & Li, 2013).

Moreover, to assess the relation between corrosion crack widths and corrosion level, knowledge of factors controlling crack initiation and propagation is essential.

Usually, the key factors include cover/diameter ratio, corrosion rate, concrete quality, and corrosion process (Alonso et al., 1998). Unfortunately, most of the previous crack propagation empirical models have been obtained and/or calibrated/validated based on limited experimental data (Vidal et al., 2004; Zhang, 2010). Also, due to the lack of experimental data, quantitative verification of the analytical/numerical model is not possible (Li, Melchers, & Zheng, 2006). Therefore, this does not provide sufficient information for identifying the most accurate and efficient models and, consequently, creates problems in selecting appropriate models for estimating the corrosion level of a cracked concrete. In this case, further research, in particular experimental, is needed to improve exist crack propagation models or establish new precise models.

This article investigates the crack propagation process by conducting wetting–drying–current accelerated corrosion tests of RC beams and provides new data on crack propagation. A new empirical model for crack propagation is proposed. Besides, a statistical relationship between the maximum crack width and average crack width is proposed. This suggests the possibility of a very much simple mean to evaluate the corrosion level of in situ RC structures.

2. EXPERIMENTAL INVESTIGATION

Experiments were conducted, which involved 24 beams with corroded stirrup and 20 beams with corroded longitudinal reinforcing steel bars. The concrete members were corroded using a designed wetting–drying–current acceleration method. The corrosion patterns and rusts characteristics of the corroded steel bars were examined and compared with those observed in real RC structures. The concrete surface crack widths were recorded, and steel rebar section loss were measured after removing the reinforcing steel from the concrete samples.

2.1. Specimen details and specimen preparation

Two types of concrete mixes (see Table 1 for the details) were used for the tested reinforced concrete members. In both mixes, ordinary Portland cement with a compressive strength of 42.5 MPa (cylinder specimen test) and coarse aggregate with 16 mm maximum size gravel were used. The designed slump constant was 70 mm. For each mix, three 150 mm × 150 mm × 150 mm cubic specimens were casted. The 28-day compressive strength of the casted cubic specimen was about 25.93 MPa for mix I and 35.55 MPa for mix II, respectively.

Figure 1 shows the details of the dimension and reinforcement arranged in the tested specimens. All of the S1 and S2 beams tested have the same

Table 1. Concrete mix proportion.

Concrete type	w/c Ratio	Water (kg/m ³)	Cement (kg/m ³)	Fine aggregate (kg/m ³)	Coarse aggregate (kg/m ³)
I	0.53	220	412.5	641.2	1046.1
II	0.36	220	611.6	486.4	1033.5

dimensions, which are 230 mm in depth, 120 mm in width, and 1200 mm in length. All of the tested beams have the top layer of steel rebar with two 10 mm rebars, the bottom layer of two 20 mm flexural steel rebar anchored using a standard hook length of 100 mm to prevent anchorage failures, a clear concrete cover of 30 mm, and two pieces of 30 mm × 300 mm × 0.2 mm stainless steel sheets located in the middle layer of the shear zones near the beam ends. S1 beams were arranged using 100 mm stirrup spacing with a diameter of 6 mm and S2 beams were arranged using 150 mm stirrup spacing with a diameter of 8 mm. S1 and S2 beams were cast using concrete type I. All of the F1 and F2 beams tested have the same dimensions, which are 200 mm in depth, 150 mm in width, and 1500 mm in length. All of the tested beams have the top layer of steel rebar with two 10 mm rebars, the bottom layer of two 16 mm flexural steel rebars anchored using a standard hook length of 100 mm to prevent anchorage failures, a clear concrete cover of 30 mm, and a piece of 30 mm × 1300 mm × 0.2 mm stainless steel sheet in the middle layer of the beams, 100 mm stirrup spacing with a diameter of 8 and 10 mm were used for F1 and F2, respectively. F1 and F2 beams were cast using concrete type I and type II, respectively. The stainless steel sheet was used to be acting as an anode for the electromigration test, which will be described in the next section. Details of the specimen setup can be found in Xia et al. (2011, 2012).

