교량의 내하력 평가 및 농로교의 하중시험

Load Rating of Bridges and Load Test of Agricultural Slab Bridge

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Abstract

The bridges, which were built between 20 and 30 years ago in rural area, have to support unexpected overload caused by excessive amount of transportation. For these rural bridges, repairs and replacements are needed. To avoid the high cost of rehabilitation, the bridge rating must correctly report the present load-carrying capacity. Rating engineers use Allowable Stress Design (ASD), Load Factor Design (LFD), and Load Resistance Factor Design (LRFD) to evaluate the bridge load carrying capacity. In this paper, the load rating methods are introduced, and it is illustrated how to use the load test data from literature survey. Load test is conducted to the bridge that was built 30 years ago in rural area. From load test results, new maintenance strategy is suggested instead of the bridge replacement.

Keywords : bridge rating; load carrying capacity; load test, maintenance

요 약

20, 30 년 전 시골지역에 건설된 교량들은 과도한 교통량의 증가에 따른 초과하중을 지탱해야한다. 이러한 교량들에 대해서 보수 보강이나, 교량의 교체가 필요하다. 고가의 보수 보강을 피하기 위해는, 현재 교량의 내하력을 정확히 알아야한다. 내하력 평가자들은 교량의 내하력을 평가하기 위해 허용응력법, 강도설계법, 그리고 하중저항계수법등을 사용한다. 본 연구에서는, 내하력 평가방법을 설명하고, 문헌조사를 통해 교량의 하중실험 자료의 이용에 대하여 설명한다. 그리고, 30년전에 시골지역에 건설된 교량에 대해 하중실험을 하였다. 시험자료로 부터, 교량의 교체를 대신한 새로운 보수보강 방법이 제시되었다.

핵심용어 : 교량평가, 내하력, 하중시험, 유지관리

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1. Introduction

The bridges, which were built between 20 and 30 years age in rural area, have to support unexpected overload (Ministry 2000). This overload was not considered at design. Because of this reason, new construction is considered for these bridges. However, new construction is very expensive and funds for the rehabilitation of the bridges are limited.

Generally, it can be seen that conservative design assumption results in reserve load capacity. Load rating computation has a trend that the load capacity of existing bridges is underestimated. From the literature survey of load test (Beal and Loftus1998), it is demonstrated that a load test can be used to quantify this reserve strength on the existing bridges. Because of these reserve strength, the limit weight could be increased or the posted limit is unnecessary.

The bridge studied in this paper was built 30 years ago in rural area. The type of the bridge is slab. This bridge has also a problem of overload, and there are many requests of reconstruction from people in this area because of a danger of bridge collapse. The load test is conducted to this bridge, and detailed analysis is performed. It is tried to find the way how to increases the load carrying capacity of the rural bridge without reconstruction.

2. Bridge Description

Fig. 1 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.

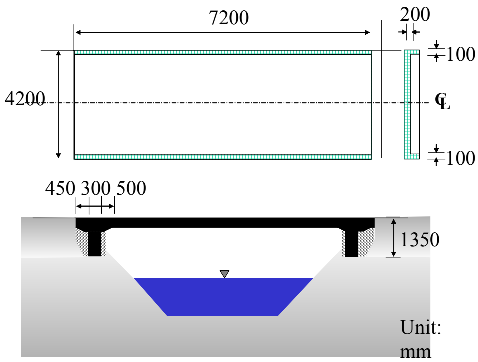

Fig. 1 Plain view and Profile of the bridge

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 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.

The measured average compressive strength of slab and pier obtained by ASTM standard, were $150 \sim 160\, kgf/cm^2$ and $216\, kgf/cm^2$, respectively. Allowable stress of reinforcing bar of the slab was assumed $1,500\, kgf/cm^2$ based on Korea Bridge Design specification (건설 교통부 1993) because the bridge was constructed in 1972.

3. Approach Method

3.1 Load Rating

In this section, the bridge load rating procedure is explained for allowable stress design rating

(ASDR) and load factor design rating (LFDR) (Manual 1983, Condition 1994). Based on the literature survey, it is illustrated how to use the load test data.

The bridges are rated by the following general equation for moment in Eq.(1).

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

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

M_{LL} = 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 used in general rating equation are in Table 1 for allowable stress design and load factor design rating.

3.2 Allowable Stress Design Load Rating Procedure

The AASHTO maintenance manual (Manual 1983, Condition 1994) provides the guideline for load rating. Fig. 2 shows a procedure for ASD load rating. Allowable stresses of each material (steel, concrete, timber, etc.) specified in the maintenance manual (Manual 1983, Condition 1994) for two rating levels, inventory and operating are used for rating computation. The inventory and operating strengths are computed by using these allowable stresses. For example, the equations of inventory and operating moment strength are in Eq.(2).

