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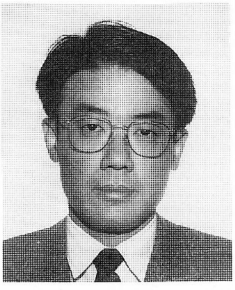
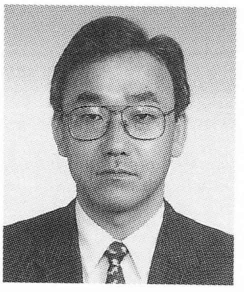
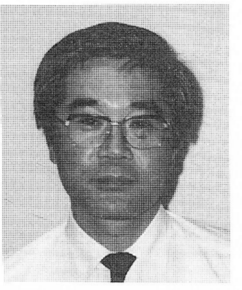
STRUCTURAL SAFETY EVALUATION AND REMAINING LIFE PREDICTION OF CONCRETE BRIDGES BASED ON STATISTICAL ANALYSIS

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This paper describes a practical evaluation method for structural safety and resulting change in service life of concrete bridges based on only brief material tests. Statistical factor analysis is used to evaluate the structural safety of actual bridges based on non-destructive loading tests and material tests. Furthermore, the remaining life based from current maintenance criteria for repair, rehabilitation and renewal, is discussed by linking results obtained by the presented method and visual diagnosis by bridge engineers. Finally, an application to an actual bridge is described so as to demonstrate the suitability of the method.

Keywords : *concrete bridge, safety evaluation, remaining life, carbonation, statistical analysis*

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1. INTRODUCTION

Since damage to and deterioration of existing concrete bridges has become more and more of an issue over recent years, the maintenance effort required has been increasing. The Japan Society of Civil Engineers (JSCE) published a tentative plan for the volume of maintenance in its standard specifications for concrete structures[1]. In this plan, four maintenance levels, defined as Level-A (preventive measures), Level-B (ex post fact measures), Level-C (Inspection and observation), and Level-D (no-maintenance requiring suspension of traffic and replacement in future), are proposed along with the basic techniques and issues requiring solution of each level. Rational non-destructive evaluation (NDE) techniques and deterioration prediction methods are listed as important issues. While visual inspections which are standard in the current maintenance method, are very easy and economic to carry out and economy, they involve subjective uncertainties which are not suitable for the maintenance of the large number of existing bridges. On the other hand, the various NDE tests and loading tests which have been developed for more objective evaluations are not widely deployed because of cost problems and the need to suspend traffic. Though the authors[2, 3] have developed a precise safety evaluation method based on dynamic loading tests and material tests, it is not suitable for periodic inspections because for the same reasons and is really a limited inspection for the final stages of maintenance. There is therefore a need to develop simpler methods for periodic inspections. In addition, decisions regarding the optimum timing for precise inspection and evaluation at the final stage after these periodic inspections are also an important issue for rational maintenance.

Regarding techniques to predict deterioration, research concerning the mechanism of carbonation of concrete[4], the effect of carbonation on concrete's mechanical characteristics[5-7], and the effect of various factors on carbonation rate[7-9] have been carried out in previous studies. However, it has been found that the actual phenomenon of concrete carbonation differs from that seen in laboratory tests[10, 11]. Though durability design for reinforcement bar corrosion based on decision on cover concrete depth considering the carbonation rate has been studied in previous work, the remaining life prediction based on the evaluation of material quality and deterioration of concrete has not been done. Further, since initial data on material and structural properties is not available for most bridges, the prediction of such data is the most important issue facing remaining life evaluations.

Given this situation, a statistical analysis based on safety evaluation results for several bridges of equivalent type and similar scale is introduced as the foundation for a method of predicting remaining life using only an inspection of current deterioration status. The carbonation rate is considered as a factor which is keenly influenced by concreting standards, environmental and traffic conditions, in the accurate evaluation on each target bridge. Using this method, it is possible to evaluate safety and remaining life by carrying out material tests only without the need for large-scale dynamic tests. Furthermore, a decision regarding the optimum timing of dynamic tests and accurate evaluations in the final stage can be made. A method for updating the statistical equation is also considered, in which the Bayesian procedure is applied to evaluation results obtained in field dynamic tests. On the other hand, since it is considered acceptable in actual bridge maintenance procedures to calibrate the safety limit -- which is the basis for decisions on repair, strengthening, and replacement -- against the current status, the safety limit is evaluated on the basis of questionnaires completed by bridge engineers.

Finally, an application to an existing concrete bridge is presented so as to demonstrate the suitability of the method by comparing it with an evaluation by bridge engineers.

2. STATISTICAL ANALYSIS METHODOLOGY FOR LOSS OF STRUCTURAL SAFETY

2.1 Safety Evaluation Procedure Based on Field Testing

Fig.1 shows the procedure for the proposed safety evaluation based on field testing. First, the system identification (SI) method is applied to modal parameters evaluated by performing dynamic loading tests and the analysis model for the target bridge is calibrated using identified structural parameters such as girder stiffness and frictional constraint coefficient of supports. Then, an analysis of sectional force is carried out using this model in order to evaluate a probability model of load effect. On the other hand, the probability model of resistance can be evaluated using statistical data on ultimate load tests of bridges other than the target and the results of material tests on the target bridge. Finally, a safety index can be estimated using these probability models based on Hasofer & Lind's method taking into account the nonlinear influence that design variables have on the performance function.