Figure 1. Details of the tested specimens.

All specimens were casted from two batches of concrete, wet cured for a period of 7 days and then permitted to dry cure for a period of at least 28 days to achieve the required design strength. After curing, an accelerated corrosion process was applied.

2.2. Accelerated corrosion

Figure 2 shows a schematic representation of the test setup for the acceleration corrosion. Sponge that soaks up 5% NaCl solution was used to keep concrete in the targeted area wet. Afterward, a piece of stainless steel net was kept close to the sponge. Finally, the outside of the beam was wrapped with a plastic sheet to keep the moisture in the sponge. The corrosion procedure can be divided into two phases, namely the electromigration phase and the wetting–drying–current cycle phase.

Figure 2. Schematic representation of accelerated corrosion test setup.

In the electromigration phase, chlorides were electromigrated into concrete cover by means of using an electrochemical method. To simulate the realistic chloride ingress in concrete, 2mol/L NaCl solution was first put in the sponge to make the concrete moisture more than 24 h. Then, the direction of the current flow was adjusted so that the outside stainless steel net became the cathode and the embedded stainless steel sheet served as the anode. Finally, a constant voltage of 30 V was applied between the outside stainless steel net and the embedded stainless steel sheet using a DC power source. The electromigration time used in the acceleration corrosion tests was determined by the following equation (Tang, 1996):

$$t_1 = \frac{RT}{zFED_{nssm}} \left[x_d - 2\sqrt{\frac{RTx_d}{zFE}} \operatorname{erf}^{-1} \left(1 - \frac{2c_d}{c_0} \right) \right] \quad (1)$$

where t_1 is the electromigration time, $R = 8.314$ J/(mol·K) is the molar gas constant, T is the average absolute temperature of the concrete beams, $z = 1$ is the charge number of chloride ions, $F = 96480$ J/(V·mol) is the Faraday constant, E is the applied electric field strength, D_{nssm} is the nonsteady state diffusion coefficient, x_d is the clear concrete cover thickness, c_d is the threshold concentration value of the initial corrosion, and c_0 is the concentration of free chlorides in the solution. The nonsteady state diffusion

coefficient was determined by separate rapid chloride migration tests of concrete cylinders with a diameter of 100 mm and a thickness of 50 mm. For the present type I and type II concrete, the nonsteady state diffusion coefficients were found to be about 6.67×10^{-12} m²/s and 4.00×10^{-12} m²/s, respectively. By taking $E = 300$ V/m, $T = 298$ K, $x_d = 0.03$ m, $c_d = 0.4225$ mol/L, and $c_0 = 2$ mol/L, we have the electromigration times of 4.19 and 6.98 days for the type I and type II concretes, respectively.

Previous experience showed that cracks evolve more rapidly in a dry environment than in a humid environment when an accelerated corrosion process is applied. This is probably due to the transport enhancement of the corrosion products away from the steel–concrete interface in the wet concrete. While in the dry concrete, most of the corrosion products stay at the steel–concrete interface. As a consequence of this, pore pressure may increase and extra tensile stresses at the interface may also be generated. This leads to the development of concrete cracks and the propagation of cracks through the concrete cover. Hence, to simulate the actual environmental degradation process, a wetting–drying–current cycle was used after the electromigration process. The wetting–drying–current cycle involves 3 days dry followed by 4 days wet. The dry process was achieved by taking off the plastic sheet to dry the sponge, whereas in the wet cycle, the plastic sheet was reapplied to cover over the beam and 5% NaCl solution was put in the sponge to make the concrete moisture. For the purpose of an accelerated corrosion, a current density of 2 A/m² was applied through the reinforcing steel (acting as the anode) and the stainless steel net (acting as the cathode) using a constant DC current power source. The total galvanizing time was determined using the following formula:

$$t_2 = \frac{\gamma_s Z_{Fe} F \Delta d_s}{M_{Fe} i} \quad (2)$$

where t_2 is the total galvanizing time, γ_s is the density of the steel, Z_{Fe} is the valence of the Fe, Δd_s is the target corrosion depth, M_{Fe} is the atomic weight of the Fe, and i is the current density. The target corrosion level and the galvanizing time calculated for each tested beam are given in Table 2.

