유지관리보수가 된 교량의 내하력평가 및 잔존수명 예측

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Rating and Lifetime Prediction of a Bridge with Maintenance

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Abstract : Bridges are rated at two levels by either Load Factor Design (LFD) or Allowable Stress Design (ASD). The lower level rating is called Inventory Rating and the upper level rating is called Operating Rating. To maintain bridges effectively, there is an urgent need to assess actual bridge loading carrying capacity and to predict their remaining life from a system reliability viewpoint. The lifetime functions are introduced and explained to predict the time-dependent failure probability. The bridge studied in this paper was built 30 years ago in rural area. For this bridge, the load test and rehabilitation were conducted. The time-dependent system failure probability is predicted with or without rehabilitation. As a case study, an optional rehabilitation is suggested, and for this rehabilitation, load rating is computed and the time-dependent system failure probability is predicted. Based on rehabilitation costs and extended service lifes, the optimal rehabilitation is suggested.

초 록 : 교량은 강도설계법이나 허용응력 설계 법에 의해, 두 가지 단계에서 평가된다. 낮은 단계를 Inventory 높은 단계를 Operating이라 부른다. 교량을 효과적으로 유지관리 보수하기 위하여, 교량의 실제하중수용 능력을 평가하고 시스템 신뢰성으로부터 교량의 잔여 수명을 예측하는 것은 매우 시급하다. 생애함수가 시간 의존적 파괴확률을 예측하기 위하여 소개되고 설명된다. 이 논문에서 연구되는 교량은 30년 전 농촌지역에 시공되었다. 이 교량에 대하여 하중시험과 보강이 이루어졌다. 시간 의존적 파괴확률이 보수보강된 경우와 그렇지 않은 경우 대하여 예측되었다. 또 다른 연구로서 새로운 보수보강이 제시되고 이 보수보강에 대하여 내하력이 평가되었고 시간 의존적 파괴확률이 예측되었다. 유지관리 가격과 확장된 교량의 생애를 기본으로, 최적의 보수보강 기법이 제시되었다.

Key Words : bridges, bridge rating, system reliability, remaining life, lifetime function, optimal maintenance

1. Introduction

Bridges are rated at two levels by either Load Factor Design (LFD) or Allowable Stress Design (ASD). The lower level rating is called Inventory Rating and the upper level rating is called Operating Rating. The inventory rating implies safe use of a highway bridge on a day-to-day basis. In AASHTO's maintenance manual[2,3], the inventory rating is "the load which can safely utilize an existing structures for an indefinite period". The operating rating relates to the absolute maximum loads that may be permitted on the bridge, which can not be exceeded in any circumstance. In AASHTO's maintenance manual[2,3], the operating rating is "a maximum permissible load to which a structure may be subjected".

To maintain bridges effectively, there is an urgent need to assess actual bridge loading carrying capacity and to predict their remaining life from a system reliability viewpoint. The lifetime functions are introduced and explained to predict the time-dependent

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failure probability.

The bridge studied in this paper was built 30 years ago in rural area. The type of the bridge is slab. This bridge had a problem of overload, and there were many requests of reconstruction from people in this area because of a danger of bridge collapse. The load test was conducted to this bridge, and detailed analysis was performed. As a test result, three steel girders were reinforced. With this rehabilitation, the load rating is computed and time-dependent system failure is predicted. The optional case is studied with different rehabilitation (four steel girders are reinforced). The optimal rehabilitation is suggested by comparing total rehabilitation cost per year.

2. Current Bridge Rating

The AASHTO maintenance manual[2,3] provides for ASD and LFD load rating. The general equation of load rating for moment is shown in Eq. 1.

$$RF = \frac{M - \gamma_D M_{Dead}}{\gamma_L M_{LL(1+I)}} \qquad (1)$$

where

RF	=	Rating factor
M	=	Moment strength of the controlling member of a bridge. Computing these is different for ASD and LFD.
M_{Dead}	=	Dead load moment on the member
MLL	=	Live load demand moment with distribution factor on the member
γ_D	=	Dead load factor
γ_L	=	Live load factor
I	=	Impact factor

The live load factors and dead load factors are in Table 1 for allowable stress design (ASD) and load factor design (LFD) load rating.