2.2 Application to Existing Bridges

The proposed method was applied to five existing bridges of the RC-T simply supported type in the range 27 to 60 years old as shown in **Table 1**. All of these bridges were scheduled for demolition due to river improvement work. The distribution of design vehicle weight was assumed as follows according to the Hanshin Expressway Public Corporation.

$$(\mu_w,\ \sigma_w) = (20.3,\ 3.37)\ \text{tf} \qquad (1)$$

Fig.2 shows the results of a safety evaluation of the three of the existing bridges. It was found that the safety of the outside girder is lower than that of the inside girder for both flexural and shear failure modes. This can be considered attributable to the concentration of sectional force on the outside girder due to an imbalance in girder stiffness between the outside and inside girders. Then, for older bridges, safety for the shear failure mode is lower than that for the flexural failure mode, contrary to the design condition. The effect of the low concrete strength measured on older bridges on the shear failure mode can be considered more sensitive. The regression equa-tions for the correlation between bridge age and safety are as follows.

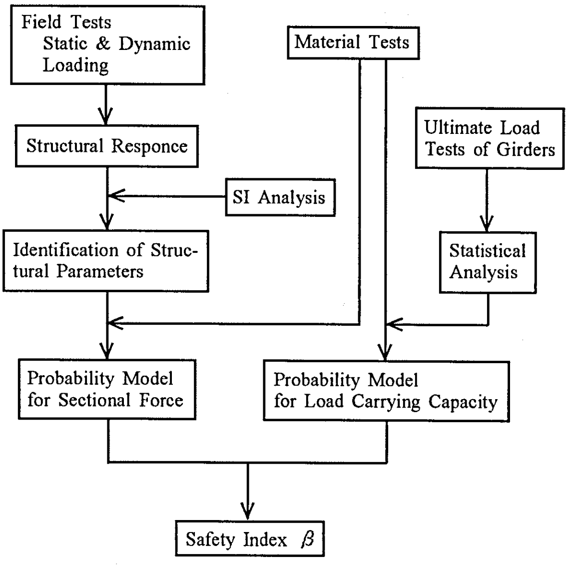
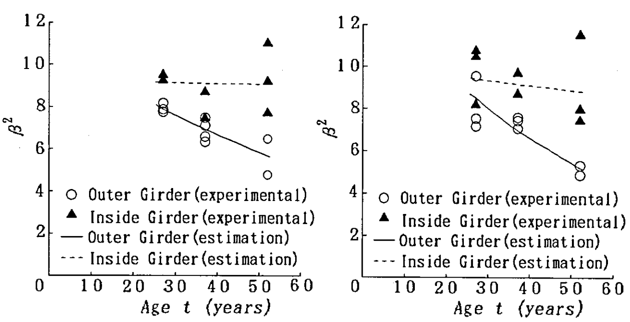


Fig.1 Flow of safety evaluation based on field tests

Table 1 Decription of target bridges

Name of bridge	Sakura	Maeno	Taita	Nakai	Oyasu
Span(m)	10.9	9.2	9.8	10.8	4.7
Age (years)	52	55	37	60	27
Number of girders	5	4	3	3	4
Cross beam	Yes	No	Yes	No	Yes



(a) Flexural mode (b) Shear mode

Fig.2 Relationship between age and safety index

$$\beta^2 = 11.88 \cdot \exp\left(-1.47 \times 10^{-2} t\right) \ :\ \text{flexural mode, outer girder} \qquad (2)$$

$$\beta^2 = 14.62 \cdot \exp\left(-1.98 \times 10^{-2} t\right) \ :\ \text{shear mode, outer girder} \qquad (3)$$

2.3 Method of Statistical Prediction of Safety Degradation Considering Influencing Factors

As mentioned, it is possible to predict safety on the basis of safety evaluated for several other bridges, though there is a large data scatter. Here, the degradation curve for the mean value of safety index is defined as the exponential function in Eq.(2) and Eq.(3).

$$\overline{F}(t) = a \cdot \exp(-bt) \tag{4}$$

where, a, b : constant, t : age(years).

The scatter in the data can be considered as consisting of a statistical error and an error related to influencing factors such as construction conditions, loading conditions, environmental conditions, and so on. The former can be improved by accumulating statistical data. For the latter, it is necessary to introduce these effects into the statistical analysis by evaluating their correlation with safety. The carbonation rate, which correlates with construction conditions (including quality of concreting and materials) and environmental conditions, can be considered a suitable influencing factor for an objective and quantitative evaluation. Furthermore, since the carbonation rate also relates to mechanical properties such as strength and modulus of elasticity, it can be considered an influencing factor on safety when the shear resistance and load distribution change. Then the deviation in safety is defined as a nonlinear function of time and deviations in the influence factors as follows.

$$\delta F = \sum_{i=1}^{n} \int_0^{\delta_i} \rho_i(t, x_i) dx_i \tag{5}$$

where, n : number of influencing factors, x_i: influencing factor,
δ_i: deviation of influencing factor

Assuming

$$\rho_i(t, x_i) = p_i \cdot \exp(q_i t + r_i x_i) \tag{6}$$

where, p_i, q_i, r_i : constant.

Eq.(5) can be transformed to

$$\delta F = \sum_{i=1}^{n} \frac{p_i}{r_i} \left[\exp(q_i t + r_i \delta_i) - \exp(q_i t)\right] \tag{7}$$

Assuming that the distribution of deviation of the influencing factor at age t_1 is the following:

$$f(\delta_i) = \frac{1}{\sqrt{2\pi}\zeta_{\delta_i}\delta_i} \cdot \exp\left[-\frac{1}{2}\left(\frac{\ln\delta_i - \lambda_{\delta_i}}{\zeta_{\delta_i}}\right)^2\right] \tag{8}$$

the variance in the deviation of safety can be expressed by:

$$\sigma_{\delta F}^2 = \sum_{i=1}^{n} \int_0^{\infty} \left\{\frac{\partial \delta F_i}{\partial \delta_i} \delta\delta_i\right\}^2 f(\delta_i)\, d\delta_i = \sum_{i=1}^{n} \int_0^{\infty} \left\{p_i \cdot \exp(q_i t + r_i \delta_i)\delta\delta_i\right\}^2 f(\delta_i) d\delta_i \tag{9}$$