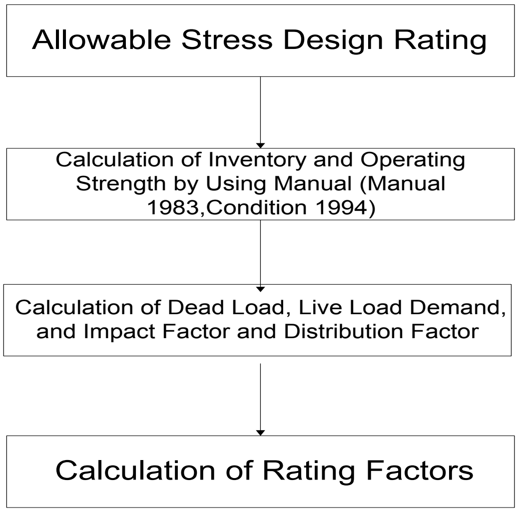

Fig. 2 ASD Rating Procedure

$$\begin{aligned} M_{inv} &= S_{non} \times f_{inv} \\ M_{ope} &= S_{non} \times f_{ope} \end{aligned} \tag{2}$$

Table 1 Live and Dead Load Factors

Type of Load Factor	ASD		LFD	
	Inventory	Operating	Inventory	Operating
Dead Load Factors,	1.00	1.00	1.30	1.30
Live Load Factors,	1.00	1.00	2.17	1.30

Where

M_{inv} = Moment strength in inventory leve

M_{ope} = Moment strength in operating level

S_{non} = Non-composite section modulus of cross section

f_{inv} = Allowable bending stress of inventory level from AASHTO Manual (Manual 1983, Condition 1994)

f_{ope} = Allowable bending stress of operating level from AASHTO Manual (Manual 1983, Condition 1994)

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 (Standard 1992), are used. For the cross section shown in Fig. 3, to calculate dead load of interior girder, its tributary width is determined using Eq.(3).

After tributary width of concrete deck is decided, with this cross section, dead load moment can be computed. The typical live load for bridge rating is either the standard HS20 truck or HS20 lane loading as defined in the AASHTO specification (Standard 1992).

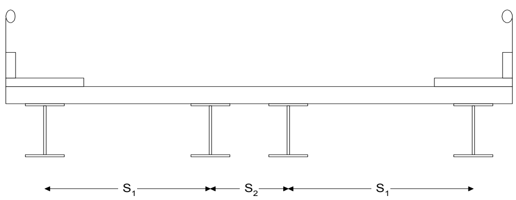

Fig. 3 Cross Section of the Bridge5

$$Tributary\ Width = \frac{S_1 + S_2}{2} \qquad (3)$$

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 (DF) for the girder. To account the dynamic effect of moving load, there is an equation for impact factor in AASHTO specification (Standard 1992) and this is in Eq.(4).

$$I = \frac{50}{125 + L} \qquad (4)$$

Where

I = Impact factor (maximum 30 %)

L = Length in feet (ft) of the portion of the span that is loaded to produce the maximum stress in the member

After moment strengths, dead load moment demand and live load moment demand are computed, the rating factors are calculated by using Eq.(1).

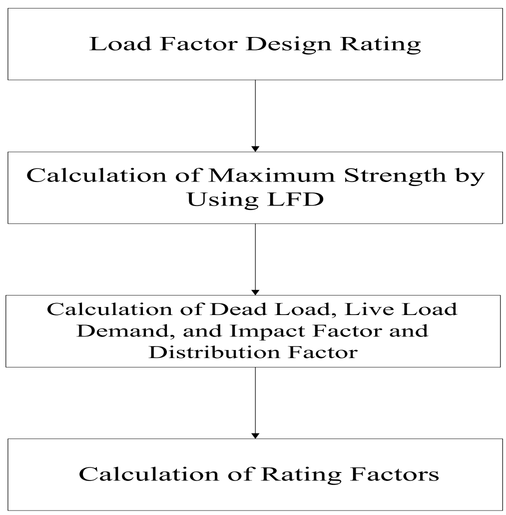

Fig. 4 LFD Rating Procedure

3.3 Load Factor Design Rating Method

LFD load ratings follow the strength design provisions in the AASHTO design specification (Standard 1992). Fig. 4 shows a procedure for LFD load rating. The moment strength of steel bridge is summarized in Table 2.

