Residual Service Life Prediction for Bridges Based on Critical Life Curves

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Residual Service Life Prediction for Bridges Based on Critical Life Curves

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Abstract: A residual service life prediction method based on critical life curves of selected bridge members is presented in this paper for bridges in service. The lowest target reliability index is selected as the criterion for the end of the lifecycle of bridges based on bridge design codes. Based on various assessment reference periods, a simple stochastic process model of resistance is adopted to evaluate corresponding resistance reduction coefficients related to the adopted lowest reliability index. By proposing the critical life curves based on flexural resistant capacity, the paper established a new method to predict the residual service life of bridges. A case study is used to demonstrate the proposed approach. The authors inspected 11 pieces of 13-m hollow-core slab beams that had been in use for 11 years. The accuracy of the prediction method was verified through a field condition investigation and a lab destructive test on a beam dismantled from the studied bridge. **DOI: 10.1061/(ASCE)CF.1943-5509.0001031.** *© 2017 American Society of Civil Engineers.*

Introduction

Concrete structures, over time, suffer varying degrees of performance degradation as a result of reinforcement corrosion and the related damage induced into concrete, which reduces the load capacity and system reliability of the structure. Therefore, it is imperative to evaluate the safety and predict the residual service life of concrete bridges. Accurate residual service life prediction for bridges also serves as an important basis for making rehabilitation decisions (Marchand and Samson 2009; Sohanghpurwala 2006; Markeset 2009). Among all the methods for residual service life prediction, the one based on time-varying reliability is most commonly used. Okasha and Frangopol employed an incremental nonlinear finite-element analysis to make lifecycle predictions and service life evaluations (Okasha and Frangopol 2010). Strauss et al. (2013) computed time-discrete chloride profiles of concrete structures and quantified the future performance of the structure by adopting nonlinear analysis and reliability assessment. Several efforts have been made to predict the service life of concrete bridges (Cheung et al. 2009; Liang et al. 2009; Paulsson-Tralla 2010; Enright and Frangopol 1998; Su et al. 2013; Sobhani and Ramezanianpour 2011).

The methods mentioned above provide more advanced and complicated approaches to evaluate the residual life for in-service bridges. For example, using Okasha and Frangopol's approach (2010), one has to build a finite-element model, calculate live load multipliers, and perform a response surface analysis and a Latin hypercube sampling for resistance computations. The approach proposed by Strauss et al. (2013) employed one package containing all the necessary elements for a safety analysis and reliability assessment (SARA) that is not always available for general engineering practice. Therefore, engineers at the local highway administration level demand a more convenient approach for routine evaluations of the residual life of bridges. Hence, the authors developed life prediction curves suitable for practical engineers to forecast the residual life of bridges in service.

To simplify the calculation of the time-varying reliability index, the authors previously investigated the residual life of reinforced concrete structures based on their shear strength and established shear strength critical life curves by calculating the limits of relative resistance reduction coefficients. These limits represent the minimum reliability indices for different assessment periods (Wang and Li 2010). This method was extended in this paper to calculate the flexural strength critical life curves of reinforced concrete structures based on the test of a corroded reinforced hollow-core beam bridge. The method was derived based on the minimum reliability index for different years in service, which was verified by practical engineering applications.

Reliability Analysis Based on Critical Life Curves

Theoretical Basis of Critical Life Curves

A structural reliability analysis normally involves a fully stochastic process model that is difficult to develop for general engineering applications. Because the reliability analysis method based on random variables is widely applied, the fully stochastic process model can be simplified as a random variable model (Wang and Li 2010; Wang 2009; Zhao 2001). Therefore, the performance function of a bridge in service can be developed by using a first-order second-moment (FOSM) method with random variables of basic structural performance functions. For a newly built highway bridge, dead load G and live load Q are dominating loads. The limit state equation can be written as [Wang 2009; GB/T 50283 (AQSIQ and MOHURD 1999)]

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$$Z = g(R, S_G, S_Q) = R - S_G - S_Q \quad (1)$$

where Z = state of safety; R = total resistance that follows a normal distribution; S_G = dead load effect that obeys a normal distribution; and S_Q = maximum live load effect over the design reference period (T_0) and it obeys a Type I Gumbel distribution [GB/T 50283 (AQSIQ and MOHURD 1999)].

The resistance of a bridge in service decays over time. Assuming the resistance varies with a simple stochastic process and can be modified by a deterministic function $k(t)$ at time T (the time period the bridge has been in service), then

$$Z = k(t)R - S_G - S_{QT} \quad (2)$$

where $k(t)$ = resistance reduction function and S_{QT} = maximum live load effect at time T.

