Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte

Band: 67 (1993)

Artikel: Field testing and safety evaluation of concrete bridges

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DOI: https://doi.org/10.5169/seals-51389

Field Testing and Safety Evaluation of Concrete Bridges
Essai sur site et évaluation de la sécurité des ponts en béton
Feldtest und Sicherheitsbegutachtung von Betonbrücken

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SUMMARY

This paper describes a method for safety evaluation of concrete bridges and its verification based upon field tests performed either under static loading by test trucks for application of the system identification method or under dynamic loading by falling mass for application to the modal analysis. The safety factors for flexural and shear failures were evaluated from these test results. Then, they were verified through the ultimate load test carried out in situ on the reinforced concrete main girders isolated by cutting off from the bridge system. Finally, the remaining life of the bridge was predicted by applying the fuzzy set theory which deals with the subjective information of bridge engineers.

RÉSUMÉ

Ce document décrit une méthode d'évaluation de la sécurité des ponts en béton et sa vérification d'après des essais sur place réalisés soit sous charge statique avec camions pour application de la méthode d'identification de système, soit sous charge dynamique avec chutes de masses pour application de l'analyse modale. Les facteurs de sécurité pour la flexion et la torsion ont été évalués à partir des résultats de ces essais. Ensuite, ils ont été vérifiés au cours d'un essai de charge limite mené sur le terrain sur les poutres principales isolées de l'ensemble du pont. Et à la fin, la durée de vie restante du pont a été prévue en appliquant la théorie de l'ensemble flou qui tient compte des informations subjectives des ingénieurs des ponts.

ZUSAMMENFASSUNG

Diese Arbeit beschreibt eine Methode zur Sicherheitsbewertung von Betonbrücken und eine dazugehörende Verifikation, basierend auf Feldtests, die entweder unter statischer Belastung durch Testlastwagen zur Anwendung der Systemidentifikationsmethode oder unter dynamischer Belastung durch fallende Massen für Anwendung der Modellanalyse durchgeführt wird. Die Sicherheitsfaktoren für Biege- und Bruchschäden werden auf der Grundlage dieser Testergebnissen bewertet. Danach werden sie durch einen endgültigen Belastungstest vor Ort auf Hauptträgern aus Stahlbeton, die durch Abschneiden vom Brückensystem isoliert sind, verifiziert. Schliesslich wird die Restlebensdauer der Brücke durch Anwendung der Fuzzy Set Theorie vorausgesagt, die subjektive Information von Brückeningenieuren verwerndet.

1. INTRODUCTION

It is an important problem for maintenance and rehabilitation of existing bridges to develop a method of safety evaluation such as remaining life and load carrying capacity of bridges. This paper describes a method of safety evaluation of concrete bridges in service and the verification of the evaluated results by field tests. The field tests to evaluate structural safety were performed as static loading test by test trucks for application to the system identification(SI) method, and also as dynamic loading test under forced vibration caused by falling mass for application to the modal analysis.

Both of the safety factors for flexural and shear failures and the change of dynamic behavior were evaluated from the field tests. These results were verified through the ultimate load test carried out in field on the reinforced concrete main girders isolated by cutting off from the bridge system. Finally, the remaining life of the bridge was predicted by application of fuzzy set theory which deal with the subjective information of bridge engineers. The fuzzy mapping which was determined based on questionnaire results performed on more than 20 experts was introduced for remaining life prediction of the existing bridges. A few concrete bridges on which field data have been collected are analyzed to demonstrate the applicability of this method. Through the application to the cracked reinforced concrete bridge girders, reasonable results were obtained by field tests.

2. FIELD TEST FOR STRUCTURAL SAFETY EVALUATION

2.1 Flow of Structural Safety Evaluation

Fig. 1 shows a general flow of safety evaluation and its verification for concrete bridges based on field tests. As shown in the figure, there are two types of field tests, one is non-destructive test to identify the system parameters which consist of the geometrical moment of inertia of both main girder and cross beam and Young's modulus of concrete, while the other is destructive tests such as ultimate load test and material tests to verify the evaluated results[1]. The non-destructive tests in situ are performed as static and dynamic loading tests, and deflection and acceleration are measured as mechanical behaviors. The stiffness(flexural rigidity) of each girder is identified by applying the SI method to the mechanical behaviors[2, 3], and then the section forces such as bending moment and shear force for the modeling of load variables are evaluated by structural analysis for the design load using the identified system parameters for each girder. Furthermore, the material tests are carried out for modeling of resistance variables linked with the statistical data of results of ultimate load tests which have been previously carried out on other bridges. Then, the structural safety for bending and shear failure are evaluated by calculating the safety factor, γ, safety index, β and the probability of failure, P_f. Finally, the ultimate load test is conducted in order to verify these evaluated results with the material tests.