2.3 Corrosion-induced cracking

The distribution of corrosion-induced cracks was copied on a paper by putting a piece of soft glass with a grid of 50 mm × 50 mm on the specimen surface. The maximum crack width in each grid was recorded. Crack width was measured using a crack visualizer with an accuracy of 0.02 mm at each grid (see Figure 3).

Table 2. Summary of experimental responses.

S.D	Concrete	Specimen	t_2 (days)	C (mm)	D (mm)	$\eta_{average}$	$w_{c,average}$ (mm)	$w_{c,max}$ (mm)
I-6-1	I	S1	23.93	24	6	10.68%	0.06	0.08
I-6-2	I	S1	49.23	24	6	27.01%	0.08	0.12
I-6-3	I	S1	76.16	24	6	37.13%	0.12	0.20
I-6-4	I	S1	105.10	24	6	42.54%	0.12	0.40
I-6-5	I	S1	136.57	24	6	54.15%	0.19	0.48
I-6-6	I	S1	23.93	24	6	12.94%	0.04	0.04
I-6-7	I	S1	49.23	24	6	21.75%	0.10	0.12
I-6-8	I	S1	76.16	24	6	29.23%	0.14	0.40
I-6-9	I	S1	105.10	24	6	41.48%	0.13	0.40
I-6-10	I	S1	136.57	24	6	51.42%	0.18	0.44
I-8-1	I	S2	31.48	22	8	6.53%	0.08	0.08
I-8-2	I	S2	64.76	22	8	11.73%	0.13	0.24
I-8-3	I	S2	100.19	22	8	19.54%	0.14	0.24
I-8-4	I	S2	138.26	22	8	25.74%	0.16	0.36
I-8-5	I	S2	179.66	22	8	32.38%	0.19	0.68
I-16-1	I	F1	25.09	30	16	3.25%	0.32	0.68
I-16-2	I	F1	31.44	30	16	4.50%	1.11	1.88
I-16-3	I	F1	37.83	30	16	5.19%	0.79	1.48
I-16-4	I	F1	44.25	30	16	6.97%	1.18	1.92
I-16-5	I	F1	50.71	30	16	7.39%	1.06	2.92
I-16-6	I	F1	57.20	30	16	8.37%	1.01	1.88
I-16-7	I	F1	63.72	30	16	9.91%	0.90	1.76
I-16-8	I	F1	71.27	30	16	10.08%	1.46	3.13
I-16-9	I	F1	77.96	30	16	11.55%	2.38	4.48
II-16-1	II	F2	25.09	30	16	2.91%	0.39	0.80
II-16-2	II	F2	31.44	30	16	3.28%	1.34	2.76
II-16-3	II	F2	37.83	30	16	4.06%	1.54	3.51
II-16-4	II	F2	44.25	30	16	5.14%	0.78	1.60
II-16-5	II	F2	50.71	30	16	6.60%	0.98	2.28
II-16-6	II	F2	57.20	30	16	7.16%	1.72	3.40
II-16-7	II	F2	63.72	30	16	8.65%	1.55	4.16
II-16-8	II	F2	69.29	30	16	9.03%	1.66	3.45
II-16-9	II	F2	75.80	30	16	10.21%	1.42	2.39

Figure 3. Measurement of corrosion-induced cracks on concrete surface. (a) Measurement grid on the specimen surface and (b) Crack width measurement.

