CONCRETE LIBRARY OF JSCE NO. 34, DECEMBER 1999

A STUDY ON CRITICAL VALUE FOR DECISION TO REQUIRE REPAIR OF CONCRETE STRUCTURES

(Translation from Proceeding of JSCE, No.544/V-32, 33-41, August, 1998)




Tomoaki TSUTSUMI Takayuki NAKAGAWA Manabu MATSUSHIMA Hiroyuki OHGA

In the maintenance of RC structures, the level of deterioration is judged through visual inspections based on a maintenance manual. If necessary, a detailed inspection is then carried out in order to determine how to repair or reinforce the structure. Decision-making with respect to repairs and reinforcement follows the maintenance manuals of various administrations based on the Standard Specification of the JSCE, JCI, and so on.

This study sets out to determine the boundary between whether or not to require repairs or reinforcement of a power plant's RC structures. Crack width and peeling are selected as measures of chloride-induced damage. The critical value for repair is obtained using actual data by inverse calculation based on reliability theory.

Keywords: crack width, peeling off, concrete cover, decision making of repair, least expected cost, reliability theory.

Dr. Tomoaki Tsutsumi is a Senior Research Engineer at the Engineering Research Center of TEPCO, Tokyo, Japan. He received his Master of Engineering Degree from Tokyo Metropolitan University in 1982. His research interests include the deterioration of RC structures and inspection methods for damaged RC structures. He is a member of the JSCE and JCI.

Mr. Takayuki Nakagawa is a Research Engineer at Tokyo Electric Power Service Co., Ltd, Tokyo, Japan. He graduated from Utsunomiya University in 1995. His research interests include the deterioration of RC structures and the application of mathematical modeling to members suffering chloride-induced damage. He is a member of the JSCE and JCI.

Dr. Manabu Matsushima is a senior research engineer at Tokyo Electric Power Service Co., Ltd, Tokyo, Japan. He received his Doctor of Engineering Degree from Tokyo Denki University in 1994. His research interest is the application of reliability theory to concrete members in RC structures. He is member of the JSCE, JCI, and AIJ.

The late Prof. Hiroyuki Ohga was Associate Professor at Tokyo Metropolitan University, Tokyo, Japan. He received his Doctor of Engineering Degree from Tokyo Institute of Technology in 1989. His research interests included the deterioration of RC structures and new concrete materials. He was a member of the JSCE and JCI

1. INTRODUCTION

Concrete structures have long been regarded as permanent, with a major advantage being low maintenance over many years. However, it has been observed in recent years that many concrete structures are suffering deterioration for various reasons. As a result, the importance of maintenance and rehabilitation work has become widely recognized again[1],[2].

The process for managing concrete structures begins with a judgment of the deterioration level using a simple visual inspection based on a maintenance manual. Then, if rehabilitation works is required, a detailed inspection is carried out. The visual judgement as to the whether or not to repair and reinforcement is required is carried out based on an administration's own standards or those of various public associations. The decision is based on whether or not the performance of the deteriorated structure continues to satisfy the required functionality. The criteria for judging deterioration level are bearing capacity, durability, serviceability, and appearance. However, the most important of these criteria are the bearing capacity of members and cracking along the reinforcement.

Bearing capacity can be obtained through structural calculations taking into account the reduced cross section of reinforcement using data obtained in situ. Static bearing capacity is not reduced by the occurrence of cracks along the reinforcement because the corrosion products are few when cracking occurs. However, bearing capacity is reduced because the bond strength of the reinforcement is reduced under repeated positive-negative loading. The corrosion rate of reinforcement increases with elapsed time after cracking.

Appearance is judged in terms of cracking due to corrosion products and rust stains. Deformation is judged by reduced stiffness of members due to peeling-off of cover concrete[3].

Visual inspection of any deterioration is the usual approach at the first stage, while the detailed, secondary stage entails investigation of chloride ion density, carbonation depth, and none detective test. Given the thoroughness of the detailed stage, these inspections are carried out only after the decision has been made to require repairs. According to the results of a questionnaire put to concrete engineers[4], the decision as to whether or not to require repairs mainly depends on concrete cracking and peeling-off. Of secondary importance are exposure of the reinforcement, corrosion products, rust stain, lack of bearing capacity, leakage of water, and appearance.