2.2 Bridge Description

The bridges for which tests were performed are six national highway bridges as shown in Table 1. These bridges consist of five RC-T simply supported beam bridges and a RC-T continuous beam bridge, and all of the bridges are located in Hyogo prefecture, Japan. It is noted that the five simply supported bridges have almost the same bridge characteristics and history of service conditions except

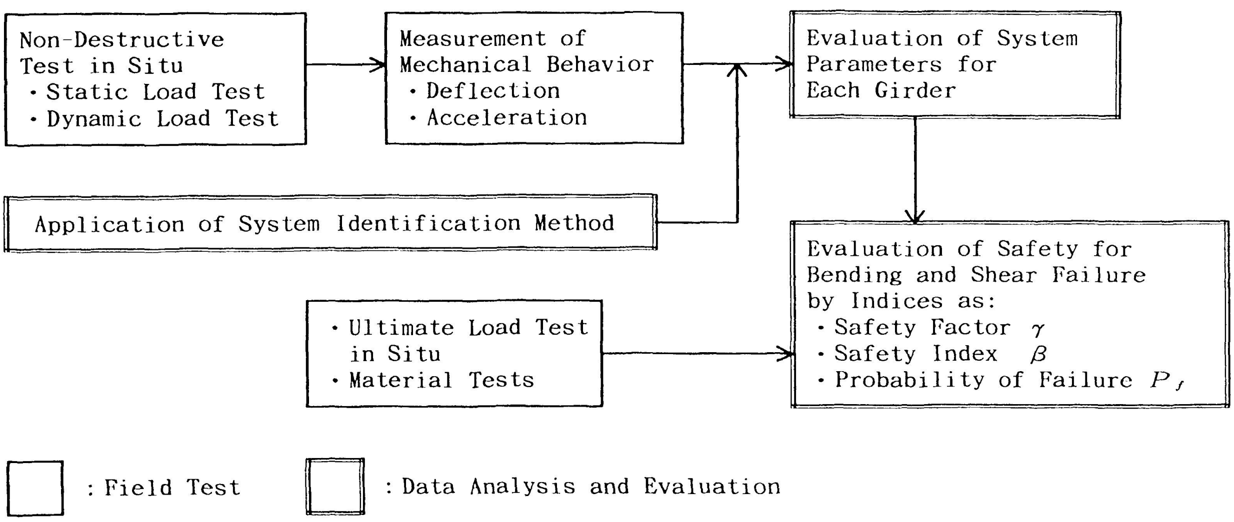


Fig. 1 Flow of safety evaluation and verification for concrete bridges

for age, number of girders, existence of cross members and type of handrail. The age of the bridges is in the region of about 40 to 60 years and the number of girders is three to five. Span length of all bridges is about 10m. Widening using an additional girder was performed on only Sakurabashi bridge in 1968. Fig. 2 illustrates the general view of Maenobashi bridge as example of the tested bridges.

2.3 Details of Field Tests

2.3.1 Non-Destructive Tests

Static loading was applied to the deck by a three-axled truck, loaded with crushed stones, with the total weight(about 20tf) being accurately measured prior to arrival on site. The truck was positioned in such a way as to cause a realistic severe loading condition on each girder as shown in Fig. 3. Positions of forced vibration by falling mass(300kgf) were arranged to obtain the various modes of vibration as shown also in Fig. 3. Typical loading procedure for the falling mass test is shown in Fig. 4. Mass dropping was carried out from about 70cm height, for ten times at the same loading point to cancel the white noise and to obtain a stable average value. Deflection and acceleration response of each girder for static and dynamic loading tests were measured with electronic deflection meters which have 1/1000mm accuracy and 10 mm capacity and ultra-small high capacity acceleration sensors which have constant frequency response of up to 700Hz and 20G capacity, respectively. Typical positions of the deflection meters and the acceleration sensors are shown also in Fig. 3. Modal analysis was applied on acceleration data to identify the modal parameters such as frequency, mode shape, damping, etc.