Table 1. Live load and dead load factors

	ASD		LFD	
	Inventory	Operating	Inventory	Operating
Dead Load Factors, γ_D	1.00	1.00	1.30	1.30
Live Load Factors, γ_L	1.00	1.00	2.17	1.30

Allowable stresses of each material (steel, concrete, timber, etc.) specified in the maintenance manual[2,3] for two rating levels, inventory and operating are used for rating computation. The inventory and operating strengths are computed by using these allowable stresses.

The dead load effects of the structure are computed based on the conditions existing at the time of analysis. When the dead loads are calculated, the unit weights of materials, which are specified in current AASHTO specification[4], are used.

The typical live load for bridge rating is either the standard HS20 truck or HS20 lane loading as defined in the AASHTO specification[4]. The live load that produces the larger bending moment is used. In order to calculate the moment in a girder, the moment calculated by HS20 truck or HS20 lane load is multiplied by the wheel-load distribution factor for the girder. To account the dynamic effect of moving load, impact factor in AASHTO specification[4] is used.

LFD load ratings follow the strength design provisions in the AASHTO design specification[4]. The moment strength of steel bridge is summarized in Table 2.

The computation of dead load and live load is the same as that of ASD.

After moment strength, dead load moment demand and live load moment demand are computed, the rating factor is calculated by using Eq. 1. Eq. 1 is used for both ASD and LDF.

Table 2. Moment strength of steel

Type of cross section	Moment strength(M)
Compact, braced, and non-composite	$f_y \times Z_s$
Compact and composite	Plastic strength of composite section
Non-compact, braced, and non-composite	$f_y \times S_s$
Non-compact and composite	Yield strength of composite section ($f_y \times S_{comp}$)
Un-braced and non-composite	Lateral torsional buckling strength

Where

Z_s = Plastic section modulus of steel girder

S_s = Elastic section modulus of steel girder

S_{comp} = Elastic section modulus of composite section

f_y = Steel yield stress

3. Prediction of the time-dependent failure probability

Most engineering systems are designed to be used over some period of time. The lifetime of some systems is short or long. But most systems are intended for use over a much longer period. It is very important to predict the condition of the system in the future. There are several lifetime distributions which describe the evolution of the risk of components. In this section, the lifetime function is used as a tool to forecast the time-dependent failure probability of the system using system reliability concept.

3.1. Condition State of Components and Systems

Structure function and Reliability function[5] are useful tools to describe the state of a system of n components. Structure function defines the system state as a function of the component state. The structure function has two values as follows

$$\phi(x) = \begin{cases} 0 & \text{if the system has failed} \\ 1 & \text{if the system is functioning} \end{cases} \tag{2}$$

where
x = System state vector; $\{\chi_1, \chi_2 \cdots, \chi_n\}$
x_i = Component i state; 0 = component has failed, 1= component is functioning

As an example to obtain the structure function, a four-component system shown in Fig. 1 is used.

The first reduction step is a parallel system between components 3 and 4. In the parallel system, the failure of all components causes the system failure. The first subsystem can be expressed as follows

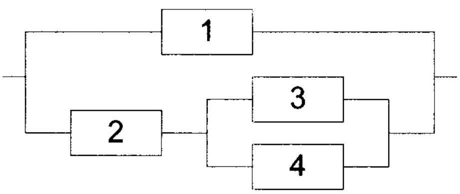

Fig. 1. Four-component system

$$\phi_{s1}(x) = 1-(1-\chi_3)(1-\chi_4) \tag{3}$$

The second reduction step is a series system between component 2 and subsystem 1. In the series system, any one component failure in a system causes the system failure. The second reduction can be expressed as follows

$$\phi_{s2}(x) = \chi_2[1-(1-\chi_3)(1-\chi_4)] \tag{4}$$

The last reduction step is a parallel system between component 1 and subsystem 2 produced by Eq. (4) and the last reduction produces the structure function of the system.