These criteria are judged by visual inspection[5]～[15]. For example, according to the Standard Specifications of the Port and Harbor Bureau, deterioration is judged using three criteria: the corrosion state of the reinforcement, concrete cracking, and concrete peeling. The authors have studied use of crack width as a criterion for repair and peeling off[16]. Miyamoto at al. [17] propose equipment for evaluating concrete bridges based on a neuro-fuzzy expert system constructed using the results of questionnaires completed by inspection engineers.

This method provides for evaluation of visual inspections also of repair intervals while taking cost-performance into consideration, and this approach will in future prove useful in maintenance and rehabilitation work. However, there is a need for more deterioration data to be gathered as in situ data remains scant.

In this paper, based on the past results of maintenance and rehabilitation works in practice, the critical value of chloride induced damage, namely crack width and peeling, at which repairs became necessary is obtained by inverse calculation using the in situ deterioration data.

2. CRITERIA IN CURRENT STANDARDS

The criteria used to judge the relationship between crack width and repair interval differ among various administrations. Some do not refer to crack width at all. Some treat crack width as related to reinforcement corrosion while others accept no link. Thus, no consensus can be reached among scholars at present.

Two crack width criteria can be considered one for design and one for repair. The critical value for design is smaller than that for repair[3]. The critical value of crack width at various administrations is shown in Figure 1 [5] ~[15]. Where no value is actually given in the specifications, it is derived by comparison with other criteria and the authors' experience with deterioration inspections. The target structure is a landing pier assumed to be located in the splash zone. According to this figure, the critical value of crack width lies between 0.2 and 0.4mm in various administrations[5].

The state of peeling at which repair becomes necessary is shown in Figure 2. The criteria of most administrations requires repair and reinforcing when partial peeling off occurs. In the case of structures, such as bridges where injury to the general public is a possibility, repairs are requited even when light damage occurs to be on the safe side.