2.3.2 Ultimate Load Test and Material Tests

Test apparatus for the ultimate load test was specially designed to apply both of the static and dynamic load to the reinforced concrete main girders which were isolated by cutting them away from the bridge system. Fig. 5 shows the details of the loading system for the ultimate load test[1]. Static load was applied at the midpoint of the span via a 100tf capacity hydraulic jack reacting against a steel beam

Table 1 Outline of tested bridges

Bridge Name	Sakurabashi Bridge	Maenobashi Bridge	Taitabashi Bridge
Total Length	21.84m	45.80m	49.00m
Span Length	2@10.9m	5@9.16m	5@9.8m
Width	6.75m	5.50m	5.50m
Construction	1933 (Repaired in 1968)	1931	1950
Applied Spec.	1926 Edition(2nd Class)	1926 Edition(2nd Class)	1939 Edition(2nd Class)
Bridge Type	5 RC-T Simple Beam	4 RC-T Simple Beam	3 RC-T Simple Beam

Bridge Name	Nakaibashi Bridge	Oyasubashi Bridge	Aokibashi Bridge
Total Length	108.00m	45.90m	15.60m
Span Length	10@10.8m	3@14.7m	2.8+10+2.8m
Width	5.00m	7.30m	6.85m
Construction	1928	1962	1950(Repaired in 1969)
Applied Spec.	1926 Edition(2nd Class)	1956 Edition(1st Class)	1939 Edition(2nd Class)
Bridge Type	3 RC-T Simple Beam	4 RC-T Simple Beam	3 RC-T Continuous Beam

Fig. 2 General view of tested bridge (Maenobashi bridge)

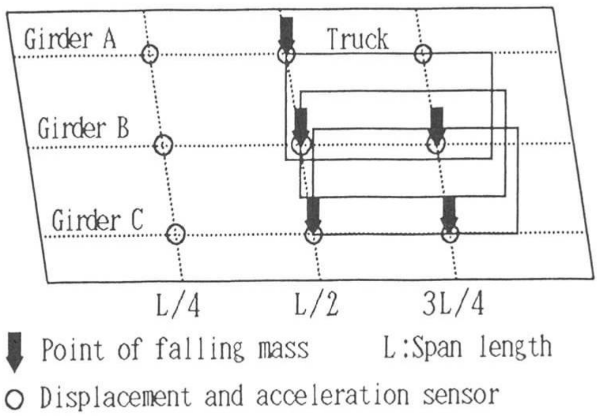
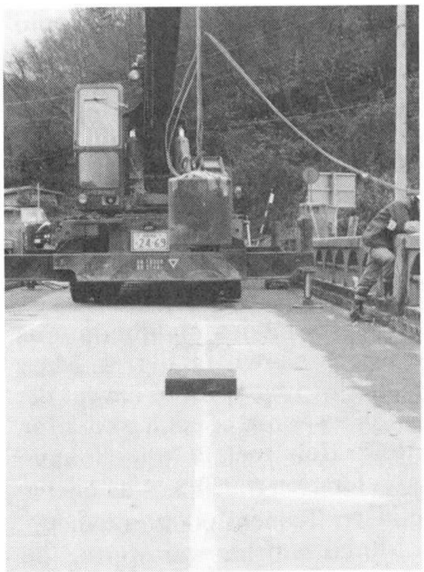


Fig. 3 Example of loading points and arrangement of sensors for static and dynamic tests



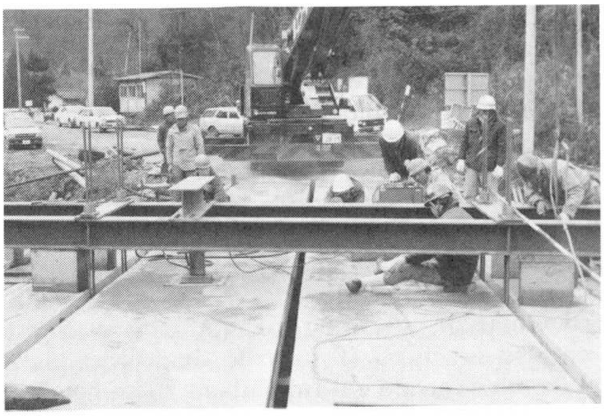


Fig. 5 Details of ultimate load test

Fig. 4 Details of falling mass (dynamic)test

embedded in the loading system. The loading scheme consisted of a series of loading-unloading cycles up to failure. The relationship between applied load and deflection at midspan was monitored using an X-Y recorder. On the other hand, a dynamic load test by falling mass at the midpoint of the span was carried out on each main girder after unloading in the static cyclic loading test, to evaluate the change in dynamic behavior up to failure. The acceleration responses were measured at a few points in the main girder. On the material tests, specimens of concrete cores and reinforcing bars extracted from all girders were tested to evaluate mechanical properties such as strength and modulus of elasticity and the depth of carbonization.