Given T and the corresponding live load effect S_{QT}, there is a certain $k(t)$ that matches the lowest reliability requirement according to Eq. (2). Then, if the probability models of the initial resistance and the live and dead load effects are known, a $K - T$ curve can be established based on the minimum reliability index. This $K - T$ curve is the critical life curve. The intersection of the actual resistance attenuation curve and the critical life curve is the critical life of the structure, and the difference between the critical life and the service period is the residual life of the structure.

The residual life of a structure is predicted by first defining the minimum reliability index and then calculating the actual reliability index $\beta(T)$ at service time T to develop a $\beta - T$ curve. The time when the actual reliability index intersects the minimum reliability index is the critical life of the structure. It is noted that the national design standard (GB/T 50283-1999) applies throughout this 84 paper.

Critical Life Criterion

Because the reliability index $\beta(T)$ varies with service life T, determining a minimum reliability index is the basis for structural service life predictions. Two criteria for selecting the minimum reliability index have been considered: one is to use $0.85\beta_0$ and the other $\beta_0 - 0.5$ (Zhang et al. 2003; Wang 2009), where β_0 is the target reliability index defined in the *Unified standards for reliability design of highway engineering structures* [China's national standard GB/T 50283 (AQSIQ and MOHURD 1999)]. The criterion $\beta_0 - 0.5$ is selected in this paper based on previous research results (AASHTO 2011; Moses 2001). According to China's current standard GB/T 50283 (AQSIQ and MOHURD 1999), the maximum reduction of reliability indexed from β_0 are 0.705, 0.63, and 0.555, for Level 1, 2, and 3 structural components, respectively. Therefore, a reduction of 0.5 is relatively conservative.

Simplified Analysis of Resistance Random Probability Model

The resistance of a structure varies with time and follows a nonstationary random process (Torres-Acosta et al. 2007; Duprat 2007; Enright and Frangopol 1998). To simplify the calculation, assume that the change of resistance of a reinforced concrete bridge with time follows a single probability model that can be expressed as (Wang 2009; Enright and Frangopol 1998)

$$R(t) = R_0\varphi(t) \quad (3)$$

where $R(t)$ = resistance at time t; $\varphi(t)$ = function of resistance attenuation; and R_0 = initial resistance when $t = t_0$. $\varphi(t)$ is correlated to factors such as material properties, environment, and maintenance conditions. For simplification, it is assumed that the mean value of the resistant capacity changes over the service life of the structure while the standard deviation does not change

$$\mu_{R(t)} = \mu_{R_0}\varphi(t), \qquad \delta_{R(t)} = \delta_{R_0} \quad (4)$$

where $\mu_{R(t)}$ = average resistance at time t; μ_{R_0} = average initial resistance; $\delta_{R(t)}$ = coefficient of variation at time t; and δ_{R_0} = initial coefficient of variation. The initial average resistance μ_{R_0} is calculated according to an applied design code, as expressed as follows:

$$\mu_{R_0} = \kappa_R \cdot R_\kappa \quad (5)$$

where R_κ = nominal resistance of the structure calculated according to its material properties and geometric parameters and κ_R = statistical parameter given by the design code GB/T 50283 (AQSIQ and MOHURD 1999). Substituting Eq. (5) into Eq. (4) gives

$$\mu_{R(t)} = \mu_{R_0}\varphi(t) = \kappa_R \cdot R_\kappa \cdot \varphi(t) = \kappa_R \cdot R_\kappa(t) \quad (6)$$

where $R_\kappa(t)$ = nominal resistance at time t.

According to Eqs. (4) and (5), the resistance adjustment parameter κ_R is specified by the design code for a given time t, and the reduction of resistance can be considered as the reduction of initial resistance R_0 by applying an attenuation function. The resistance attenuation function of a corroded reinforced concrete flexural beam can be expressed as (Wang 2009)

$$\varphi(t) = 1 - \eta(t) = 1 - 2\xi t + \xi^2 t^2 \quad (7)$$

where $\eta(t)$ = corrosion ratio of the reinforcement steel area; ξ = corrosion ratio of reinforcement diameters, defined as $\xi = \Delta d/d$, where Δd is the corrosion rate of steel in terms of the change of effective diameters; and d = original diameter of the reinforcement. Eq. (7) can be used to predict the service life of reinforced concrete structures based on the change of the effective area of steel according to the rate of corrosion (Zhao 2001; Enright and Frangopol 1998).

Stochastic Model of Load Effects

Dead Load Effect

The dead load effect S_G is assumed to be normally distributed; thus its statistical parameters are given by

$$\mu_{SG} = \kappa_{SG} S_{GK}, \qquad \sigma_{SG} = \delta_{SG}\mu_{SG} \quad (8)$$

where μ_{SG}, σ_{SG}, S_{GK}, and δ_{SG} = mean, variance, standard value, and coefficient of variation, respectively, and κ_{SG} = ratio of the measured dead load effect to the standard dead load effect.