$$\phi_s(x) = 1-(1-\chi_1)\{1-\chi_2[1-(1-\chi_3)(1-\chi_4)]\} \tag{5}$$

The structure function is deterministic. This assumption may be unrealistic for certain types of components or a system. So, reliability function[5], r(p), is necessary to model the structures. χ_i was defined to be the deterministic state of component i in the structure function. In the reliability function, χ_i is a random variable. The probability that component i is functioning is given by

$$P_i = P[\chi_i = 1] \tag{6}$$

Where
P_i = Probability that component i is functioning

If there are n components, the reliability vector of a system can be written as follows

$$p = \{P_1, P_2, \cdots P_n\} \tag{7}$$

The system reliability is defined

$$r(p)=P[\phi(x)=1] \tag{8}$$

The reliability function gives the probability that the system is functioning. In order to obtain the reliability function for a four-component system shown in Fig. 1, the same procedure is necessary. However, the component reliability function is used in each step instead of component state x.

3.2. Type of Lifetime Functions

There are several lifetime functions to describe the evolution of the probability of failure. In this paper, survivor function is introduced and explained. The survivor function can be applied to both discrete and continuous lifetime.

The survivor function is the generalization of reliability because the survivor function gives the reliability that a component is functioning at one particular time. The survivor function is expressed

$$S(t) = P[T \geq t] \quad t \geq 0 \tag{9}$$

It is assumed that when $t \geq 0$, $S(t)$ is one. The survivor function has to satisfy three conditions. These are

1) $S(0) = 1$
2) $\lim_{t \to \infty} S(t) = 0$
3) $S(t)$is non-increasing without any maintenance

Several distributions are used as survivor functions. The exponential distribution, Weibull distribution, log-logistic distribution, and exponential power distribution are introduced in this paper. These survivor functions are shown in Table 3.

Table 3. Survivor function

Distribution	Survivor function
Exponential	$\exp(-\lambda t)$
Weibull	$\exp(-(\lambda_s t)^\kappa)$
Log-logistic	$\frac{1}{1+(\lambda_s t)^\kappa}$
Exponential- power	$\exp(1-\exp(\lambda_S t)^\kappa)$

Where
λ = Failure rate
λ_S = Scale factor
κ = Shape factor
t = Time, $t \geq 0$

For a four-component system shown in Fig. 1, if each component has an exponential survivor function, the system reliability function is shown in Eq. 10.

$$r(p)=1-(1-e^{-\lambda t})\{1-e^{-\lambda t}[1-(1-e^{-\lambda t})(1-e^{-\lambda t})]\} \tag{10}$$

3.3. Lifetime Parameters

In order to apply the proposed methodology to an existing bridge, the parameters of lifetime function for each bridge component are necessary from the data analysis. It is very important to find out which distribution type can be used for each component (deck, steel girder, and concrete girder .etc) of a bridge, and how to obtain parameters of lifetime distribution functions is an important subject.

The report, "Serviceable Life of Highway Structures and their Components[6]", summarizes the work done by Maunsell Ltd. for the Highways Agency. This report presents the parameters of the lifetime function for each bridge component. In Maunsell[6], the serviceable life is defined to be the time taken for a significant defect requiring attention to be recorded at an inspection. According to defect severity, four levels are classified.

- Severity1 : no significant defects
- Severity2 : minor defects of a non urgent nature
- Severity3 : defects which shall be included for attention within the next annual maintenance program
- Severity4 : the defect is severe and urgent action is needed

Therefore, the serviceable life from this report is defined to be the time taken for a structural defect to be recorded for attention in next annual year or the time taken for a structural defect to be needed for urgent action.

In the state-of-the-art Rilem 14 on the Durability of Concrete Structures[7] the RILEM Technical Committee states that the probability density of service life generally pecks rapidly before decreasing gently towards zero when approaching an infinitely long service life. This type of distribution must be selected.