3. SAFETY EVALUATION AND ITS VERIFICATION

3.1 SI Method using Sensitivity Analysis

The SI method is one type of back analysis method, which can be used to identify system parameters such as flexural rigidity, corresponding to the degree of damage in the problem, by minimizing the error between the mechanical behavior as obtained from test and analysis. For the modeling of the target bridge in the SI method, a lumped mass(for modal analysis) model of gridform using finite beam elements possessing flexural, shearing and torsional rigidity, was applied for rationality in iterative calculation for this problem. In addition, this model has spring elements for friction restraint of rotation at the supports, corresponding to the progress of damage in the support region[4].

In the procedure of the SI method in this study, a sensitivity analysis of damage to mechanical behavior and the sequential linear programming(SLP) method were applied[4]. In this procedure, the objective function was defined as minimizing the total squared error between the mechanical behavior obtained from field tests and analysis. For the dynamic problem, the normalized objective function was defined as follows:

$$F = W_1\left(\frac{\mu_p}{\mu_p^m} - 1\right)^2 + W_2\sum_{k=1}^{n}\left(\frac{Z_{pk}}{Z_{pk}^m} - 1\right)^2 \rightarrow min \qquad (1)$$

where, p is the order of normal vibration, n is the number of measuring points, μ, μ^m are the eigen values obtained from analysis and field tests, respectively, Z, Z^m are the normalized modes of vibration, and W_1, W_2 are weights for the eigenvalue and vibration mode. Here, it is assumed that $W_1 = 1.00$, $W_2 = 1/n$.

For static loading, the objective function can be expressed by:

$$F = \sum_{k=1}^{n}\left(\frac{\gamma_k}{\gamma_k^m} - 1\right)^2 \rightarrow min \qquad (2)$$

where, γ, γ^m are deflections obtained from analysis and measurement, respectively.

Following this, identification of design variables can be performed by applying the SLP method using the objective function and its derivative for the design variables. Fig. 6 shows the flow of the SLP method for the dynamic problem. In the first step, the initial values of design variables such as flexural rigidity of the main girder and the spring coefficient of rotation of the support, are assumed, and modal parameters such as eigen values and vibration mode are evaluated by analysis. Next, linearization of the objective function is carried out within the region of movement limits for design variables, using Eq. (1), and a search for the minimum point of the objective value is tried using the simplex method. In the event that the change of design variables exceeds the movement limits, reanalysis of modal parameters and restart of the search for the minimum values from updated initial values are executed in the same procedure iteratively up to the stage in which the objective function is within the region of allowable limit.

3.2 Evaluation of Load Carrying Capacity

For material tests, the specimens of concrete core and reinforcing bar extracted from the main girders of the six existing bridges were tested to evaluate the deterioration of material. It was found that the compressive strength and modulus of elasticity of concrete were evaluated to be very low compared with the design value, and the degree of carbonization of concrete was very great. However, the effective section area of reinforcing bars providing tensile strength maintained their initial value, although surface corrosion was detected. Since the carbonization of concrete is considered to be the index of deterioration which includes the effects of quality of material and construction standards, traffic loading condition, permeability of slab and web concrete, acid environment, and so on, the relationship between the depth of carbonization and the compressive strength of concrete, as shown in Fig. 7, is suitable for the evaluation of the load carrying capacity considering the effects of these factors.

Ultimate load tests of main girders were carried out on five existing bridges, and results of these tests are summarized in Table 2, compared with calculated values based on the evaluation equations defined in the specification, and using compressive strength obtained from material tests. In this case, the load carrying capacity was defined to be bending moment at yield point of reinforcing bar for bending failure and shearing force at yield point of stirrup for shear failure. The results show that the calculated values give good agreement with the results of field tests, except for the value of bending failure on Taitabashi bridge which was affected by the bond failure between the reinforcing bars and concrete, and the value of shear failure on Sakurabashi Bridge, including the effect of scatter in the section area of the compression zone.