The variation of the dead load effects from its standard value obeys following random variable probability model [GB/T 50283 (AQSIQ and MOHURD 1999)]:

$$F_{SG}(x) = \frac{1}{0.0437 S_{GK}\sqrt{2\pi}} \int_{-\infty}^{+\infty} \exp\left(-\frac{x - 1.0148 S_{GK}}{0.0038 S_{GK}^2}\right) dx \quad (9)$$

where x = random variable of the dead load, and F_{SG} is its probability distribution. The standard dead load S_{GK} is calculated by using the nominal dimensions and material properties defined in the applied design code. To determine the variation of actual dead loads from their standard values and verify the distribution model, the self-weight of 1,488 pieces of beams and slabs and the thickness of bridge deck pavements of 36 bridges were measured within a nationwide investigation in China. The dead load effect was found to be distributed normally with $\kappa_{SG} = 1.0148$ and

Table 1. Probability Distribution Functions and Statistical Parameters for Maximum Load Effects over the Design Life (Data from AQSIQ and MOHURD 1999)

Operation state	Load effects	κ_{SQ}	δ_{SQ}	Probability distribution function
Normal operation state	Moment	0.6861	0.1569	$F_M(x) = \exp\left\{-\exp\left[-\frac{(x-0.6376S_{QK})}{0.084S_{QK}}\right]\right\}$
	Shear	0.6083	0.1581	$F_M(x) = \exp\left\{-\exp\left[-\frac{(x-0.5650S_{QK})}{0.0750S_{QK}}\right]\right\}$
Intensive operation state	Moment	0.7995	0.0862	$F_M(x) = \exp\left\{-\exp\left[-\frac{(x-0.7685S_{QK})}{0.0537S_{QK}}\right]\right\}$
	Shear	0.7187	0.0769	$F_M(x) = \exp\left\{-\exp\left[-\frac{(x-0.6938S_{QK})}{0.0431S_{QK}}\right]\right\}$

Note: S_{QK} = standard load effect obtained from the design code; $\kappa_{SQ} = S_Q/S_{QK}$, where S_Q is the mean value of the measured live load effect under a normal or intensive operation state; σ_{SQ} = coefficient of variance of live load effects. The design life T in this table is set as 100 years.

$\delta_{SG} = 0.0431$ [GB/T 50283 (AQSIQ and MOHURD 1999)]. This model was used for the dead load effect distribution in this paper.

Live Load Effect

A maximum live load effect stochastic model is exploited to calculate the residual life of a structure. The model is considered to obey a Type I Gumbel distribution, which can be obtained from experimental tests or calculated according to the applied design codes. Hence, the stochastic process of the live load effect $S_Q(t)$ can be simplified as the change of the maximum live load's random variables. The random reliability analysis method can then be used to predict the residual life of a structure at a given service time.

The probability distribution function of $S_{QT1}(x)$, the maximum live load effect during the design reference period T_1, can be expressed as

$$F_{SQ}(x) = \exp\left[-\exp\left(-\frac{x-u_i}{\alpha_i}\right)\right] \tag{10}$$

$$F_{SQT1}(x) = [F_{SQ}(x)]^{m_{T1}} = \exp\left[-\exp\left(-\frac{x-u_{T1}}{\alpha_{T1}}\right)\right] \tag{11}$$

where x = random variable of the live load effect; $F_{SQ}(x)$ = truncated distribution of the live load effect stochastic process, which is calculated by the maximum live load effect probability distribution function; μ_i and α_i = parameters of truncated distribution of the live load effect; m_{T1} = number of occurrences of the maximum live load effects during the design reference period T_1; μ_{T1}, α_{T1} = parameters of maximum live load effect S_{QT1} during the design reference period T_1, with $\alpha_{T1} = \sqrt{6}\sigma_i/\pi$, where σ_i is the standard deviation of the Gumbel distribution for the truncated distribution of live loads, and $u_{T1} = u_i + \alpha_i \ln m_{T1}$, where $u_i = m_i - 0.5772\alpha_i = m_i - 0.45\sigma_i$, $\alpha_i = \alpha_{T1}$, and m_i is the mean value of the truncated distribution of the live load effect. If the time interval is 1 year, one can get $m_{T1} = T_1$, $m_T = T$, and the mean value μ_{SQT1} and deviation σ_{SQT1} of S_{QT1} can be further derived

$$\mu_{SQT1} = \mu_i + 0.5772\alpha_i + \alpha_i \ln T_1 = \mu_{SQT} + \alpha_i \ln T_1/T \tag{12}$$

$$\sigma_{SQT1} = 1.28255\alpha_i = \sigma_i \tag{13}$$

$$\delta_{SQT1} = \frac{\sigma_{SQT1}}{\mu_{SQT1}} = \frac{1.28255\alpha_i}{\mu_i + 0.5772\alpha_i + \alpha_i \ln T_1} \tag{14}$$

Based on Eqs. (10)–(14), and the probability distribution functions and statistical parameters defined in Table 1 according to China's unified highway reliability design standard GB/T 50283 (AQSIQ and MOHURD 1999), the statistical parameters of the maximum live load effects (bending moment for this study) at different service years are calculated and the results are given in Table 2.