Table 5. Parameters of weibull distributions of serviceable life for severity 4 defects(adapted from maunsell 1999)

Category	Description	x	$1/\lambda_s$	MODE
Structure forms	Years			
A1	Arches, concrete	Insufficient Data for Analysis		
A2	Slab decks	2.37	130.50	103
A3A	RC beam and Slab, slabs	3.76	83.36	76
A3B	RC beams and Slab, Beams	1.66	228.50	119
A4A	Composite, slabs	2.91	98.98	85
A4B	Composite, beams	2.86	94.70	81
A5A	Pretensioned slabs	1.90	223.39	119
A5B	Pretensioned beams	3.19	80.23	71
A6A	Post tensioned, slabs	3.03	104.20	91
A6B	Post tensioned, beams	2.60	100.83	83

Table 4. Parameters of weibull distributions of serviceable life for severity 3 defects(adapted from maunsell 1999)

Category	Description	x	$1/\lambda_s$	MODE
Structure forms	Years			
A1	Arches, concrete	2.44	41.28	33
A2	Slab decks	1.40	56.12	22
A3A	RC beam and Slab, slabs	2.98	27.73	24
A3B	RC beams and Slab, Beams	2.88	30.26	26
A4A	Composite, slabs	2.84	37.72	32
A4B	Composite, beams	1.47	26.66	12
A5A	Pretensioned slabs	1.70	68.74	40
A5B	Pretensioned beams	1.41	69.45	28
A6A	Post tensioned, slabs	2.62	51.55	42
A6B	Post tensioned, beams	3.29	23.97	21

Weibull distribution was selected as best fit of the data and summarized in the report[5]. Tables 4 and 5 contain the parameters of Weibull distribution for Severity 3 and 4, respectively.

In application in next section, the severity 4 defect is used to predict the time-dependent system probability.

4. Rating and Prediction of the time-dependent failure probability of a bridge with maintenance.

4.1. Introduction

The bridge studied in this paper was built 30 years ago in rural area. The type of the bridge was slab. This bridge had a problem of overload, and there were many requests of reconstruction from people in this area because of a danger of bridge collapse. The load test was conducted to this bridge, and detailed analysis was performed. As a result of load test, the rehabilitation was decided instead of a reconstruction. Three steel girders were attached to the slab bridge with a uniform girder span.

4.2. Bridge Description

Fig. 2 shows the profile and plain view of the bridge. The bridge has not been significantly rehabilitated since it was constructed (early 1970s). The bridge has 7.2 m simple span and a type is a slab bridge.

Reinforcement concrete substructures support the bridge with fixed bearing at two ends. Field measurements were performed to obtain necessary information for bridge load rating.

The bridge is a typical slab bridge for transporting rural machinery and light weighted truck for agricultural products. Since, most of rural bridge, now, is used to connect transportation network from rural area to urban or center of regional industry, there are frequently unexpected heavy trucks, even though they are legal load based on the current bridge specification.

Material properties of concrete bridges have deteriorated and neutralized since they was built. In order to know material properties of the existing concrete bridges, there are two types of experiment. The first one is Schmidt hammer and the second one is a compressive test conducted on 100 mm diameter core removed from the slab deck. The measured average compressive strength of slab and pier obtained by ASTM standard, were 150 ~ 160 kgf/cm^2 and 216