3.3 Modeling of Resistance and Load Variables

In the safety evaluation method, the probability model for load effect and load carrying capacity should be constructed considering the scatter of data and error in evaluation. The probability model for load carrying capacity can be evaluated by considering the correction coefficient corresponding to the ratio of the value obtained from the ultimate load test to that from analysis. The correction coefficient a is assumed to be a random variable characterized by the normal distribution, $N(\mu_a, \sigma_a)$ and then, the load carrying capacity for bending and shear failure can be expressed by[4]:

$$N(\mu_{Mut}, \sigma_{Mut}) = N(\mu_a M_{ucal}, \sigma_a M_{ucal}), \; N(\mu_{Sut}, \sigma_{Sut}) = N(\mu_a S_{ucal}, \sigma_a S_{ucal}) \tag{3}$$

where, M_{ucal} and S_{ucal} are calculation results of the load carrying capacity for bending failure and shear failure, respectively, and M_{ut} and S_{ut} are results of ultimate load tests for bending failure and shear failure, respectively. The coefficient, $N(\mu_a, \sigma_a)$ can be determined through comparison of estimated and measured values for load carrying capacity, as shown in Table 2 and Table 3.

On the other hand, for the evaluation of the probability model for load effect, the mean value, μ_S and standard deviation, σ_S of the section force, S for total weight of the three-axled truck, $W = N(\mu_W, \sigma_W)$ can be expressed by the following equation using the influence surface[4]:

$$\left.\begin{aligned} \mu_S &= \mu_W \left(0.1 \textstyle\sum (\eta_L + \eta_R)_F + 0.4 \sum (\eta_L + \eta_R)_R\right) \\ \sigma_S &= \sigma_W \left(0.1 \textstyle\sum (\eta_L + \eta_R)_F + 0.4 \sum (\eta_L + \eta_R)_R\right) \end{aligned}\right\} \tag{4}$$

Here, the distribution of the total weight of three-axled truck was assumed to be as given in the results of research by Hanshin Expressway Public Corporation [5].

In this way, the safety factor, γ, safety index, β and probability of failure, P_f can be evaluated by the

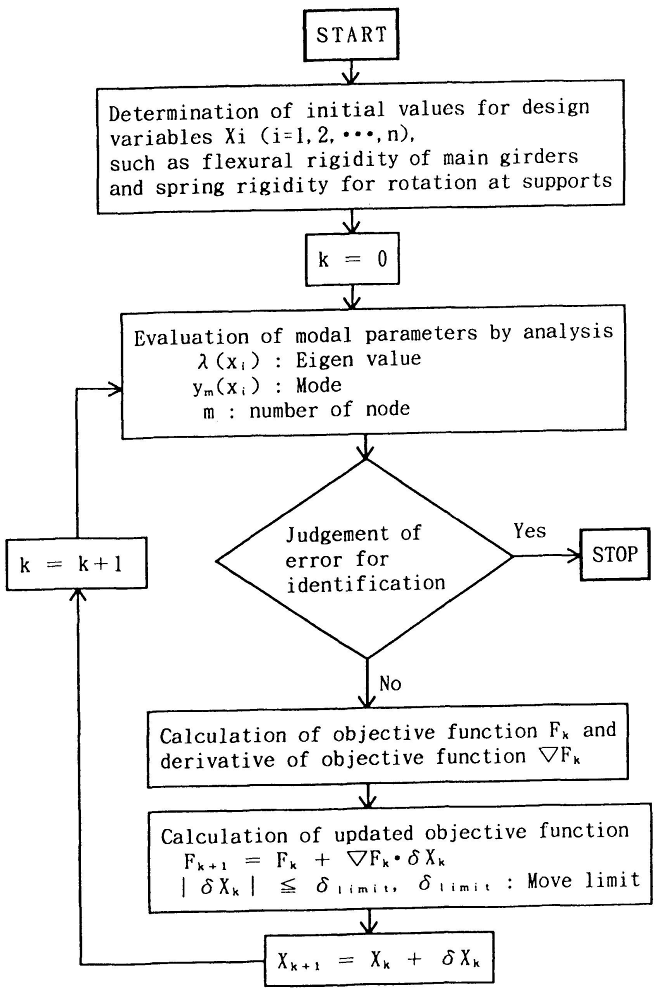


Fig. 6 Flow of sequential linear programming (for dynamic problem)

Table 2 Comparison of estimated and measured values for load carrying capacity

(a) Case of bending failure (tf•m)

Bridge Name	Span No.	Girder	M_{ucal}	M_{ut}
Maenobashi	1	C	73.28	78.22
	2	B	73.28	74.15
	2	C	73.28	69.82
Taitabashi	1	B	78.51	34.46
	2	B	79.24	41.30
	3	B	79.77	56.23
Oyasubashi	3	B	318.4	388.78