Table 2. Statistical Parameters of the Maximum Live Load Effects (Moment) at Various Years of Service

Assumed year in service T_1 (years)	Normal operation state		Intensive operation state	
	κ_{SQ}	$\sigma_{SQ} \cdot S_{QK}$	κ_{SQ}	$\sigma_{SQ} \cdot S_{QK}$
10	0.4927	0.1077	0.6759	0.0689
20	0.5509	0.1077	0.7131	0.0689
30	0.5850	0.1077	0.7348	0.0689
40	0.6091	0.1077	0.7503	0.0689
50	0.6279	0.1077	0.7623	0.0689
60	0.6432	0.1077	0.7721	0.0689
70	0.6561	0.1077	0.7803	0.0689
80	0.6674	0.1077	0.7875	0.0689
90	0.6772	0.1077	0.7938	0.0689
100	0.6861	0.1077	0.7995	0.0689

Note: Design life of the bridge in the case study of this research is 100 years, so the design life used in calculating Table 2 is taken as 100 years and the parameters for the 100-year period are the same as those in Table 1.

Assuming for example a normal operation state, with the design life T set to 100 years and $\mu_{SQT} = 0.6861S_{QK}$, when the service year $T_1 = 10$,

$$\begin{aligned}\mu_{SQT1} &= \mu_{SQT} + \alpha_i \ln T_i/T \\ &= 0.6861S_{QK} + 0.084S_{QK}\ln(10/100) = 0.4927S_{QK}\end{aligned} \tag{15}$$

$$\sigma_{SQ} = 1.28255\alpha_i = (1.2855)(0.084S_{QK}) = 0.1077S_{QK} \tag{16}$$

The data presented in Table 2 indicate that κ_{SQ} increases with time as a result of the increased live load effect. The coefficient of variation σ_{SQ} and the standard value of live load effect S_{QK} do not change with time because the properties of the live load effect model do not change with time.

Development of Critical Life Curves for Flexural Beams

The flexural resistance reduction coefficients of reinforced concrete beams corresponding to the required lowest reliability index of the beams at a given year of service are calculated in order to develop

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Table 3. Partial Safety Factors for Flexural Resistance (Moment) Corresponding to the Lowest Reliability Index and for Various Service Years

Vehicle operation state	ρ	γ_R^2	Assumed years in service 10	20	30	40	50	60	70	80	90	100
Normal operation state	0.1	1.2297	1.1234	1.1293	1.1327	1.1352	1.1371	1.1387	1.1400	1.1411	1.1421	1.1430
	0.25	1.1644	1.0396	1.0523	1.0597	1.0650	1.0691	1.0724	1.0753	1.0777	1.0799	1.0818
	0.5	1.1020	0.9529	0.9731	0.9849	0.9933	0.9999	1.0052	1.0097	1.0137	1.0171	1.0202
	1.0	1.0650	0.8814	0.9087	0.9248	0.9362	0.9451	0.9524	0.9585	0.9639	0.9686	0.9729
	1.5	1.0606	0.8558	0.8866	0.9048	0.9177	0.9278	0.9361	0.9430	0.9491	0.9545	0.9593
	2.5	1.0646	0.8357	0.8704	0.8909	0.9054	0.9168	0.9261	0.9339	0.9408	0.9468	0.9522
Intensive operation state	0.1	1.2419	1.1419	1.1457	1.1479	1.1495	1.1507	1.1517	1.1525	1.1532	1.1539	1.1544
	0.25	1.1875	1.0770	1.0851	1.0898	1.0932	1.0959	1.0980	1.0999	1.1014	1.1028	1.1041
	0.5	1.1278	1.0040	1.0173	1.0250	1.0306	1.0349	1.0384	1.0413	1.0439	1.0462	1.0482
	1.0	1.0675	0.9260	0.9450	0.9561	0.9641	0.9702	0.9752	0.9795	0.9832	0.9864	0.9893
	1.5	1.0413	0.8883	0.9101	0.9229	0.9321	0.9392	0.9450	0.9499	0.9542	0.9579	0.9612
	2.5	1.0212	0.8540	0.8787	0.8931	0.9035	0.9115	0.9181	0.9236	0.9284	0.9327	0.9366

Note: γ_R^2 = partial resistant factor corresponding to the target reliability index for the design life (here, 100 years) with $\beta_0 = 4.2$. A reliability index $\beta = \beta_0 - 0.5 = 3.7$ was used in the calculations; $\rho = S_{QK}/S_{GK}$, the ratio of average live load effects/average dead load effects.