Finally, the safety of the target bridge considering the influencing factors can be estimated by:

$$F(t_1) = \overline{F}(t_1) + \delta F(t_1) = a \cdot \exp(-bt_1) + \sum_{i=1}^{n} \frac{p_i}{r_i}\left[\exp(q_i t_1 + r_i \delta_i) - \exp(q_i t_1)\right] \tag{10}$$

By obtaining the deviation of influencing factors by inspections at age t_1, the safety can be estimated from Eq.(10). Secondly, it is need to predict the degradation characteristics in the safety from the age t_1. For this purpose, the safety is normalized as follows.

$$\overline{R}(t) = \frac{\overline{F}(t)}{a} = \exp(-bt) \tag{11}$$

By approximating Eq.(10) into the form of Eq.(11),

$$\overline{R}(t) = \frac{F(t)}{A} = \exp\{-B(t-t_0)\} \tag{12}$$

where, $t > t_1 > t_0$

$$A = a \cdot \exp(-bt_0) + \sum_{i=1}^{n} \frac{p_i}{r_i}\left[\exp\left(q_i t_0 + r_i \delta_i^{t_0}\right) - \exp(q_i t_0)\right] \tag{13}$$

$$B = \frac{-1}{t-t_0} \ln\left[\frac{a}{A}\exp(-bt_1) + \frac{1}{A}\sum_{i=1}^{n}\frac{p_i}{r_i}\left\{\exp\left(q_i t_1 + r_i \delta_i^{t_1}\right) - \exp(q_i t_1)\right\}\right] \tag{14}$$

where, $\delta_i^{t_0}, \delta_i^{t_1}$: deviation of influencing factors at t_0, t_1

By obtaining the deviation value of the influencing factors at age t_0, the safety degradation curve for the target bridge can be drawn using Eq.(12). Using the conceptual figure shown in **Fig.3**, the safety at age t_1 is evaluated by the statistical standard degradation curve Eq.(4) and the nonlinear function Eq.(7) for the deviation from the standard value. Then the degradation curve is made by estimating the deviation of the influencing factors at age t_0.

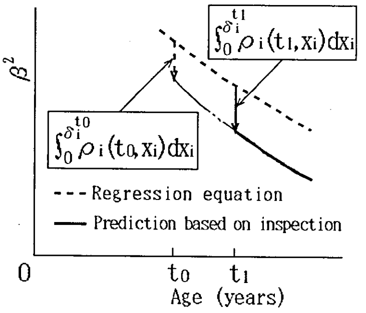


Fig.3 Conceptual figure of making prediction equation

2.4 Updating the Degradation Curve by Verification Testing

By using the degradation curve, the degradation in safety can be estimated based on the results of measurement of influencing factor deviations. The degradation curve should then be updated according to the results of verification tests performed at an appropriate age.

Coefficient B in Eq.(14), which can be evaluated from the statistical data obtained from safety evaluations, may include uncertainties related to data scatter, insufficient statistical data, and so on. Here, we assume that the distribution of coefficient B is a log-normal function as follows.

$$f_0(B) = \frac{1}{\sqrt{2\pi}\zeta_B B} \cdot \exp\left[-\frac{1}{2}\left\{\frac{\ln B - \lambda_B}{\zeta_B}\right\}^2\right] \tag{15}$$

Assuming that $\lambda_B = \mathrm{E}[\ln B]$, $\zeta_B^2 = \sigma^2[\ln B]$, variance of safety R can be expressed as:

$$\sigma_R^2 = \int_0^{\infty}\left[\exp\{-B(t-t_0)\}\right]^2 f_0(B)\,\mathrm{d}B \tag{16}$$

Then, the coefficient can be expressed using the influencing factor deviations as:

$$\zeta_B^2 = \ln\left(1 + \frac{\sigma_B^2}{\overline{B}^2}\right) \tag{17}$$

where, $\sigma_B^2 = \sum_{i=1}^{n}\left(\frac{\partial B}{\partial \delta_i}\delta\delta_i\right)^2$

Assuming that safety at age t reaches R_{limit}, which is the limit value of safety from a maintenance viewpoint, the probability of $R>R_{\text{limit}}$ can be expressed by:

$$P_r[R \geq R_{\text{limit}}] = \int_0^{B'} f_0(B)\mathrm{d}B \tag{18}$$

where,

$$B' = -\ln\frac{R_{\text{limit}}}{(t-t_0)} \tag{19}$$

Assuming that G represents the event where the normalized safety value R based on field testing at age t_2 fulfills the condition $R>R_{\text{limit}}$, the distribution of coefficient B, $f_o(B)$ can be transformed by the Bayesian theorem.

$$f_1(B) = f[b = B \mid G] = \frac{P_r[G \mid b = B]f_0(B)}{\int_0^{\infty} P_r[G \mid b = B]f_0(B)\,\mathrm{d}B} = \frac{f_0(B)}{\int_{B_1}^{B_2} f_0(B)\mathrm{d}B} \tag{20}$$

where, $B = -\frac{\ln R}{t_2 - t_0}$, $B_2 > B > B_1$ is obtained from field tests and it is assumed that $P_r[G \mid b = B] = \text{const.}$

Accordingly, the safety and its variance at age t_2 can be expressed by:

$$R(t_2) = \int_{B_1}^{B_2} \exp\{-B(t_2 - t_0)\} f_1(B)\,\mathrm{d}B \tag{21}$$

$$\sigma_R^2 = \int_{B_1}^{B_2} \left[\exp\{-B(t_2 - t_0)\} - R(t_2)\right]^2 f_1(B)\,\mathrm{d}B \tag{22}$$