(b) Case of shear failure (tf)

Bridge Name	Span No.	Girder	S_{ucal}	S_{ut}
Sakurabashi	1	C	16.43	12.49
	1	D	16.43	9.92
Nakaibashi	1	B	18.20	22.29
	2	B	25.85	25.59
	3	B	26.63	26.59
Oyasubashi	1	B	57.91	62.49
	1	C	56.20	64.09
	2	B	53.61	75.24
	2	C	54.88	62.10

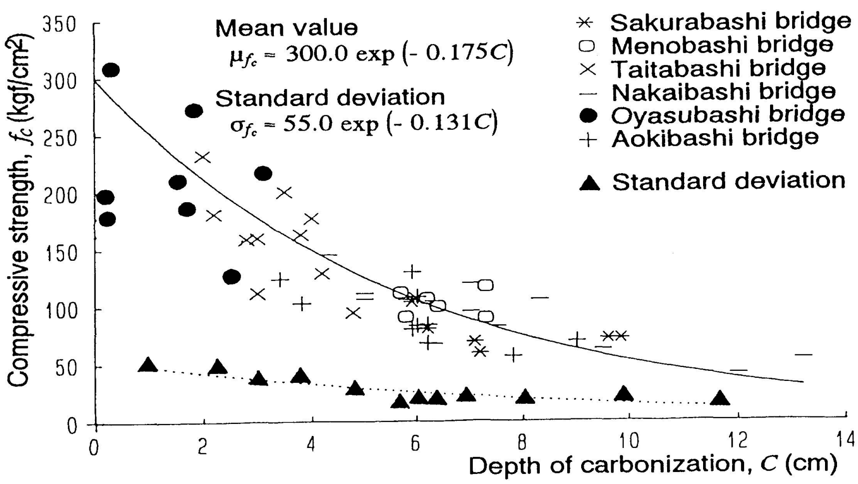


Fig. 7 Relationship between compressive strength and depth of carbonization for concrete

Table 3 Ratio of measured value to estimated value for load carrying capacity

	M_{ut}/M_{ucal}	M_{ut}/M_{ucal} (ex. Taitabashi)	S_{ut}/S_{ucal}
Mean Value	0.845	1.063	1.034
Standard Deviation	0.294	0.115	0.239

following equations:

$$\gamma = (\mu_{Fu} - 1.1 \cdot \mu_{Fd}) / \mu_{Fl}, \quad \beta = (\mu_R - \mu_S) / \sqrt{\sigma_R^2 + \sigma_S^2}, \quad P_f = P((R\text{-}S) < 0) \qquad (5)$$

3.4 Evaluation Results and Verification

The proposed method for structural safety evaluation based on field tests was applied to the existing bridges as shown in Table 1. The flexural rigidity identified by applying the SI method to results of field tests, and the safety evaluation based on safety factor, γ, safety index, β and probability of failure, P_f are summarized in Table 4. The reduction of the flexural rigidity of the oldest bridge, Nakaibashi bridge, was found to be remarkable compared with the design value, that is, the detected degree of damage for this bridge is very large. From the comparison between the safety evaluation for bending failure and shear failure, it can be judged that the failure mode for Aokibashi bridge would be of the bending type and that for Sakurabashi and Nakaibashi bridges would be of the shear type. In particular, since the probability of failure for Nakaibashi bridge is extremely large compared with the standard for the initial condition, of which the range is about 10^{-3} to 10^{-5}, the safety condition of this bridge seems to be critical from the bridge maintenance viewpoint.

The comparative study between the above mentioned evaluation and the results of the ultimate load test or material tests, can be considered to be available for the verification of suitability and utility of the proposed method. Firstly, for the girder *B* of Nakaibashi bridge, of which the degree of damage was estimated to be the largest of all tested bridges, both results of Table 4 and the ultimate load test can be seen to show that the failure mode of this girder would be of the shear type. Similarly, for the girder of Taitabashi bridge which has the second highest grade of damage of all tested bridges succeeding Nakaibashi bridge, both results of Table 4 and the ultimate load test suggest that the failure mode would be of the bending type. Since remarkable damage was not detected in the evaluation for inside girder of Oyasubashi bridge, the possibility and verification of bending failure or shear failure cannot be distinguished exactly. Accordingly, the prediction of the failure mode at the stage of heavy damage can be considered to be appropriate. In addition, the results of the SI method and safety evaluation shown in Table 4 can be found to be adjusted by the results of material tests, where compressive strength and modulus of elasticity of concrete for Nakaibashi bridge are lower than half the design values. For Aokibashi bridge, compressive strength and modulus of elasticity of concrete were about half the design values and the order of the resulting values of the three girders *A*, *B* and *C* was *A* > *B* > *C*. Therefore, the results of the SI method shown in Table 4 can be seen to be reasonable.