Table 4. Limit Values of Relative Resistance Reduction Coefficients Corresponding to the Lowest Reliability Index for Various Service Years

Vehicle operation state	ρ	Assumed years in service 10	20	30	40	50	60	70	80	90	100
Normal operation state	0.1	0.914	0.918	0.921	0.923	0.925	0.926	0.927	0.928	0.929	0.929
	0.25	0.893	0.904	0.910	0.915	0.918	0.921	0.923	0.926	0.927	0.929
	0.5	0.865	0.883	0.894	0.901	0.907	0.912	0.916	0.920	0.923	0.926
	1.0	0.828	0.853	0.868	0.879	0.887	0.894	0.900	0.905	0.909	0.914
	1.5	0.807	0.836	0.853	0.865	0.875	0.883	0.889	0.895	0.900	0.904
	2.5	0.785	0.818	0.837	0.850	0.861	0.870	0.877	0.884	0.889	0.894
Intensive operation state	0.1	0.919	0.923	0.924	0.926	0.927	0.927	0.928	0.929	0.929	0.930
	0.25	0.907	0.914	0.918	0.921	0.923	0.925	0.926	0.927	0.929	0.930
	0.5	0.890	0.902	0.909	0.914	0.918	0.921	0.923	0.926	0.928	0.929
	1.0	0.867	0.885	0.896	0.903	0.909	0.914	0.918	0.921	0.924	0.927
	1.5	0.853	0.874	0.886	0.895	0.902	0.908	0.912	0.916	0.920	0.923
	2.5	0.836	0.860	0.875	0.885	0.893	0.899	0.904	0.909	0.913	0.917

resistance critical life curves. The lowest reliability index is then represented by its corresponding resistance (loading capacity) relative reduction coefficient. According to China's load-capacity grading standard (Lv 2006; Wang and Li 2010), resistance partial factors corresponding to the lowest reliability index can be calculated. Resistance reduction coefficients are then obtained by the ratio of the calculated resistance partial factor to the target resistance partial factor defined in the design code GB/T 50283 (AQSIQ and MOHURD 1999). Partial resistance factors for flexural reinforced concrete beams are calculated for various service years and listed in Table 3 based on the lowest reliability index equal to 3.7. The limit state of the structure is taken as [GB/T 50283 (AQSIQ and MOHURD 1999)]

$$\gamma_G S_{GK} + \gamma_Q S_{QK} = R_K/\gamma_R \tag{17}$$

where S_{GK}, S_{QK}, and R_K = dead load effect, live load effect, and the resistance of the structure, respectively, and γ_G, γ_Q, γ_R are corresponding adjustment factors for them.

The limit values of the relative resistance reduction coefficients based on the partial safety factors for resistance from Table 3 are presented in Table 4. The relative resistance reduction coefficients are obtained by dividing the partial safety factors by γ_R.

Resistance reduction coefficient limit curves can be obtained for different vehicle operational states and ρ according to Table 4. These curves are defined as the structure's critical life curves. The actual resistance reduction coefficients can be calculated according to traffic and environment conditions and structural properties. The intersection of the actual resistance reduction curve and the critical life curve defines the structure's total service life. Because the critical life curves are derived from the lowest reliability index, the residual life prediction based on critical life curves is equivalent to that based on time-varying reliability indexes.

Case Study: Assessment of a Simply Supported Bridge

A highway bridge has been considered in this case study. Each bridge consists of several simply supported spans; each of them is 13 m long and 11 m wide (Fig. 1). Each span consists of 11 single hollow-core slab beams with the cross section shown in Fig. 2.

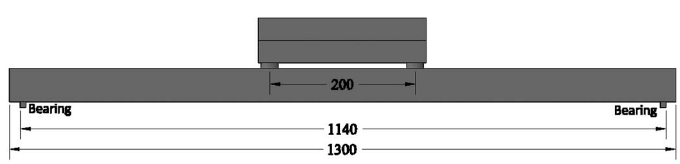

Fig. 1. Elevation of the studied bridge (in millimeters)

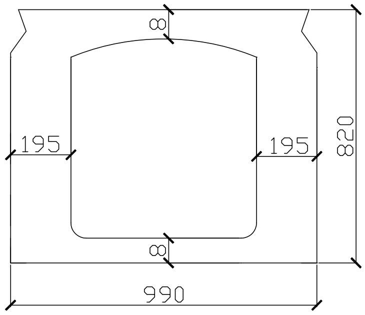

Fig. 2. Single hollow-core slab beam cross section (in millimeters)

The design live loads used are QC-20 and G-120 for cars and trucks, respectively, according to China's bridge design code in the 1990s when the bridge was originally designed, as shown in Figs. 3 and 4 (Department of Transport 1989). The design service life T is 100 years. The bridge had been in service for 11 years at the time of the assessment. According to the condition survey, the chloride content of the beams exceeded the threshold values. The beams experienced serious cracking and spalling, and the main longitudinal reinforcement was exposed to the atmosphere, as shown in Figs. 5 and 6. The residual life of the flexural beams of this bridge is calculated based on the safety requirements for Grade 1 highway bridges [GB/T 50283 (AQSIQ and MOHURD 1999)].