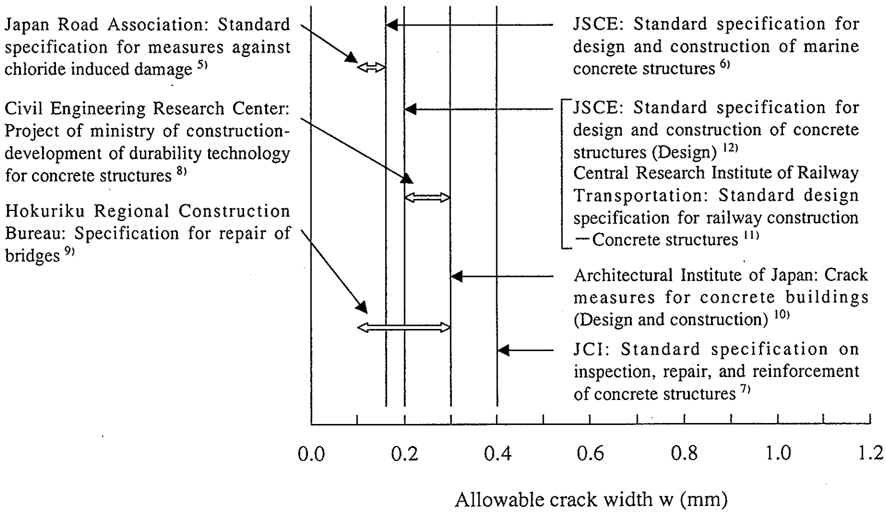


Figure 1 Criterion for crack width

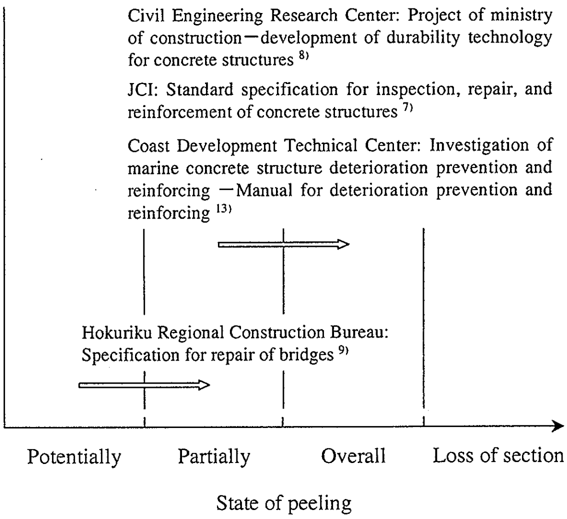


Figure 2 Criterion for peeling

3. DECISION AS TO WHETHER OR NOT TO REQUIRE REPAIR

The deterioration level of a target structure must be evaluated in order to manage its maintenance. Although each administration has its own criteria for deterioration level, assumptions are based on the characteristics of the target structure. Typical criteria are as given in investigation of marine concrete structures on deterioration prevention and reinforcing-manual for deterioration prevention and reinforcing-edited by Coast Development Technical Center[13], Standard specification for maintenance management of structures, edited by Central Research Institute of Railway Transportation[14], and Inspection specification on road structures edited by Hanshin Expressway Public Corporation[15]. In these specifications, concrete cracks, peeled off area, and rust stains are mainly judged by visual inspection and the results used to determine the deterioration level. Once the deterioration level has been determined, a decision is made according to 1) the importance of the structure, 2) the results of visual inspection, and 3) budget. Criterion 3) is artificial and varies with social conditions. Criteria 1) and 2) are universal and can be explained as follows.

The decision as to whether or not to require repair depends on the importance of the structure and the function of its structural members. The importance of repair can be determined by considering the function of structural members in the order 1) column, 2) beam, 3) slab, and 4) other non-structural members.

Table 1 Maintenance Management Diagram using Visual Inspection

Items	Damage Level		
	II	III	IV
(1) Crack width	Crack pattern*1) not affecting structure and function. When cracks affect structural function, crack width*2) is w<0.005Cmm.	Crack pattern not affecting structure and function. Even if crack width w<0.005mm, propagation of crack progresses.	Crack width affecting bearing capacity of members
(2) Amount of Peeling	Peeling with diameter less than 50cm and depth less than 2.5cm.	Peeling with diameter over 50cm and depth over 2.5cm.	Peeling affecting bearing capacity of members
(3) Exposure State of Aggregate	Exposure of aggregate.	Aggregate to separate separating off or likely	States of aggregate affecting bearing capacity of members
(4) Rust Stain	Spotty rust stains	Widespread rust stains	-
(5) Exposure of Reinforcement	Exposure of reinforcement of non-structural members.	Exposure of reinforcement of structural members.	Exposure of reinforcement of structural members affecting bearing capacity of members.

*1) Crack not affecting structural members is not bending or shear crack but crack parallel to main bars or crack due to drying and shrinking.

*2) Calculation using cover thickness. Generally, the relationship between cover thickness and crack width is as follows;

Cover thickness t(mm)	30	50	70
Crack width w_{cr}	0.10	0.20	0.35

Visual inspections are mainly evaluated with respect to a) crack width, b) concrete peeling, c) rust stains, d) exposure of reinforcement, and e) exposure of aggregate. Of these, deterioration depends directly on a) crack width and b) concrete peeling while c),d), and e) are subordinate. These conclusions can be substantiated by statistical analysis using periodical in situ inspection data[17].

An example of the criteria used for visual inspections in a marine environment is shown in Table 1. The deterioration level is divided into four stages: Ⅰ: no deterioration, Ⅱ: slight deterioration, Ⅲ: mild deterioration, and Ⅳ: severe deterioration[18]. Rehabilitation work is required when the deterioration exceeds Ⅲ and the boundary between deterioration levels Ⅱ and Ⅲ is the criterion for repair. Although the decision mainly depends on crack width and peeling, the need for rehabilitation work may be judged synthetically from the importance of the structure and the location of deteriorated members.

4. CRITERIA FOR DETERIORATION LEVEL USING INVERSE ANALYSIS

An inverse method using crack width is explained below. Concrete structural members are divided into four types, a) column, b) beam, c) wall, and d) slab. Crack width distributions reflecting deterioration levels Ⅱ and Ⅲ, as obtained from visual inspections of existing concrete structures located in Tokyo Bay, are shown in Figure 3. Although the boundary between Ⅱ and Ⅲ seems vague, the border between Ⅱ and Ⅲ can be found. These data are for cases where only cracking occurs but no peeling. Therefor, an analysis considered both complex evaluations don't require. The probability density function for each deterioration level is modeled using the crack width data at deterioration levels Ⅱ and Ⅲ. Since the result is not symmetrical about the mean of the data, a log-normal distribution gives an approximation as shown in the figure 4.