The safety and its variance after age t_2 can be expressed by:

$$\overline{R}(t) = \exp\{-B_1(t - t_0)\} \tag{23}$$

$$B_1 = \int_{B_1}^{B_2} f_1(B)\,\mathrm{d}B \tag{24}$$

$$\sigma_{B_1}^2 = \int_{B_1}^{B_2} (B - B_1)^2 f_1(B)\mathrm{d}B \tag{25}$$

Fig.4 shows a conceptual outline of this procedure. First, when obtaining B with the condition $B_2>B>B_1$ as a result of verification test, the distribution of coefficient B, $f_0(B)$: (① in **Fig.4**) is updated to the distribution $f_1(B)$: (② in **Fig.4**). From the viewpoint of practical use, $f_1(B)$ can be replaced by an equivalent log-normal function $f_1'(B)$: (③ in **Fig.4**) which has the same mean value and variance as $f_1(B)$. On the other hand, the timing of the verification test should be determined using same basis which unifies the level of safety of bridges at the age of verification. Here, the probability of $R<R_{\text{limit}}$ can be expressed by:

$$p = 1 - P_r[R \geq R_{\text{limit}}] \tag{26}$$

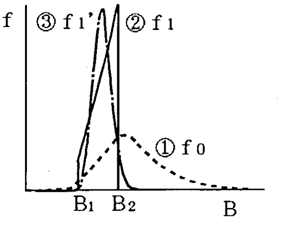


Fig.4 Updating of prediction equation

Defining p_{limit} as the basis of p for verification testing, then t_{limit} when $p=p_{\text{limit}}$ can be calculated using Eq.(26), Eq.(19), and Eq.(18). Here, p_{limit} should be determined using statistical data on actual maintenance work carried out by each maintenance organization.

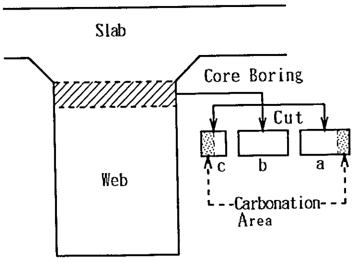


Fig.5 Concrete core boring

Table 2 Results from chemical analysis of concrete

Span & Location		f_c kgf/cm^2	E_c $\times 10^5$ kgf/cm^2	C cm	ρ kgf/m^3	W_c kgf/m^3
1	a	135.3	1.10	4.15	2196	288
	b	178.7	1.35	—	—	—
	c	112.4	—	3.17	—	—
	a	216.1	1.98	2.72	2274	235
	b	199.3	2.10	—	—	—
	c	150.2	—	6.50	—	—
	a	213.1	2.12	3.33	2266	239
	b	211.5	2.14	—	—	—
	c	170.1	—	3.42	—	—
2	a	83.6	—	6.83	2160	170
	b	86.1	1.07	—	—	—
	c	—	—	5.85	—	—
	a	131.2	1.43	5.06	2230	224
	b	132.4	1.45	—	—	—
	c	138.4	—	3.83	—	—
	a	99.1	0.87	5.85	2174	221
	b	88.0	—	—	—	—
	c	88.4	—	6.20	—	—

where f_c:Strength of concrete, E_c:Modulus of elasticity,
C:Carbonation depth, ρ:Unit weight,
W_c:Cement weight content in unit volume of concrete

2.5 Statistical Consideration of Concrete Carbonation

In order to maintain concrete structures effectively, it is necessary to evaluate the degradation in concrete quality by understanding the mechanism of deterioration as well as to evaluate the status of the concrete at any time through inspections. Here, a statistical approach to the former is described by using the results of material tests on several existing concrete bridges.

The target bridges are five RC T-beam bridges as described in **Table 1** and another RC T-beam bridge "Kashitanibashi" which is 41 years old. Core specimens were extracted from the upper part of the web concrete. After compressive tests in which the compressive strength and modulus of elasticity were measured, the carbonation depth was measured by a chemical method. In the case of the "Kashitanibashi" bridge, the core specimens were divided into three pieces as shown in **Fig.5** and the material properties for each piece were tested. Three to six core specimens were extracted from each girder of two spans of the "Kashitanibashi" bridge, which is a three-span three girder bridge.

Table 2 shows the results of the material tests, including the evaluation of mix proportion by chemical analysis. Here, since the carbonated area of the specimen is restrained by the loading plate, the compressive strength measured in the middle part of the specimen is that of the non-carbonated concrete. According to previous laboratory tests on the compressive strength of carbonated concrete[5, 6], it was found that concrete strength increase due to an increase in density (a decrease in fine voids). On the contrary, the Japanese Concrete Institute points to a degradation in material properties for carbonated concrete in a report by a special committee on the concrete carbonation. It can thus be said that the effects of carbonation on the material properties of concrete are not clearly known. Further, it should be noted that even strength and carbonation depth data for the same bridge has a high degree of scatter. Most of the specimens indicate extremely low strength and deep carbonation. According to chemical analysis, the weight per unit volume and cement volume for such specimens are extremely low as compared with standard concrete.