Load Effects

According to the current bridge design code, the Grade 1 highway load ($q_k = 10.5$ kN/m) was used. The No. 2 beam of the bridge received most of the load with a transverse distribution factor $m_{cq} = 0.2520$. With this load distribution, the midspan bending moment of dead load $M_g = 276.7$ kN · m, and the mid-span bending moment of live load $M_q = 302.0$ kN · m. The sum of the factored dead and live load bending moment $M_j = 754.8$ kN · m.

Resistance Attenuation Model

According to the design documentation and field inspection, the concrete has a nominal compressive strength f_{ck} of 18.74 MPa. There are 11 longitudinal steel bars (Type HRB335, whose diameter is 22 mm) with an area of 41.811 cm^2. The tensile yield

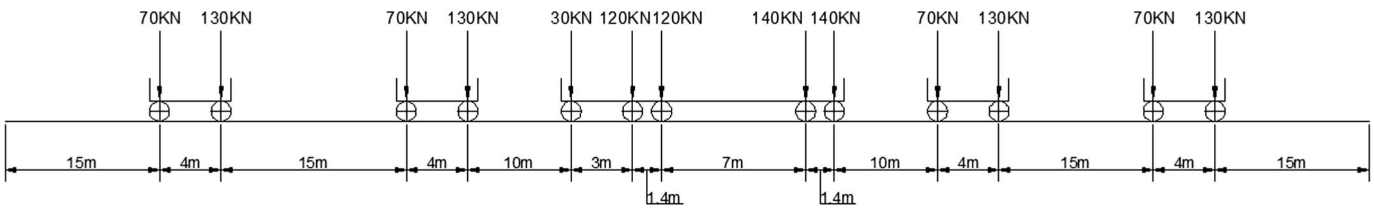

Fig. 3. Model of QC-20 live load (adapted from Department of Transport 1989)

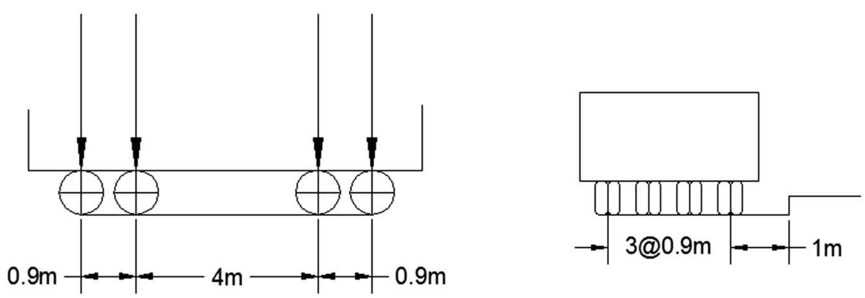

Fig. 4. Model of G-120 live load (adapted from Department of Transport 1989)

Fig. 5. Spalling of the outside beam (images by authors)

Fig. 6. Severe corrosion of reinforcements (images by authors)

Fig. 7. Longitudinal reinforcement samples

strength f_y of the Type HRB335 steel bars is 335 MPa, and the concrete cover thickness is 3.7 cm. With these material properties and the geometric configurations shown before, the original beam flexural resistance R_0 was equal to $1043.9\ \text{kN} \cdot \text{m}$.

The field condition survey indicated that 2 of the total 11 reinforcements had almost lost all their effective cross sections as a result of severe corrosion. The remaining 9 reinforcements had no obvious corrosion, as shown in Fig. 7. Based on strength tests of the 9 reinforcements, the yield and ultimate strengths were found to be 366.0 and 542.0 MPa, respectively, which indicated that the reinforcements had not lost any tensile strength. Therefore, the effective area of the reinforcements was reduced to 9/11 of their original value. The chloride concentration near the surface of the reinforcements reached 0.028% of the total weight of concrete and 0.15% of the weight of cement (Table 5), which exceeds the code limit. At the same time, the chloride concentration did not change along the depth of the concrete cover, indicating that the chloride ion existed at the beginning of the construction. The equivalent diameter of the 11 reinforcements before corrosion was 7.296 cm. After 11 years in service, the equivalent diameter decreased to 6.6 cm. The annual diameter reduction percentage ξ based on the measurement at the end of the 11th year was calculated as $(72.96 - 66.00\ \text{mm})/(72.96\ \text{mm} \cdot 11\ \text{years}) = 0.8677\%$. The resistance attenuation model can be derived based on this corrosion rate according to Eqs. (3) and (7), assuming the same rate of corrosion for the remaining life:

$$R_k(t) = (1 - 0.017354t + 0.0000753t^2)R_0 \tag{18}$$

where $R_k(t)$ = resistance at time t (in years). The resistant bending moment at various years in service and their corresponding reduction coefficients are calculated using Eq. (17) with $R_0 = 1043.9\ \text{kN} \cdot \text{m}$. The results are shown in Table 6, where S is the combination of the dead and live load effects.