2.4. Corrosion level and rust evaluation

After it had corroded, loading bearing test was carried out for each specimen, which has been discussed in

authors' other articles (Xia et al., 2011, 2012) and will not be the focus in this article. After the failure of specimen, the corroded reinforcing steel bars were taken out from the specimen and cut into specimens for further tensile tests (Xia et al., 2013). For the identification phase of deposits, X-ray diffraction and thermal analysis were also performed to characterize corrosion products (Zhao, Ren, Dai, & Jin, 2011). The bars were then cleaned and measured according to the method mentioned in ASTM G1-03 (ASTM, 2003), from which the average cross-section loss ratios η average were calculated using the weight

ratios between the corroded and uncorroded bars (see Table 2).

3. RESULT AND DISCUSSION

3.1 Corrosion pattern and rush composition

Figure 4 gives typical corrosion patterns of the reinforcing steel bars corroded in concrete. It is clear that the corrosion of the steel bar not only reduced its cross-section irregularly but also altered the rib shape on a ribbed bar surface. The steel bar presented a more serious corrosion on the surface facing to the concrete cover than the surface away from the concrete cover, which is much alike what happen in long-term field corrosion in concrete (Trejo, Pillai, Hueste, Reinschmidt, & Gardoni, 2009; Yuan & Ji, 2009). It is noticed from the figures that the corrosion of the steel bar was pitted with somewhat abrupt changes in bar geometry and much variation in the cross-sectional area and is much like the corrosion pattern-induced by chloride aggression.

X-ray diffraction and thermal analysis results for characteristics of corrosion products were presented in Zhao et al. (2011), which showed that, by the above corrosion methodology, both the composition and the expansion rate of the corrosion rust were much alike the corrosion rust from the steel bar corroded in a natural chloride aggressive environment. Especially, the expansion coefficient shows to be 3.14 for the above test and 3.02 for a sample rebar, which was corroded in RC port, Yokosuka, Japan, for 40 years. The similar expansion coefficient of the wetting–drying–current test to a natural explosion can develop a similar corrosion-induced crack propagation process to the in situ RC structures in chloride aggressive environment.

Figure 4. Corrosion patterns for reinforcing steel bars corroded in concrete. (a) Plain bars and (b) ribbed bars.

3.2 Determination of the pit concentration factor

Rodriguez, Ortega, Casal, and Diez (1996b) gives the relation between pit attack and reinforcement diameter loss:

$$d = d_0 - \alpha x \quad (3)$$

where d is the residual rebar diameter; d_0 is the initial rebar diameter; x is the pit penetration; and α is the pit concentration factor.

The rebar cross-sectional area of the pit corroded steel bar can be evaluated by the following analogy equation (Xia et al., 2013):

$$A_s = (1 - 2.1\eta_{average})A_{s0} \quad (4)$$

where A_s is the residual rebar area; A_{s0} is the initial rebar area; x is the pit penetration; and $\eta_{average}$ is the average cross sectional loss of the corroded rebar.

We then can obtain the relation between pit concentration factor and average cross-sectional loss of the corroded rebar:

$$\alpha = \frac{2(1 - \sqrt{1 - 2.1\eta_{average}})}{1 - \sqrt{1 - \eta_{average}}} \quad (5)$$

Then we can deduce the value of pit concentration factor by Eq. (5). Figure 5 gives the relationship between pit concentration factor and average cross-sectional loss in the range of 0–50%. It shows the pit concentration factor in this experiment is between 4.2 and 6.0, which is in accordance with the conclusion of Gonzalez, Andrade, Alonso, and Feliu (1995), $4 < \alpha < 8$.

Figure 5. The relationship between pit concentration factor and average cross-sectional loss of the corroded rebar.

3.3 Steel rebar cross-section loss initiating cracking

We choose the model proposed by Vidal et al. (2004) to predict the steel rebar cross-section loss initiating cracking:

$$\eta_{cr} = 1 - \left[1 - \frac{\alpha}{d_0} \left(7.53 + 9.32 \frac{c}{d_0} \right) \times 10^{-3} \right]^2 \quad (6)$$

where c is the concrete cover depth. It should be noticed that this model does not take the concrete characteristics into account. Thus, further study is needed as we only used two types of concrete with different mixtures. According to the experimental result, the rebar cross-section loss-inducing concrete cracking is below 10% for all specimen in the current study; thereby the pit concentration factor α can be taken as 4.3.