The log-normal probability density function model $f_2(w)$, $f_3(w)$ for deterioration levels Ⅱ and Ⅲ may be defined as Eq.(1) and Eq.(2)[19].

$$f_2(w)=\frac{1}{\sqrt{2\pi}\xi_2 w}\exp\left[-\frac{1}{2}\left(\frac{\ell n w-\lambda_2}{\xi_2}\right)^2\right] \tag{1}$$

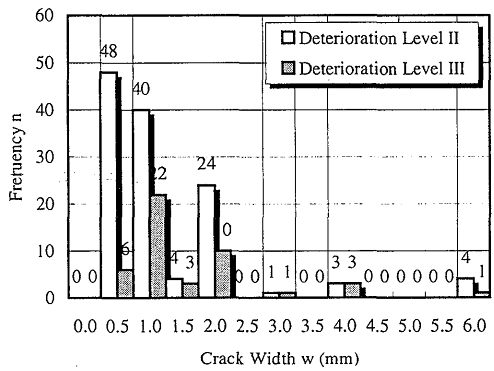


Figure 3 Crack width of deterioration levels Ⅱ and Ⅲ (Wall)

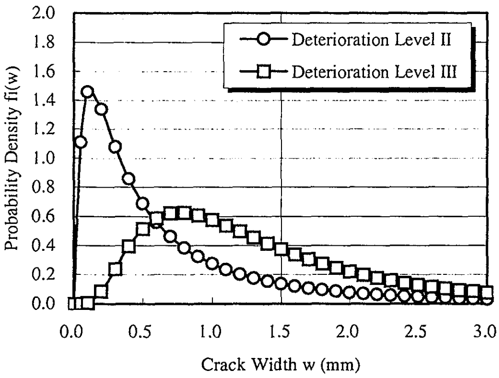


Figure 4 Relationship between $f_2(w)$ and $f_3(w)$

Table 2 Mean and Standard Deviation of Each Deterioration Level

Items	Member	Deterioration level Ⅱ		Deterioration level Ⅲ	
		Mean	S.D.	Mean	S.D.
Crack width w_{cr}(mm)	Column	0.50(14)	1.23	2.77(21)	2.21
	Beam	0.75(22)	0.68	3.39(54)	2.37
	Slab	0.00(3)	0.0	10.44(24)	5.97
	Wall	0.89(74)	1.24	1.49(47)	1.12
Peeling A_{cr}(m^2)	Column	0.40(16)	0.16	1.54(9)	0.67
	Beam	0.33(17)	0.16	0.49(3)	0.00
	Slab	0.05(12)	0.04	0.87(7)	0.35
	Wall	0.41(57)	0.63	3.26(9)	1.48

Number in () indicates number of data.

$$f_3(w)=\frac{1}{\sqrt{2\pi}\xi_3 w}\exp\left[-\frac{1}{2}\left(\frac{\ell n w-\lambda_3}{\xi_3}\right)^2\right] \quad (2)$$

Where, λ_L and ξ_L are the coefficients of the log-normal probability density function and described as $\lambda_L=\ell n\mu_L-\frac{1}{2}\xi_L{}^2$ and $\xi_L{}^2=\ell n\left(1+\frac{\sigma_L{}^2}{\mu_L{}^2}\right)$, representatively. μ_L and σ_L are the mean and standard deviation of crack width at deterioration level L. Suffices 2 and 3 indicate deterioration levels Ⅱ and Ⅲ, respectively. The value in () in Table 2 indicates the number of data obtained in situ. The relationship between $f_2(w)$ and $f_3(w)$ is shown in Figure 4 using a wall as an example. Deterioration level Ⅱ may be selected when the crack width is small. On the contrary, deterioration level Ⅲ is indicated when the crack width is large. The boundary between Ⅱ and Ⅲ is a gray zone that is not easy to divide. Assuming that the crack width at which repair becomes necessary is w_{cr}, the probability of misjudging a deterioration level of Ⅲ when the actual structure is at deterioration level Ⅱ is described as Eq.(3).