Prediction of Structural Residual Life

The structural residual life prediction curves can be constructed by using the data in Table 7, as shown in Fig. 8. The figure shows that the predicted reduction coefficient curve intersects the minimum requirement curve at $t = 16.26$ years, which is the end of the service life. Thus, the residual life $T_r = t - t_0 = 16.26 - 11 = 5.26$ years, where 11 is the years the bridge had already been in service.

Table 5. Chloride Content in the Slab

Location	Sample depth (cm)	Free chloride content in the concrete (%)	Free chloride content in the cement (%)	Total chloride content in the concrete (%)	Total chloride content in the cement (%)
Midspan of No. 4 beam (bottom)	0–1	0.045	0.25	0.054	0.29
	1–2	0.031	0.17	0.071	0.39
	2–3	0.024	0.13	0.068	0.37
	3–4	0.037	0.20	0.064	0.35
	4–5	0.028	0.15	0.048	0.26
	5–6	0.028	0.15	0.052	0.28
Near the supporting end of No. 4 beam	0–1	0.058	0.32	0.08	0.44
	1–2	0.044	0.24	0.071	0.39
	2–3	0.033	0.18	0.06	0.33
	3–4	0.035	0.19	0.071	0.39
	4–5	0.038	0.21	0.061	0.33
	5–6	0.029	0.16	0.046	0.25
Midspan of No. 4 beam (on the side)	0–1	0.038	0.21	0.049	0.27
	1–2	0.039	0.21	0.073	0.40
	2–3	0.031	0.17	0.064	0.35
	3–4	0.038	0.21	0.065	0.35
	4–5	0.036	0.20	0.063	0.34
	5–6	0.028	0.15	0.046	0.25

Note: Components of concrete are water:cement:sands:coarse aggregates = 0.42:1:1.291:2.743.

Table 6. Resistance Reduction Coefficients for Various Years in Service

Items	Assumed years in service										
	0	10	20	30	40	50	60	70	80	90	100
$R_k(t)$ (kN · m)	1,043.9	870.6	713.0	571.2	445.0	334.6	239.9	161.0	97.7	50.2	18.4
S (kN · m)	754.8	754.8	754.8	754.8	754.8	754.8	754.8	754.8	754.8	754.8	754.8
γ_R	1.0611	1.0611	1.0611	1.0611	1.0611	1.0611	1.0611	1.0611	1.0611	1.0611	1.0611
$R_k(t)/\gamma_R S$	1.185	0.988	0.809	0.648	0.505	0.380	0.272	0.183	0.111	0.057	0.021

Note: γ_R = partial resistant factor by interpolation from Table 3, corresponding to $\rho = 1.0914$ (ratio of live to dead load).

Table 7. Resistance Reduction Coefficients for Various Years in Service: Predicted versus Minimum Required

Resistance reduction coefficients	Service period (years)									
	10	20	30	40	50	60	70	80	90	100
Predicted $(R/\gamma_R S)$	0.988	0.809	0.648	0.505	0.380	0.272	0.183	0.111	0.057	0.021
Minimum $[R/\gamma_R S]$	0.864	0.883	0.894	0.902	0.908	0.913	0.917	0.920	0.923	0.926

Note: Predicted $(R/\gamma_R S)$ is obtained from Table 6; $[R/\gamma_R S]$ = minimum required resistance factor corresponding to $\rho = 1.0914$, which is derived by interpolation from Table 4.

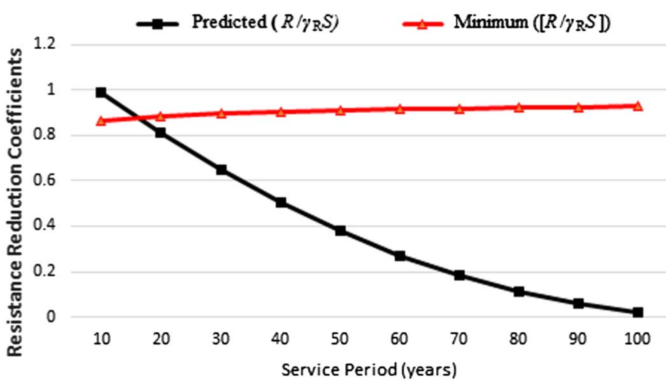

Fig. 8. Life prediction curves for the flexural capacity of the hollow slab

Fig. 9. Two-point loading testing (image by authors)

Based on the chloride ion content measurement (Table 5), the chloride ion concentration near the longitudinal reinforcements reached 0.028%, and amounted to 0.15% of the cement content, which exceeded the limit imposed by code. Furthermore, based on the fact that the chloride ion content did not show any regular change along the depth of the concrete cover, it is concluded that the chloride ion already existed during the construction process. From the visual inspection, 2 of the 11 total longitudinal reinforcements had completely lost the ability to carry any forces as a result of severe corrosion. In addition, there were large areas of cracking and spalling on the surface of the slab and a considerable number of reinforcements were exposed to the outside environment and corroded severely. In addition, it is observed that the deficiencies develop rapidly.