Figures 6 and **7** show the relationship between carbonation rate as measured on existing bridges and material properties; that is, compressive strength and modulus of elasticity. A negative correlation can be recognized, as indicated by previous research[7, 8, 11]. The regression equations were derived as follows.

$$f_c = 260.6 \cdot \exp(-1.072A) \quad (\text{kgf}/\text{cm}^2) \quad (\rho = -0.788 : \text{correlation coefficient}) \tag{27}$$

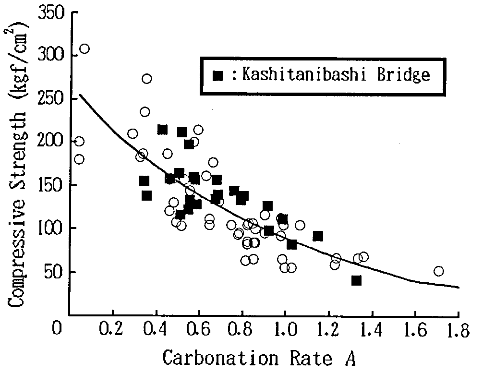


Fig.6 Relationship between carbonation rate and strength

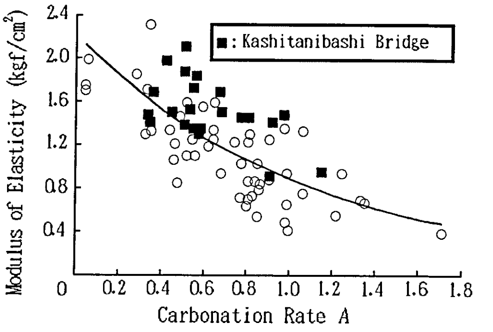


Fig.7 Relationship between carbonation rate and modulus of elasticity

$$E_c = 2.198 \cdot \exp(-0.915A) \quad (\mathrm{kgf/cm^2})$$
$$(\rho = -0.742 : \text{correlation coefficient}) \qquad (28)$$

Fig.8 shows the relationship between age and carbonation rate for each bridge. The coefficient of carbonation rate in the case of on 80% water-cement ratio as estimated by the Kishitani Equation is also given in the Figure. According to these results, the older the bridge, the higher its carbonation rate. In particular, the carbonation rate for bridges older than about 40 years is higher than the value estimated by the Kishitani Equation. If it is assumed that the carbonation rate does not change with aging, the construction quality, including concreting standards, can be considered to have been low, given that environmental conditions, loading, and other factors are little different among the target bridges. If the Kishitani equation is applied to the results for old bridges ignoring the change in carbonation rate, the water cement ratio can be estimated to have been extremely high at 157% for the 41 year-old bridge. This estimation agrees with the results as 151% water cement ratio of estimation from the cement volume of 170kgf/m^3 obtained from the chemical analysis by taking the standard assumption for various condition. Then, for concrete with the same water-cement ratio, the smaller the quantity of cement volume, the higher the carbonation rate becomes because of the small quantity of calcium hydroxide[7]. In that case, it is impossible to apply the conventional prediction equation for carbonation. On the other hand, the possibility of a change in the carbonation rate due to the occurrence and growth of micro cracks in the concrete compression zone might be considered. According to fatigue tests on concrete slabs in which the effect of water on the slabs was considered, the compressive zone of the concrete slabs deteriorates severely[19]. Since there is a possibility that the concrete in main girders might sustain such damage due to cyclic loading with water effects, the carbonation rate in the main girders might change with aging. If the splitting of calcium oxide - silicate - water (in short, C-S-H) compounds is considered, the characteristics of change in carbonation rate are likely to be complicated.

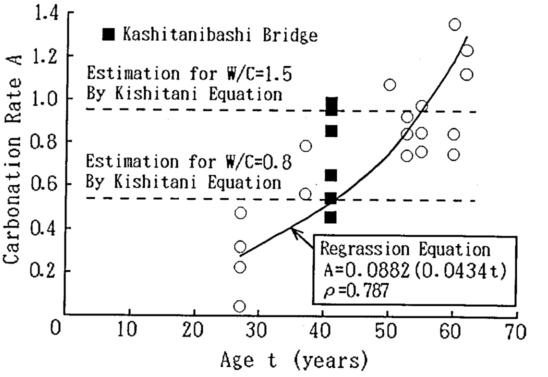


Fig.8 Relationship between age and carbonation rate

2.6 Safety Evaluation based on the Carbonation Rate as an Influencing Factor

According to the above results, since carbonation rate correlates with compressive strength and modulus of elasticity which affect safety, it can be used as an index for evaluating degradation in safety.

On the other hand, since the carbonation rate correlates with the water-cement ratio, it is also a useful index of construction conditions. If concrete deteriorates due to cyclic loading and the effects of water, the carbonation rate may change with aging. However, since the relationship between concrete deterioration and carbonation has not been resolved clearly, we make an assumption of the relationship based on two cases. Here, since the carbonation rate correlates with the compressive strength of uncarbonated concrete, we ignore the difference between the strength of carbonated and uncarbonated concrete. Our assumptions are, ①: a case where material deteriorationis not considered, and ②: a case where material deterioration is considered. In each case, the change in safety with aging is calculated by the procedure given below.

① Without material deterioration: Since the material properties of concrete correlate with carbonation, the carbonation rate can only change if environmental conditions change. Here, if changes in environmental conditions are ignored, it can be assumed that the deviation of influencing factor at t_1 is equal to that at t_0 for the calculation of degradation in safety using Eq.(21).