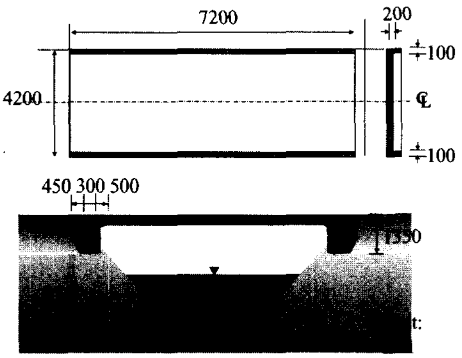

Fig. 2. Plain view and profile of the bridge

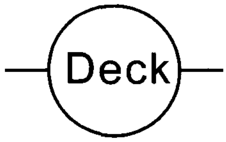

Fig. 3. System failure model before rehabilitation

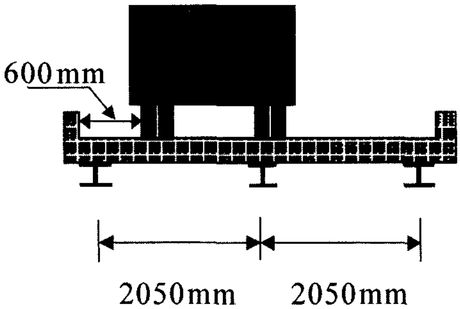

Fig. 4. Cross section of the bridge after rehabilitation

gf/cm², respectively. Allowable stress of reinforcing bar of the slab was assumed 1,500*kgf/cm²* based on Korea Bridge Design specification(1) because the bridge was constructed in 1972.

4.3. System Reliability Model

In order to compute the system reliability, it is necessary to assume the models which describe the behavior of the bridge system. In modeling this slab bridge, the following bridge system is assumed.

4.3.1 Before Rehabilitation

Before rehabilitation, since the slab failure causes the bridge failure, the bridge failure model is as follows

4.3.2. After Rehabilitation

After rehabilitation, the cross section of the bridge is shown in Fig. 4.

As a system failure model, it is assumed that the failure of any one composite girder cause the bridge failure. The system failure model is shown in Fig. 5.

4.3.3. Optional Case Study

Although three girders were reinforced in real rehabilitation, another analysis is conducted with an assumption that four steel girders are reinforced. The cross section is shown in Fig. 6.

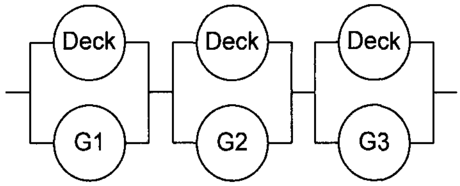

Fig. 5. System failure model after rehabilitation

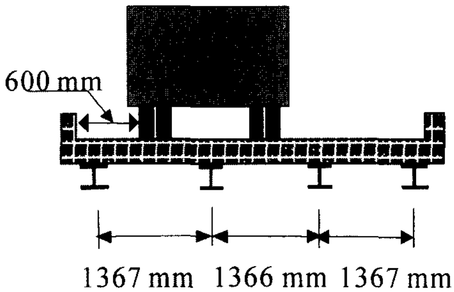

Fig. 6. Cross section of the bridge with an optional rehabilitation

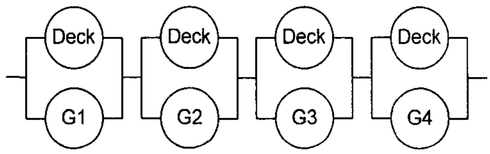

Fig. 7. System failure model with an optional rehabilitation

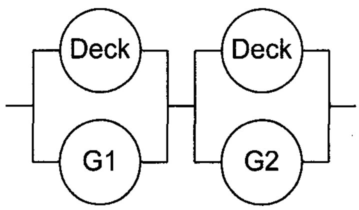

Fig. 8. Reduced system failure model with an optional rehabilitation

As a system failure model, it is assumed that the failure of any one composite girder cause the bridge failure. The system failure model is shown in Fig. 7.

The system model shown in Fig 7 can be reduced to the system model shown in Fig. 8 with considering the symmetry.

4.4. Rating and Prediction of the Time-Dependent Failure Probability

Based on the AASHTO's manual (AASHTO 1994), rating factors are computed. The rating factors are shown in Table 6.