Table 4 Summary of structural safety evaluation results for existing concrete bridges

Bridge Name	Girder	Flexural Rigidity ($\times 10^4$ tf·m)		Safety Evaluation					
				Bending Failure			Shear Failure		
		Design Value	Estimated Value	γ_M	β_M	P_{fM}	γ_S	β_S	P_{fS}
Sakurabashi	A	7.66	6.14	3.88	2.173	1.483×10^{-2}	2.53	1.934	2.631×10^{-3}
	B	7.01	2.88	8.22	2.789	2.626×10^{-3}	4.31	2.543	5.451×10^{-3}
	C	7.01	1.68	16.12	3.174	7.502×10^{-4}	8.94	3.318	4.494×10^{-4}
	D	7.66	4.73	5.64	2.422	7.685×10^{-3}	4.39	2.525	5.734×10^{-3}
	E	13.22	7.80	3.75	2.097	1.793×10^{-2}	2.68	2.280	9.872×10^{-3}
Taitabashi	A	8.38	5.00	4.25	2.424	7.665×10^{-3}	3.64	2.594	4.740×10^{-3}
	B	10.03	6.23	5.34	2.491	6.363×10^{-3}	5.05	2.806	2.508×10^{-3}
	C	13.25	6.63	3.68	2.259	1.194×10^{-2}	3.37	2.483	6.506×10^{-3}
Nakaibashi	A	25.90	7.91	4.80	2.380	8.664×10^{-3}	1.80	1.326	9.238×10^{-2}
	B	17.21	9.11	5.49	2.414	7.886×10^{-3}	1.97	1.383	8.332×10^{-2}
	C	25.90	8.93	4.31	2.265	1.176×10^{-2}	1.67	1.185	1.181×10^{-1}
Oyasubashi	A	101.04	110.25	7.68	2.567	5.103×10^{-3}	7.02	2.899	1.846×10^{-3}
	B	88.32	92.27	16.32	2.877	1.998×10^{-3}	11.55	3.013	8.355×10^{-4}
	C	88.32	86.41	16.02	2.880	1.980×10^{-3}	14.00	3.050	1.128×10^{-3}
	D	101.04	89.68	8.41	2.638	4.158×10^{-3}	7.96	2.895	1.272×10^{-3}
Aokibashi	A	9.98	5.74	4.23	2.367	8.916×10^{-3}	4.58	2.797	2.561×10^{-3}
	B	10.87	4.48	5.86	2.560	5.225×10^{-3}	6.12	2.866	2.058×10^{-3}
	C	18.50	3.21	5.29	2.578	4.973×10^{-3}	5.91	3.005	1.342×10^{-3}

4. REMAINING LIFE PREDICTION

Based upon the above mentioned evaluated results, it is of interest to predict the remaining life of the bridges. However, the practical procedure for remaining life prediction involves an uncertainty scheme such as engineering judgement based on technical knowledge with professional experience. Therefore, the fuzzy set theory was introduced in a process of remaining life prediction for dealing with the subjective uncertainty.

The relationship between the integrity(soundness), S and the remaining life, t is assumed based on questionnaire results for the serviceability of actual bridges(Maenobashi bridge, Taitabashi bridge and Nakaibashi bridge). The questionnaires were performed on more than 20 experts for each bridge. The remaining life, t can be assumed as in the following equation[6]:

$$t = 3.03 \times 10^{-1}\, S^{0.98} \tag{6}$$

Since the deviation of the data from the equation is very large, the treatment of the subjective uncertainties included in this evaluation should be considered. In order to represent these uncertainty, the "kernel" of log-normal distribution is introduced for membership function of fuzzy set theory[6].

$$\mu_R(s, t) = \frac{t_m}{t} \exp\left[-\frac{1}{2}\left(\frac{\ln[t] - \lambda_S}{\phi}\right)^2\right] \quad \text{where,} \quad \lambda_S = \ln[t_m] = \ln\left[\left(3.03 \times 10^{-1}\right) S^{0.98}\right] \tag{7}$$

Next, the membership function for the remaining life, t is related to the membership function for the integrity, S by Fuzzy synthesis with aid of fuzzy relation, R[6]. Relative prediction of remaining life for the bridges is performed by integrating the forecast of respective remaining life, T_i based upon the corresponding membership function obtained from evaluated results for each judgement factor.