② With material deterioration: If it is assumed that concrete deteriorates with aging, the carbonation rate might increase. If applying the assumption ((1)) to the structure under such condition, the evaluation might be at the danger side. On the other hand, as mentioned earlier, since the mechanism of carbonation with material deterioration is not clear, there is a need to measure the changes in carbonation and material properties. However, there are no examples of such measurements on existing deteriorated bridges. In this research, by observing the carbonation rate on existing bridges of the same type and of similar scale and by inspecting the target bridge, we evaluate the bridge at the safe side as far as possible, taking evaluations based on assumptions ((1)) and ((2)) above as the lower and upper bounds

3. EVALUATION OF DEGRADATION CURVE AND SAFETY LIMIT FOR MAINTENANCE

3.1 Evaluation of Degradation Curve

We evaluated the degradation curve by carrying out a sensitivity analysis. Before doing so, it was necessary to determine the standard value of the coefficient of carbonation rate so as to calculate the first term of Eq.(10). Here, we use a standard value measured on existing bridges as shown in **Fig.8**. That is, the following regression equation was introduced as the standard:

$$A = 6.51\times10^{-2}\cdot\exp\left(4.75\times10^{-2}\,t\right) \quad (\rho = -0.85:\text{correlation coefficient}) \tag{29}$$

In order to evaluate the effect of deviations in the coefficient of carbonation on the safety of the bridges, we changed the deviation in three steps in both the positive and negative directions as shown in **Table 3**, and then evaluated the strength and modulus of elasticity by using Eq.(27) and Eq.(28). Finally, we calculated the degradation in safety based on the probability model for sectional force and resistance. Here, the maximum value was set higher than the maximum value measured on the "Nakaibashi" bridge, which had the highest carbonation rate of all the bridges on which we conducted material inspections. **Fig.9** shows the results

Table 3 Variation of carbonation rate δA

	δA		
Age(yrs)	27	37	52
STEP(-3)	-0.235	-0.377	-0.770
STEP(-2)	-0.157	-0.252	-0.513
STEP(-1)	-0.078	-0.126	-0.257
STEP(0)	0.0	0.0	0.0
STEP(1)	0.312	0.312	0.312
STEP(2)	0.623	0.623	0.623
STEP(3)	0.935	0.935	0.935

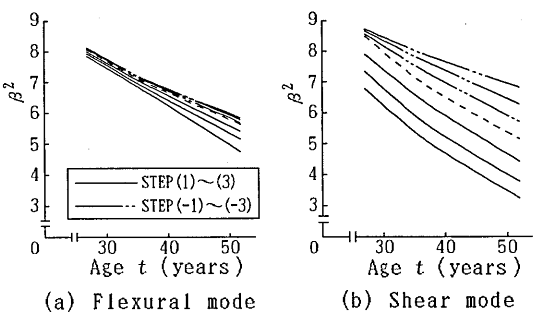
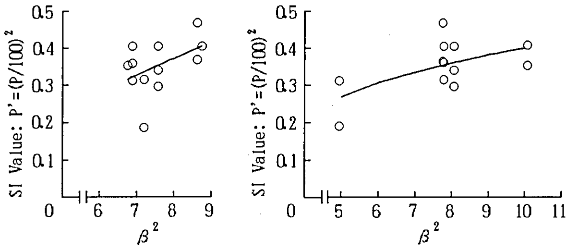


Fig.9 Influence of variation of carbonation rate on safety



(a) Flexural mode (b) Shear mode

Fig.10 Relationship between safety index and NSI value

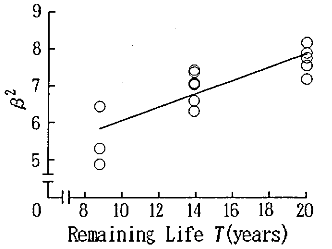


Fig.11 Relationship between remaining life and safety index

of the safety evaluation. According to these results, it can be said that while the change in safety in the flexural failure mode due to a deviation in carbonation rate increases with aging, the shear safety deviation remains constant. It can also be concluded that the change in safety for the shear failure mode is larger than that for the flexural failure mode. This is probably because the aspect of concrete strength that correlates with carbonation is principally shear resistance rather than flexural resistance. Using these results, a degradation curve including a variable for the deviation of carbonation can be evaluated as follows.

$$\beta^2 = 11.91\cdot\exp\left(-1.46\times10^{-2}t\right)+12.5\cdot\exp\left(6.00\times10^{-2}t\right)\cdot\left\{\exp\left(-2.51\times10^{-3}\delta A\right)-1\right\}$$

: flexural mode in the case $\delta A \geq 0$ (30)

$$\beta^2 = 11.91\cdot\exp\left(-1.46\times10^{-2}t\right)+15.2\cdot\exp\left(4.08\times10^{-2}t\right)\cdot\left\{\exp\left(-2.64\times10^{-3}\delta A\right)-1\right\}$$

: flexural mode in the case $\delta A < 0$ (31)

$$\beta^2 = 14.70\cdot\exp\left(-2.02\times10^{-2}t\right)+8.43\cdot\exp\left(2.20\times10^{-3}t\right)\cdot\left\{\exp\left(-2.22\times10^{-1}\delta A\right)-1\right\}$$

: shear mode in the case $\delta A \geq 0$ (32)

$$\beta^2 = 14.70\cdot\exp\left(-2.02\times10^{-2}t\right)+12.4\cdot\exp\left(6.38\times10^{-2}t\right)\cdot\left\{\exp\left(-1.22\times10^{-1}\delta A\right)-1\right\}$$

: shear mode in the case $\delta A < 0$ (33)

3.2 Remaining Life Prediction Based on Questionnaires to Bridge Engineers

The most important issue in applying the method to actual maintenance is evaluation of the safety limit which forms the basis for decisions to repair, strengthen, or replace. The safety index is defined in this paper with respect to the design load and it must be determined from a maintenance viewpoint. When introducing the new approach to an actual maintenance procedure, the easy way to do this is to calibrate the limit value against current conditions. In this research, the safety limit was evaluated according to the judgment of bridge engineers by carrying out a questionnaire survey. Here, we define serviceability as a synthetic index including durability and load carrying capability. Answers of the questionnaire are related to the serviceability expressed by 0 to 100 points defined as follows and remaining life by the number of years.