3.4 Corrosion-induced crack analysis

For one location of the specimen, the sum of crack widths on the two beam surfaces, due to the same corroded area of one bar, was calculated. Figure 6 shows an example of two crack configurations, in which the configuration one has an equivalent width w_c, eq corresponding to the sum of the width of two cracks, while the configuration two has an equivalent width w_c, eq corresponding to the width of only one crack (Vidal et al., 2004).

Figure 6. Two equivalent crack configurations for the same corrosion state.

Using the method shown in Figure 3, surface crack maps in the tested specimen were obtained and are plotted. Crack distribution and crack width (in millimeters) at each grid of the corrosion zone are drawn on the maps. According to the above assumption, the average crack width $w_{c,average}$ and maximum crack width $w_{c,max}$ for each tested specimen were also calculated, and the corresponding results are given in Table 2.

Figure 7. The relationships between the corrosion induced average crack width and average cross-section loss ratio.

Figure 7 shows the relationship between the average cross-section loss ratio and the average crack widths. It can be seen that the average crack widths increase with the average cross-section loss ratio, which is similar to what was found previously (Alonso et al., 1998; Malumbela et al., 2011; Mullard & Stewart, 2011; Val, Chernin, & Stewart, 2009; Vidal et al., 2004; Zhang, 2010). However, these previous studies (Alonso et al., 1998; Mullard & Stewart, 2011; Vidal et al., 2004; Zhang, 2010) with regard to corrosion-induced crack widths have linear relationship with the corrosion ratio. On the contrary, Vu and Stewart (2005) proposed a nonlinear crack propagation relationship. However, as can be seen in Figure 6, the crack widths increase with corrosion rate as expected; at the time

that concrete cracks, there is an abrupt increase in crack width, which reflects the assumed quasi-brittle nature of concrete. This phenomenon is consistent with the analytical model proposed by Li et al. (2006) and Zheng, Zhou, and Xu (2006); unfortunately, no experimental data were available to verify these models in their articles. After cracking, the crack width's increasing rate slows down as the corrosion develops. This can be explained as the corrosion products penetrating into the concrete pores and cracks, and, moreover, it is expected that the corrosion products amount should depend on the crack propagation. The cracks are being filled gradually over time, and the thicker the concrete cover the longer it will take to fully fill a crack (Val et al., 2009). By multivariate regression analysis, we can obtain the corrosion-induced average crack width as:

$$w_{c,average} = K \ln \frac{\eta_{average}}{\eta_{cr}} \quad (7)$$

where K is a parameter that should depend on the concrete cover, rebar diameter, concrete porosity and, subsequently, on the water-cement ratio, and the concrete compressive strength. In this article, experimental result from two types of concrete cannot quantitatively but qualitatively draw conclusion considering the influence of concrete mixture. Further research, and especially experimental research, is needed.

Besides, the cover depth has double effects on the corrosion-induced crack width and should be considered serious for different situations. Generally, the initial corrosion ratio inducing cracking increases proportionately with the cover depth, but inversely with the rebar diameter. In another words, with the same corrosion ratio, the smaller the rebar diameter or the larger the cover depth, the smaller the corrosion-induced crack width. However, in another aspect, the measurement of the crack width was conducted on the surface of the concrete, so the larger cover depth will induce a larger corrosion-induced crack width (Wei, Zhou, Zhang, & Li, 2008).