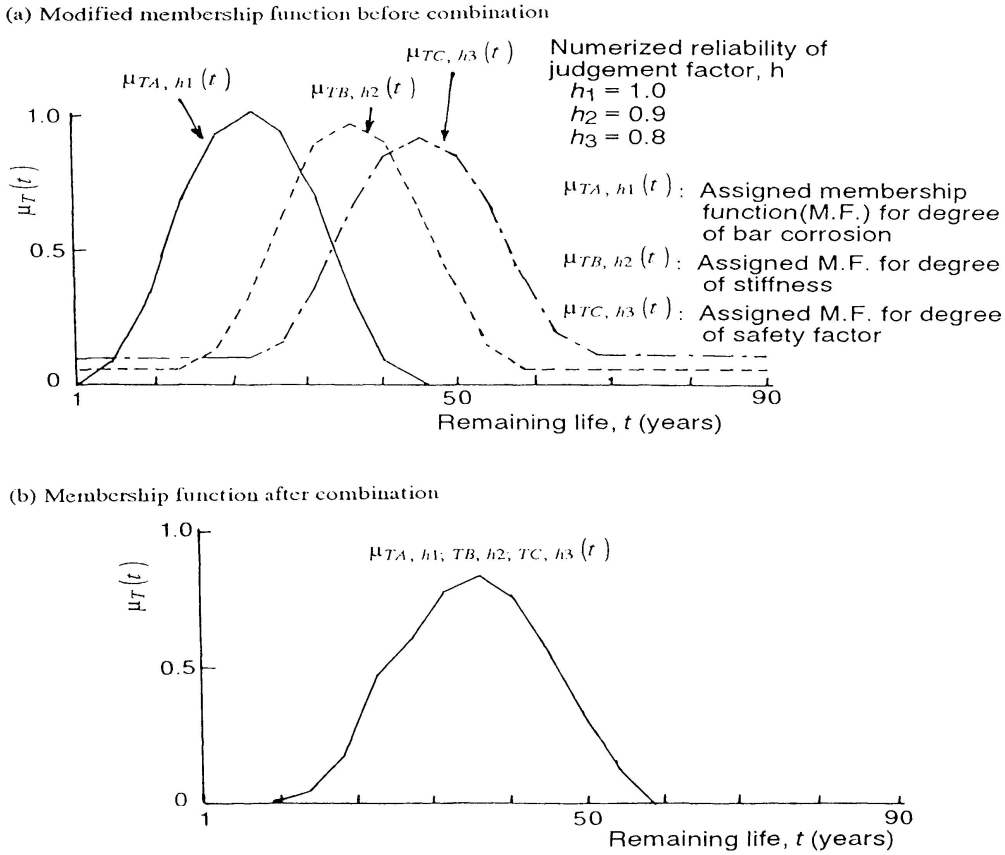


Fig. 8 Prediction of remaining life

As an example, Fig. 8 shows the result of calculating the remaining life for Taitabashi bridge using the "degree of bar corrosion", "stiffness of girder" and "safety factor" evaluated by field inspections and tests as a measure of the integrity. From this figure, the remaining life of the bridge is predicted to be about 35 years corresponding to the peak value of the membership function after combination. Furthermore, the predicted values for Sakurabashi bridge, Maenobashi bridge and Nakaibashi bridge are roughly about 15 years, 18 years and 18 years, respectively.

5. CONCLUSIONS

The major part of bridge diagnosis which is the kernel of the systematization for bridge maintenance is to develop a method of safety evaluation on items such as remaining life and load carrying capacity. The main conclusions obtained in this study can be summarized as follows:
(1) A few examples of the field testing and the measuring procedures, the evaluation methods of both the safety factors, including the safety index and the probability of failure, for flexural and shear failure and the verification concept of the evaluated results by field tests were performed on six reinforced concrete bridges.

(2) The prediction of remaining life of concrete bridges in service, which is a major problem faced in the process of bridge maintenance, can be quantitatively evaluated based on the evaluated results for the judgement factors. The relation among the factors are obtained through the fuzzy relations in the questionnaire results performed on experts.

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