$$C_2(w_{cr}) = \int_{w_{cr}}^{+\infty} f_2(w)dr \tag{3}$$

In the same way, the probability of judging a deterioration level of Ⅱ when the target structure is at deterioration level Ⅲ is described as Eq.(4)

$$C_3(w_{cr}) = \int_{0}^{w_{cr}} f_3(w)dr \tag{4}$$

Therefore, w_{cr} is chosen as the optimum crack width at which repair becomes necessary when $C_I(w_{cr})$ obtained as the summation Eq.(3) and Eq.(4) is minimized as shown in following equation:

$$\begin{aligned} C_T(w_{cr}) &= C_2(w_{cr}) + C_3(w_{cr}) \\ &= \int_{w_{cr}}^{+\infty} f_2(w)dr + \int_{0}^{w_{cr}} f_3(w)dr \end{aligned} \tag{5}$$

The distribution of crack width to require repair is described as Eq.(6) by conversing $C_T(w_{cr})$ to the equation to take a maximum value when $C_T(w_{cr})$ may be minimized.

$$f_{w_{cr}}(w_{cr}) = 1 - C_T(w_{cr}) \tag{6}$$

The probability distributions of crack width $f_{wcr}(w)$ and peeled area $f_{Acr}(A)$ obtained in this paper are shown in Figure 5(a)~(d) and Figure 6(a)~(d) respectively. $C_2(w), C_3(w), C_2(A)$, and $C_3(A)$ are also shown in the same figures.

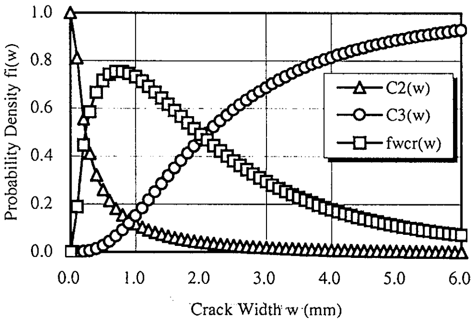
5. CLASSIFICATION OF REPAIRS

The maximum likelihood estimator of crack width distribution and peeled area is assumed to coincide with the decision point as to whether or not to require repair in this paper. Likelihood values obtained using the method described above are shown in Table 3. Crack width for slabs and peeled area for beams are excluded from the evaluation because of a shortage of in situ data as indicated in the table. Crack width obtained in the above manner indicates 0.6~1.2 mm, a value larger than the 0.2mm given by the Standard Specification for Design and Construction of Concrete Structures (JSCE; environment: severe; cover thickness: 7cm). This value of 0.2mm is obtained by considering the allowable crack width for design, and its value is chosen on the safe side. According to the Standard Specification for Inspection, Repair, and Reinforcement of Concrete Structures (JCI) [11], 0.4mm is the allowable crack width at which repair becomes necessary and this value is also small even if scatter is considered. According to the results on peeling, as shown in the table, the value at slab indicates about only $0.3m^3$. Critical value at column and wall indicate about $1.0m^3$. The data at slab can't be reliability as shown in Figure 6(c) because the whole data at unrepair is nearly zero. Therefore, the results of peeling off at slab are to deal as reference after this.

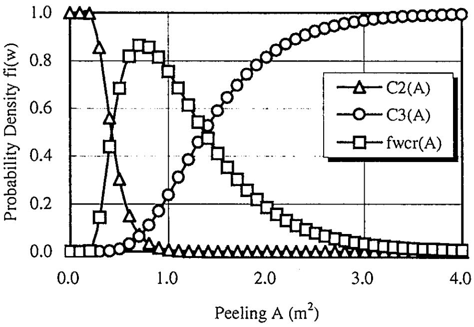
The concept behind the classification of rehabilitation work is non described. The probability distribution indicating repair is described as Eq.(6). Therefore, assuming that the integral of Eq.(6) over the whole distribution is 1.0, the risk probability of crack width whether or not to require repair $F_{wc}(w_c)$ is described as Eq.(7).