Table 6. Rating factors

ASD	Before Rehabilitation	Current Rehabilitation		Optional Rehabilitation	
		Internal Girder	External Girder	Internal Girder	External Girder
Inventory	0.22	0.78	0.84	1.52	1.04
Operating	0.66	1.38	1.62	2.40	1.88

The rating factors were obtained based on the section 2. Since DB 13.5 was a design truck of the bridge, the bridge was rated by using the same design truck. A static load-displacement was 6.4 mm. Rating values of the slab bridge are 0.22 for inventory level and 0.66 for operating level using ASD. Since the rating values are less than one, it can be assumed that the bridge cannot support the design load, and the maximum load is also smaller that design load because the operating rating value is less than one. The current rehabilitation (3 girders are added) increases the load carrying capacity of the bridge from 0.66 to 1.38 at operating level. For an optional case (four girders are added), the load carrying capacity of the bridge also increases from 0.66 to 1.88 at operating level. Using the lifetime parameters shown in Table 5, the time-dependent failure probabilities are predicted. The results are shown in Fig. 9.

The horizontal axis is time and the vertical axis is failure probability of the system.

From this figure, it can be seen that if there is no rehabilitation, the probability of failure of the bridge is increased with time. Both an current rehabilitation and an optional rehabilitation decrease the failure probability of the system at the time the rehabilitation is performed.

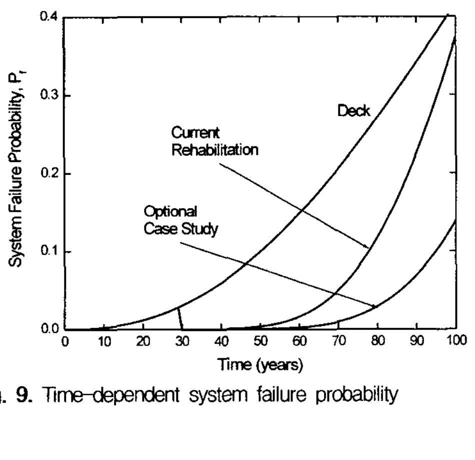

Fig. 9. Time-dependent system failure probability

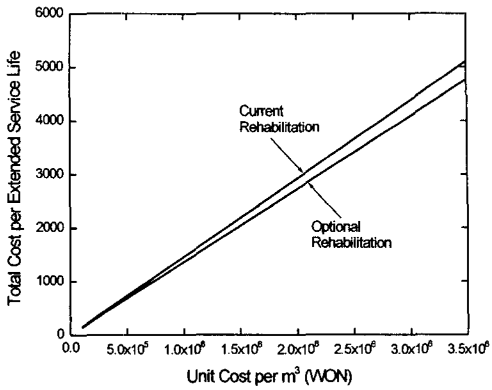

Fig. 10. Total cost per extended service life

4.5. Comparison of Maintenance Cost and Effectiveness

The costs of rehabilitation are computed with several unit costs of the steel. The results are shown in Fig. 10. In order to choose an optimal rehabilitation between current rehabilitation and optional case of rehabilitation, the extended service life is computed for both rehabilitations. Because the rehabilitation was performed when the failure probability of the deck reached 0.03055 without any rehabilitation, the target failure probability is assumed as 0.03055. It is also assumed that when the system failure probability reaches 0.03055 again after rehabilitation, rehabilitations are needed. For both cases (current and optional), the rehabilitation is performed at the same time. The extended service life of current rehabilitation is 35 years and that of the optional case is 50 years. The total rehabilitation cost is divided by an extended service life to obtain the optimal rehabilitation. The optimal rehabilitation can be chosen comparing the total cost per extended service life.

From this figure, it can be seen that when the unit cost is increased, the gab of the total cost per extended service life between current and optional rehabilitation is increased. The optimal maintenance is the optional rehabilitation.

5. Conclusion

Load rating methods were explained for ASD and LFD. Lifetime functions were used to predict the time-dependent failure probability of components or system. The current rehabilitation increased the load carrying capacity of the bridge.

In this paper, for an existing bridge, the load rating was computed and the time-dependent system failure probability was predicted. Based on the total rehabilitation cost per extended service life, the optimal rehabilitation was suggested although another rehabilitation was already conducted.

In design or rehabilitation, it is really important to predict the time-dependent system failure probability to obtain the optimal design or rehabilitation.

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