Therefore, in this article, we only consider the influence of cover/diameter ratio to determine parameter K . According to the result of above multivariate regression analysis, K for average crack width can be given as the following expression:

$$K = \left(\frac{c}{d} \right)^{-12} \times 10^3 + 0.087 \quad (8)$$

In comparison with the proposed model, most of the experimental values are close to the modeling natural logarithm curves, except the point of I-16 with average cross-section loss of 11.55%. This significant distinction may be produced by experimental error and can be

excluded from the tested data. As mentioned above, this model does not take the concrete characteristics, such as concrete porosity, and compressive strength, into account. Thus, the experimental data available do not allow any quantitative conclusion. However, by comparing the test data from I-16 and II-16, it can be seen in Figure 8 that, generally, the corrosion-induced crack of II-16 is opening wider than those of I-16 for the same corrosion level. This may due to the concrete of II-16 has a lower water to cement ratio than I-16. Therefore, the concrete of II-16 has a relative smaller porosity than I-16, then fewer corrosion products can penetrate into concrete pores, and the same amount of corrosion products can produce a wider crack width. From another aspect, the concrete of II-16 has a higher compressive strength than I-16, so the concrete of II-16 is more brittle than I-16, which will also help to produce a wider crack width.

Figure 8. The influence of concrete type on crack propagation.

Figure 9 shows the relationship between the maximum crack width and the average crack width. The figure shows that the maximum crack width increases with the average crack width, and the relationship between them can be approximately represented by a linear line with a slope of 2.1. It seems that this relation have no correlation with the specimen type. Note that the average crack width in real structures is very difficult to obtain; this can conveniently afford us a simple visual inspection method to estimate the average crack width by just measuring the maximum crack width on the surface of in situ RC structures. For a field structure, maximum crack widths can be measured with reasonable accuracy with no difficulty.

Figure 9. The relationship between the average width and maximum width of the corrosion-induced cracks.

Figure 10 shows the relationship between the average cross-section loss ratio and the maximum crack widths. Although the data are somewhat scatter, the figure does show that the maximum crack widths also increase with the average cross-section loss ratio. According to the result of above multivariate regression analysis, K for maximum crack width can be given as the following expression:

$$K = 2 \left(\frac{c}{d} \right)^{-12} \times 10^3 + 0.22 \quad (9)$$

Figure 10. The relationships between the corrosion-induced maximum crack width and average cross-section loss ratio.

4. PREDICTION OF CORROSION LEVEL OF CORRODED REBAR IN RC STRUCTURES

An attempt has been made to use the experimental data in proposing a predictive model for the estimation of the corrosion level of a corroded rebar that are subjected to reinforcement corrosion. Note that the crack widths are affected by not only the rebar diameter and concrete cover thickness but also the concrete characteristics. Therefore, the formulae derived here may need to be verified with more experimental data to take the concrete properties in to account. Whether or not they can also be applied to other concrete requires a further study. Nevertheless, it is conceivable that using the crack width to evaluate the corrosion damage amount of embedded rebar in concrete is much easy and convenient.

The proposed crack width prediction model can be used to find out the corrosion level of RC structure that has suffered corrosion damage. As an example, to evaluate the corrosion level of an on-site concrete structure damaged by the reinforcing steel corrosion, the maximum crack width could be first measured and used to find the corrosion level for a pre-evaluation. If a further validation is needed, grid net could be drawn on the target member to obtain an average crack width. Based on the obtained crack widths, one can calculate the corrosion-induced damage of corroded rebar.

5. CONCLUSIONS

This work attempted to obtain information contributing to the development of an empirical formula to predict

the crack width of RC structures due to the corrosion of reinforcing steel. Based on the experimental results obtained with this investigation, the following conclusions can be drawn:

- Both the average crack widths increase with the average cross-section loss ratio. At the time that concrete cracks, there is an abrupt increase in crack width, which reflects the assumed quasi-brittle nature of concrete. After cracking, the crack width's increasing rate slows down as the corrosion develops.
- The maximum crack width seems to have a linear relationship with the average crack width. Therefore, the maximum crack width can be used to provide a prediction of corrosion damage of corroded rebar in a corrosion environment.
- The proposed crack width prediction model can be used to find out the corrosion level of RC structure that has suffered corrosion damage. Besides using the average corrosion-induced crack width to evaluate the corrosion damage of corroded rebar in a corrosion environment, the maximum corrosion-induced crack width can also be a feasible and easy-get choice. However, the influence of the concrete characteristics on the crack width should be considered.

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