$$F_{w_c}(w_c) = \int_0^{w_c} f_{w_{cr}}(w_c)dw \tag{7}$$

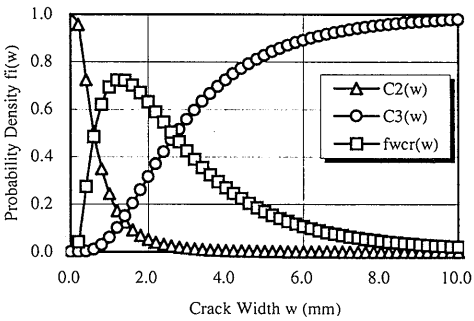
Where, w_{cr} indicates the critical value of crack width with risk probability taken into account. The relationship between crack width w_{cr} and risk probability $F_{wc}(w_c)$ is also shown in Figure 8(a)~(d). The integral of the whole crack width distribution is adjusted to equal 1.0 when the crack width is 10.0mm and when the area of peeling is $4.0m^3$. Although the deterioration value may reach infinity in a mathematical sense, an upper limit value is assumed in the manner described earlier in this paper, so the actual phenomenon has an upper limit. Equation (7) means that rehabilitation work is definitely not required at zero and is certainly required at 1.0. Therefore, Eq.(7) represents the probability of repair being required. Figure 7 indicates that the cumulate probability density function at walls and beams may increase rapidly to 1.0 with crack width w_{cr} while it may increase slowly to 1.0 for other members. The difference can be explained by the difference in scatter of deterioration levels Ⅱ and Ⅲ, as shown in Figure 5. In case of the cumulate probability density function increasing rapidly to 1.0, the point at which repair become necessary is ill-defined because the overlap is larger and the variation of both distribution is spread. On the contrary, when it increases slowly to 1.0, the decision is clear because the overlap is smaller and the variation is narrowed. The critical value of crack width and peeling with considering risk probability taken into account is obtained as follows. Generally, 5% or 1% is statistically permitted when a rare event occurs[20]. It is said that those values are appropriate to a human sense[20]. A 5% risk probability, namely a 95% reliability value, is also described in the Standard Specification for Design and Construction of Concrete Structures (JSCE) if it is assumed that the distribution of materials is a normal distribution. A 95% reliability value is accepted in this paper according to the same argument. The critical values mentioned above are the case of severe condition that a members may be caused immediately by received damage. On the other hand, in the case that the bearing capacity of the structure has deteriorated moderately due to chloride-induced damage and unaffected immediately bearing capacity by the occurrence of deterioration cracking, the risk probability can take larger than 5% risk probability. Although the same discussions are left how to choose the risk probability, a 15% risk probability, namely a 85% reliability value, is adopted in this paper. The critical values of crack width obtained using Figure 7 are shown in Figure 9. The critical values of peeling obtained using Figure 8 are shown in Figure 10. The crack width is 0.3~0.6mm for a 85% reliability value and the critical values coincide roughly with various administrations (0.2~0.4mm) because various standard specifications for repair are developed in consideration of future deterioration at the design stage.