100: Initial condition where design and construction are good.
75: Slightly deteriorated, Enough integrity to remain serviceable.
50: Moderate integrity
25: Severely deteriorated or damaged, Repair or strengthening needed
0: Limit State for serviceability

Fig.10 shows the relationship between normalized serviceability index (NSI) evaluated from the questionnaires and the safety index derived from the field tests. According to these results, there is a correlation between two indices. **Fig.11** shows the relationship between remaining life as obtained from the questionnaires and the safety index. The regression equation for this correlation was obtained as follows.

$$F = \beta^2 = 0.177T + 4.33 \quad (\rho = 0.839 : \text{correlation coefficient}) \tag{34}$$

This safety limit can be evaluated in case T equal zero in Eq. (34).

$$F_{\text{limit}} = 4.33 \quad (\beta = 2.08) \tag{35}$$

4. APPLICATION TO AN EXISTING BRIDGE

This proposed degradation curve based on three bridges (the Sakurabashi, Taitabashi, and Oyasubashi bridges) was applied to another existing bridge, Kashitanibashi, which is 41 years old. The evaluation results were verified by comparison with the results of a questionnaire to bridge engineers connected with the Kashitanibashi bridge.

4.1 Outline of Kashitanibashi Bridge

The target Kashitanibashi bridge is 41 years old and has three spans (named span 1, 2, and 3 from the left bank) and has three girders (A, B, and C from the upstream side). It is a simply supported RC T-shaped girder bridge. The damage status of the girder was evaluated from visual inspections, and flexural cracks were found throughout. The maximum crack width was about 0.8mm. Water leakage due to choking of a drain pipe was found. On the other hand, damage to the slab consisted of two-directional cracks of width less than 0.3mm. Loss of cover concrete and evidence of repair work were found at span-1.

4.2 Evaluation of Degradation Curve and Its Effectiveness

Table 4 shows measurements of carbonation rate made on the Kashitanibashi bridge. It can be seen that the carbonation rate for span-2 is much higher than that for span-1. By applying the results for girder-C, which has a higher carbonation rate, to Eq. (21), a degradation curve was obtained. Adopting assumption (①) described in 2.6, the following equations are then obtained:

Table 4 Carbonation rate obtained from Kashitanibashi bridge

Span	Girder	Carbonation Rate
1	A	0.476
	C	0.542
2	A	0.865
	C	0.961

$$\beta^2 = 7.63 \cdot \exp\left\{-1.49 \times 10^{-2}(t-30)\right\} \quad : \text{for flexural mode on girder C of span 1} \tag{36}$$
$$\beta^2 = 7.85 \cdot \exp\left\{-2.07 \times 10^{-2}(t-30)\right\} \quad : \text{for shear mode on girder C of span 1} \tag{37}$$
$$\beta^2 = 7.55 \cdot \exp\left\{-1.61 \times 10^{-2}(t-30)\right\} \quad : \text{for flexural mode on girder C of span 2} \tag{38}$$
$$\beta^2 = 7.07 \cdot \exp\left\{-2.36 \times 10^{-2}(t-30)\right\} \quad : \text{for shear mode on girder C of span 2} \tag{39}$$

where, since the sample data used to evaluate these equations is from bridges ranging in age from 27 to 52 years old, it is assumed that t=30 years.

On the other hand, when adopting assumption (②), the carbonation rate for the Kashitanibashi bridge was assumed to be described by the following equations for each span on the basis of carbonation depth measurements and statistical equations for several existing bridges:

$$A = 7.73 \times 10^{-2} \cdot \exp\left(4.75 \times 10^{-2} t\right) \quad : \text{for girder C of span 1} \tag{40}$$
$$A = 1.37 \times 10^{-1} \cdot \exp\left(4.75 \times 10^{-2} t\right) \quad : \text{for girder C of span 2} \tag{41}$$

Then safety beyond the age of t_1 was calculated by assuming that t_1=t in Eq.(12) to Eq.(14). **Fig. 12** shows both the predictions by the proposed method and results based on field loading tests. It can be seen that the predicted results are on the safe side, giving conservative estimates compared with the field tests.

4.3 Updating of degradation curve based on field testing

When adopting the safety limit F_{limit} given as Eq.(35) and assuming a coefficient B of 0.2 as the standard value, optimum timing for field tests can be determined as in **Table 5**. For the shear failure mode, a simulation of carrying out the field tests at the age of 53 years on girder C of span 1 and at 45 years on girder C of span 2 was conducted. Assuming that coefficient B as evaluated from the field tests is in the range [1]$1.1B'<B<1.2B'$ and [2]$0.7B'<B<0.9B'$ (B': mean value for B in the degradation curve) , an updated prediction can be obtained as in **Table 6**. It can be seen that the dispersion of results is reduced by performing this update while the difference between assumptions (①) and (②) is greater for span 2, which has the higher carbonation rate. If coefficient B is assumed to be 0.1 and 0.3, the same tendency is obtained.

Next, the degradation curve was updated using the results of field testing carried out on the Kashitanibashi bridge at the age of 41 years, as shown in **Fig. 12** and **Table 7**. Comparing the updated remaining life with results from the questionnaire completed by bridge engineers at the bridge, it can be seen that the questionnaire result for span 2 is within the range given by assumptions (①) and (②). This demonstrates that it is possible to maintain bridges effectively according to current standards by using the proposed prediction method. However, for span 1, the values differ considerably. According to the questionnaires, the value for span 1 is lower than that for span 2, contrast with the results given by the method and obtained in material tests, as shown in **Table 4**. In this case, it can be considered that the engineers' judgment was affected by the damage to the slab of span 1, which had been repaired, but which is not a significant factor as regards the safety of the

Table 5 Optimum timing of field tests

Span		1	2
Girder		C	C
t_{limit} (years)	Flexural①	62.7	59.9
	Flexural②	61.7	58.4
	Shear ①	54.8	47.9
	Shear ②	53.2	45.5