A complete load test was conducted on the slab that was dismantled from the bridge at the 11th year as a result of severe damage (Figs. 9 and 10). When the strain of the reinforcements at the midspan reached 985 $\mu\varepsilon$, the reinforcements were considered to yield. At the midspan, the yield moment was 883.2 kN · m and the moment when the beam broke was 955.3 kN · m (Figs. 11 and 12). Comparing the yield moment with the predicted resistance at year 10 (i.e., 870.6 kN · m from Table 6), it is concluded that the residual life prediction of the slab beam is very close to the test results. The predicted resistance was calculated using the load and resistance factor design (LRFD) method that has safety margins provided by the resistance factors designated in the design code. It is usually carried out conservatively by applying the code factors,

Fig. 10. Complete damage because of bending (image by authors)

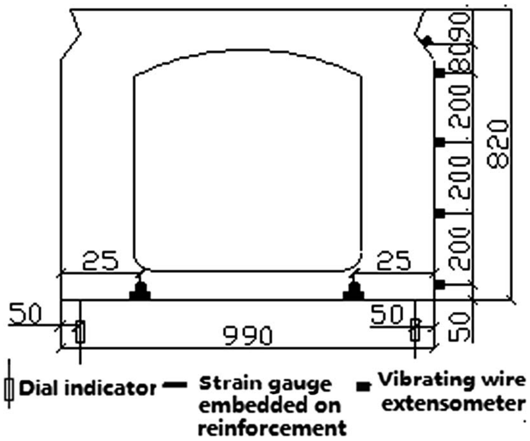

Fig. 11. Sensors layout in the midspan (in millimeters)

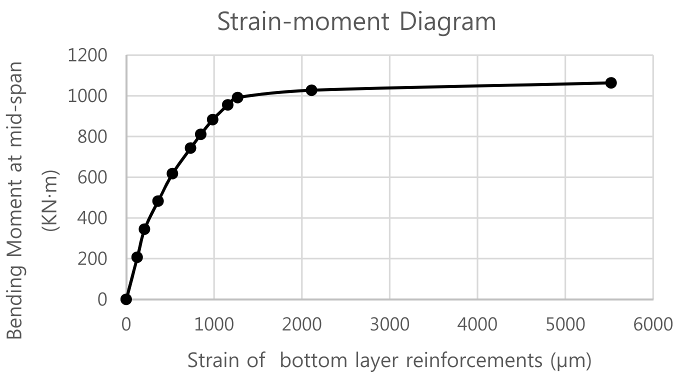

Fig. 12. Measured deflection-moment curve

which was confirmed in this destructive test (the predicted moment is a little less than the yield moment).

Summary and Conclusions

The residual life of a bridge can be predicted by studying the resistance attenuation and live load variation models based on the lower limit reliability of the bridge. This method can eliminate complex reliability calculations and analyses. The field inspection and condition survey and the lab destructive test proved the accuracy of the approach. The following conclusions are drawn from this study:

- The minimum requirement for the resistance of a bridge depends on the combination of dead and live load effects. The uncertainty of these loads is analyzed through their probability distributions. The dead load effect follows a normal distribution according to a nationwide survey on 36 bridges in China. The live load effect analysis can be simplified by using a maximum live load distribution model truncated from the whole live load distribution process, which is found to obey Type I Gumbel distribution. Partial safety factors for the resistance of bridges at various years in service can be obtained based on the combination of the dead and live load effects with a minimum reliability index defined by an applied design code, which was taken as the target reliability index β minus 0.5 in this study according to China's bridge reliability design code.
- Resistance reduction coefficients that define the minimum requirement of resistance for bridges at various years in service can be obtained by dividing the partial safety factors by the partial resistant factor corresponding to the target reliability index. The curve determined by the predicted resistance reduction coefficients is defined as the critical life curve of the bridge.
- A resistance attenuation model can be determined through the predicated reduction rate of the effective diameters of reinforcement of a reinforced concrete bridge. The reduction rate should be determined through field investigations such as the measurement of chloride contents in concrete and the effective diameters of reinforcements. A resistance attenuation curve can be plotted based on the resistance attenuation model.
- The time when the predicted critical life and the resistance attenuation curves intersect is considered the end of the bridge's life. The difference between the time in service and the end of the bridge's life is the residual life of the bridge.

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