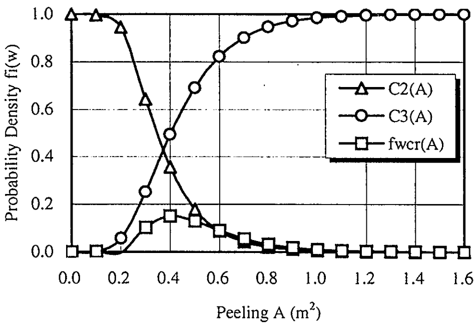
(a) Column



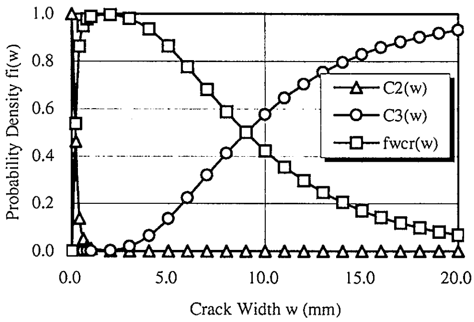
(a) Column



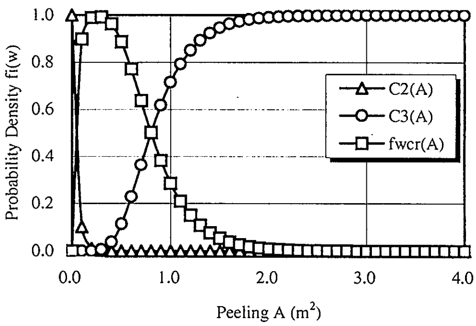
(b)Beam



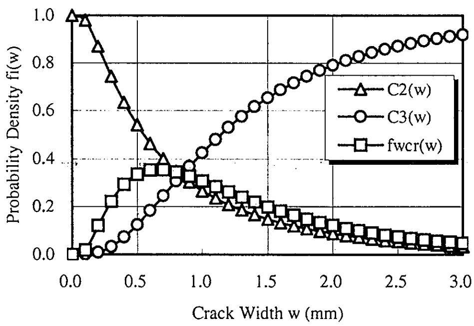
(b)Beam



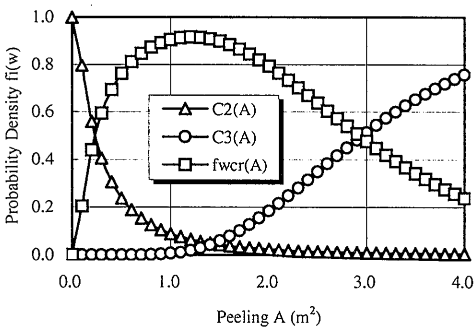
(c)Slab



(c)Slab



(d)Wall



(d)Wall

Figure 5 Probability distribution of crack width $f_{wcr}(w)$ Figure 6 Probability distribution of peeled are $f_{Acr}(A)$

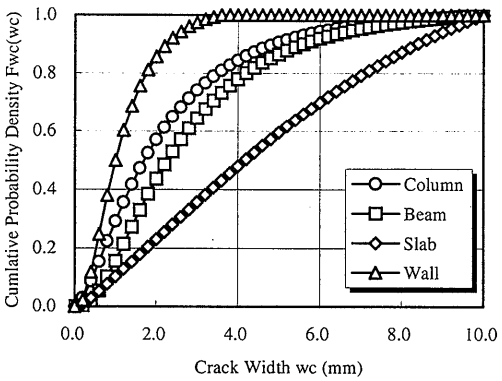


Figure 7 Relationship between $F_{wc}(w_c)$ and crack width w_c

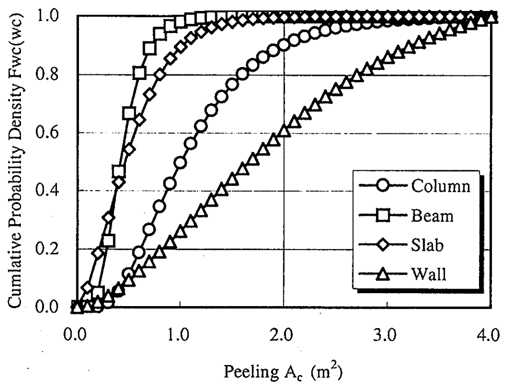


Figure 8 Relationship between $F_{Ac}(A_c)$ and peeled area A_c

Table 3 Critical Values

Items	Crack Width W_c(mm)	Peeling A_c(m^2)
Column	0.8	0.7
Beam	1.2	(0.4)
Slab	(2.0)*	0.3*
Wall	0.6	1.3

Data in () for reference only due to shortage at data.
Data marked * for reference due to whole data of deterioration II nearly equaling to 0.0.

The crack width for an 85% reliability value is 0.5～1.0mm. These values are larger than existing standard specifications and coincides with the likelihood value (0.8～1.0mm) obtained from the distribution of crack width as shown in Table 3. It seems that the critical value in maintenance mutual is in too safety side with considering the fact that rehabilitation management in practice has been carried out well. Consequently, it can be proposed that the critical value of crack width in actual rehabilitation management can be eased as a result of the argument above.

The area of peeling is $0.4m^2$ at a 95% reliability value and $0.6\sim0.7m^2$ at on 85% reliability value. The critical value of $0.5m^2$ at which repairs are necessary, as shown in Table 1, coincides roughly with the critical values obtained here.

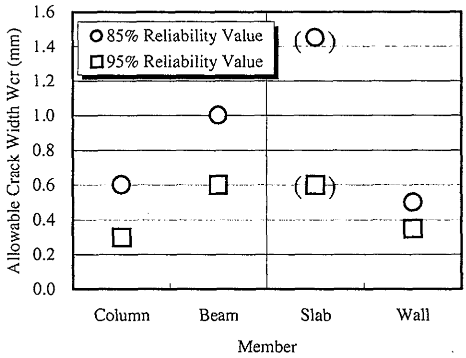


Figure 9 Allowable crack width w_{cr}

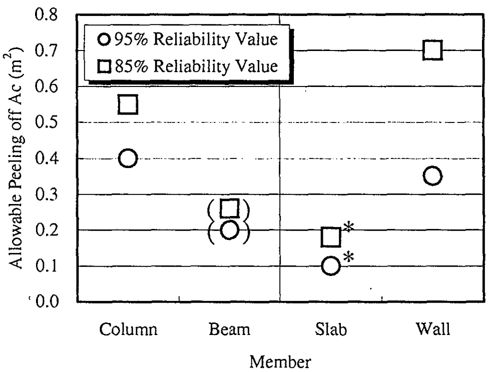


Figure 10 Allowable peeled area A_c

6. CONCLUSION

In this paper, based on the past results of maintenance and rehabilitation works in practice, critical values of major parameters of chloride-induced damage, namely crack width and concrete peeling at which repairs becomes necessary are obtained by inverse calculation using deterioration data obtained in situ.