①, ② : Assumption for concrete deterioration

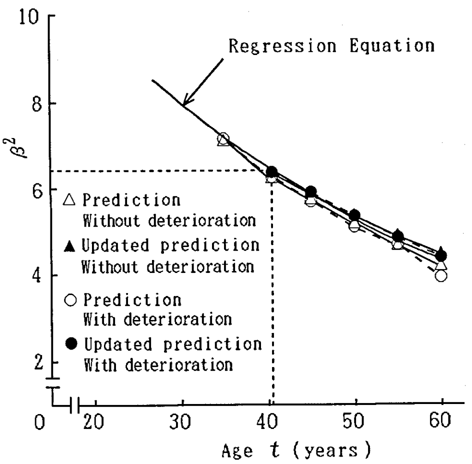
Table 6(a) Verification of safety based on field tests
(Case of the girder C of span 1 by field tests at 53 years old)

		μ(B)(X10^{-2})		σ(B)(X10^{-3})		μ(R)(X10^{-1})		σ(R)(X10^{-2})		β		Remaining Life
		Original	Updated	Original	Updated	Original	Updated	Original	Updated	Original	Updated	(years)
Shear [1]	①	2.07	2.37	3.89	0.59	6.21	5.80	5.40	0.79	2.21	2.13	2.1
	②	2.23	2.56	4.16	0.64	5.99	5.56	5.56	0.82	2.18	2.10	0.6
Shear [2]	①	2.07	1.70	3.89	1.10	6.21	6.76	5.40	1.70	2.21	2.30	10.4
	②	2.23	1.83	4.16	1.18	5.99	6.57	5.56	1.80	2.18	2.28	10.0

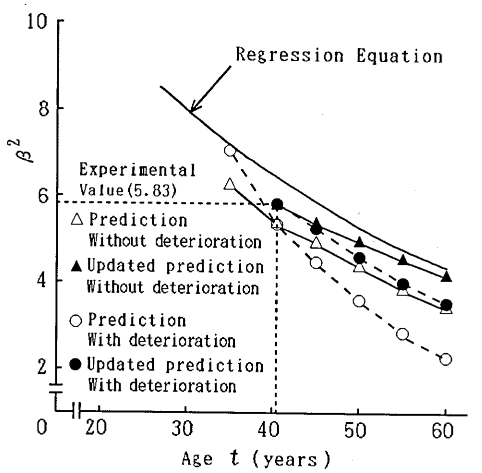
Table 6(b) Verification of safety based on field tests
(Case of the girder C of span 2 by field tests at 45 years old)

		μ(B)(X10^{-2})		σ(B)(X10^{-3})		μ(R)(X10^{-1})		σ(R)(X10^{-2})		β		Remaining Life
		Original	Updated	Original	Updated	Original	Updated	Original	Updated	Original	Updated	(years)
Shear [1]	①	2.37	2.71	4.31	0.68	7.02	6.67	4.44	0.68	2.23	2.17	3.1
	②	3.26	3.74	6.08	0.93	6.13	5.71	5.43	0.80	2.19	2.11	0.7
Shear [2]	①	2.37	1.94	4.31	1.24	7.02	7.47	4.44	1.40	2.23	2.30	10.2
	②	3.26	2.68	6.08	1.73	6.13	6.69	5.43	1.75	2.19	2.28	6.9

[1],[2] : Assumption for range of B, ①,② : Assumption for concrete deterioration



(a) Shear mode for Girder C of span 1



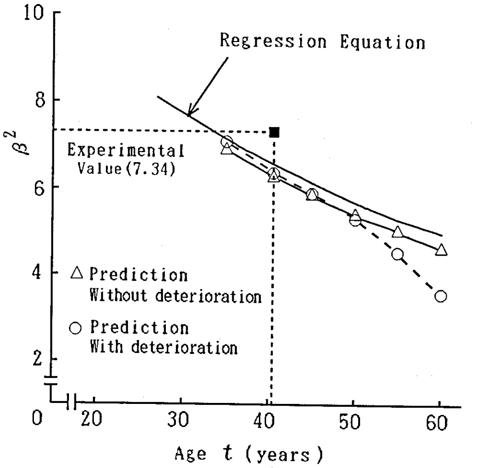
(b) Shear mode for Girder C of span 2

whole bridge system. Considering this, there is a need_to evaluate safety at the member level separately from the proposed method and to decide on the necessity of repair work.

5. CONCLUSIONS

The main conclusions obtained in this study can be summarized as follows.

(1) Statistical analysis of a safety index taking into account the influence of carbonation as a measure of material quality is proposed as a way to estimate safety based on material inspections only without the need for field loading tests.
(2) The safety limit from a viewpoint of maintenance was evaluated by surveying bridge engineers currently working on exist-ing bridges in order to calibrate to the current limit level.
(3) A method of determining the optimum timing for field loading tests is proposed to minimize the maintenance effort.
(4) A multi-step procedure consisting of visual inspections, material inspections (as periodical inspections), and field loading tests utilizing the degradation curve and Bayesian theorem is proposed.
(5) The proposed method was applied to an existing bridge, demonstrating its effective-ness



(c) Flexural mode for Girder C of span 2

Fig.12 Safety prediction and updating based on field tests

Table 7 Verification of shear safety based on field tests at 41 years old

		β		Remaining Life(years)	
		Original	Updated	Prediction	Questionnaire
Span 1 Girder C	①	2.50	2.53	21.1	13.0
	②	2.49	2.53	20.0	
Span 2 Girder C	①	2.33	2.41	17.0	15.0
	②	2.33	2.41	11.2	

①, ② : Assumption for concrete deterioration and rationality.

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