The following is a summary of the findings:

(1) A stochastic model is proposed using the distribution for two deterioration levels Ⅱ and Ⅲ using existing investigation data.

(2) The crack width at which repair is required is obtained using the proposed model. A value of 0.0~0.6mm at a 95% reliability level is obtained, and coincides with various current standards. A value of 0.0~1.0mm at an 85% reliability level is obtained, and coincides with records of past rehabilitation work.

(3) The area of concrete peeling is $0.4m^2$ at a 95% reliability level and $0.6\sim0.7m^2$ at an 85% reliability level. The boundary value of $0.5m^2$ between repair being required at not, or between deterioration levels Ⅱ and Ⅲ as shown in Table 1, coincides roughly with the

critical value obtained in this paper.

REFERENCES

1) Ootuki N., and Sekino T,: Studies on the durability of old concrete structures in HOKAIDO, Proceeding of JSCE,No.366/V-4,pp.27-37,1996.2 (in Japanese).

2) Kobayashi, K.: Durability of concrete structures, Proceeding of Japan Concrete Institute,Vol.23, No.2,pp.4-12, Feb.,1985 (in Japanese).

3) Manabu Matsushima, Hiroshi Seki, Yuuichi Kanekko, and Kunihito Kaneko: Evaluation on crack width required to repair RC structures, Proceeding of JSCE,No.420/V-13,pp.181-190,1990.8 (in Japanese).

4) Investigation results of questionnaire on inspection engineering of damage level, Central Research Institute of Electric Power Industry, 1992.10 (in Japanese).

5) Japan Road Association: Standard specification of measures for chloride induced damage, 1984.2 (in Japanese).

6) Japan Society of Civil Engineering: Standard Specification for Design and Construction of Marine Concrete Structures, 1977.3 (in Japanese).

7) Japan Concrete Institute: Standard specification for inspection, repair, and reinforcement of concrete structures, 1987.2 (in Japanese).

8) Civil Engineering Research Center: Project of Ministry of Construction -Development of durability technology for concrete structures, 1989.5 (in Japanese).

9) Hokuriku Regional Construction Bureau: Specification for repair of bridges (in Japanese).

10) Architectural Institute of Japan: Crack measures for concrete buildings (Design and Construction), 1990 (in Japanese).

11) Central Research Institute of Railway Transportation, Standard design specification for railway construction - Concrete structures, 1992 (in Japanese).

12) Japan Society of Civil Engineering: Standard Specification for Design and Construction of Concrete Structures-1996, part 1 (Design).

13) Coast Development Technical Center, Investigation of marine concrete structures on deterioration prevention and reinforcing-manual for deterioration prevention and reinforcing, 1987.9 (in Japanese).

14) Central Research Institute of Railway Transportation: Standard specification for maintenance management of structures, 1987.9 (in Japanese).

15) Hanshin Expressway Public Corporation: Inspection specification for road structures, 1985.9 (in Japanese).

16)Miyamoto A., Morikawa H., Kinoshita K,. and Takeuchi K.: Development of neuro-fuzzy system for serviceability assessment of concrete bridges, Proceeding of JCI, Vol. 15,No.2,pp.771-776,1993.6.

17) Yasuda N., Siraki W., Matsushima M., and Tustsumi T.: Evaluation of deteriorating concrete structures using neural network, Proceeding of JSCE,No.496/V-24,pp.41-49,1994.8 (in Japanese).

18) Tokyo Electric Power Company: Manual on maintenance management - Civil engineering structures for thermal power stations and nuclear power stations, 1988.3 (in Japanese).

19) Alfredo H-S. Ang and Wilson H. Tang: Probability Concepts in Engineering Planning and Design, 1975, John Wiley and Sons.

20) Okamura H.: Limit state design method for concrete structures, Kyoritsu Publication, 1984