Table 2 Moment Strength of Steel

Type of cross section	Moment strength(M)
Compact, braced, and non-composite	$f_y \times Zs$
Compact and composite	Plastic strength of composite section
Non-compact, braced, and non-composite	$f_y \times Ss$
Non-compact and composite	Yield strength of composite section ($f_y \times Scomp$)
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

For reinforced concrete, moment strength is computed as the ultimate moment strength. Table 3 shows yield stresses of reinforcing steel.

Table 3 Yield Stresses for Reinforcing Steel (Adapted from Condition 1994)

Reinforcing Steel	Yield stress,fy, (psi)
Unknown steel (prior to 1954)	33,000
Structural grade	36,000
Billet or intermediate grade and unknown after 1954 (Grade 40)	40,000
Rail or hard grade (Grade 50)	50,000
Grade 60	60,000

3.4 Data Analysis For Load Rating

The test data are strains measured during the load tests. Stresses are calculated from these strains. Based on literature survey, it is illustrated how the measured stresses are used to calculate section property, distribution factor, support restriction, and impact factor.

3.4.1 Computing Distribution Factor from Test Data

In load test of Chajes et al. (1997), finite element models were calibrated using test data and were used to compute distribution factor (DF). The maximum possible girder moment ($M_{\max}$) due to multiple lane-loadings was computed by using those models. And then, the maximum wheel moment (M_{wheel}) is calculated with the same loading and idealized boundary conditions. The measured distribution factor is defined as following Eq.(5).

$$DF = \frac{M_{\max}}{M_{wheel}} \tag{5}$$

Yoo and Stalling (1993) used Eq.(6) to compute the distribution factor for wheel line load.

$$DF_i = \frac{n \cdot \varepsilon_i}{\sum_{j=1}^{k} \varepsilon_j \cdot w_j} \tag{6}$$

Where

n = Number of wheel lines of applied loading

ε_i = Bottom flange strain at the ith girder

w_j = The ratio of section modulus of the ith girder to the section modulus of a typical interior girder

k = Number of girders

For Moses et al. (1985), the distribution factor for a girder is equal to the ratio of the strain at the girder to the sum of all the bottom flange strains. The equation is in Eq.(7).

$$DF_i = \frac{\varepsilon_i}{\sum_{j=1}^{k} \varepsilon_j} \tag{7}$$

For Beal and Loftus′ load test (1988), the distribution factors from load test were calculated with maximum measured stress, section modulus (If bridge acted compositely, the section modulus is fully composite. If not, section modulus is non-composite), AASHTO distribution factor, and analytical wheel live moment demand due to load test trucks can be calculated by Eq.(8).

$$DF = \frac{\sigma_{\max} \times S}{M_{wheel}} DF_{AASHTO} \tag{8}$$

Where

$\sigma_{\max}$ = Maximum measured stress

S = Section modulus (either non-composite section or composite section)

M_{wheel} = Wheel live load moment demand due to load test truck with distribution factor

DF_{AASHTO} = AASHTO distribution factor

When Moses distribution factor is used, number of wheel line of loaded truck should multiply to Moses distribution factor. For example, when one truck is loaded on the bridge, two should multiply Moses distribution factor. The results of using both equations (Stallings and Moses) give the same result if the cross sections of interior girder and exterior girder are same. But most of bridges have different section properties for interior and exterior girders.

3.4.2 Computing Impact Factor from Test Data

From the load test results, the maximum measured strains and impact factors are used to assess bridge rating. Three researchers performed dynamic load tests to compute the impact factor based on the load test (Kissane et al. 1980, Moses et al. 1985, Yoo and Stallings 1993). Kissane et al. (1980) and Stallings and Yoo (1993) used the following Eq.(9) for impact factor. Although dynamic load tests were done in Yoo and Stallings (1993), the impact factors from the results were not used to rate the bridge. Instead, impact factors of the AASHTO were used.

$$I + 1 = \frac{\varepsilon_{dynamic}}{\varepsilon_{static}} \tag{9}$$

Where

$\varepsilon_{dynamic}$=Strain recorded due to the moving truck test

ε_{static} =Strain recorded due to the stationary truck test

Moses (et al. 1985) separated the dynamic and static components of the bridge response to moving trucks through careful inspection of dynamic strain records. On dynamic stress trace, the dynamic effects appear as vibration superposed on the static response. It can be calculated by Eq.(10).

$$I = \frac{dynamic\ amplitude}{static\ response} \tag{10}$$

3.4.3 Computing Section Properties from Test Data

In the bridges built with non-composite section, the test results usually showed that the bridges acted compositely. Most of researchers didn't try to calculate section properties by using test data. Instead, they identified the sections as either fully composite or non-composite depending on degree of unintended composite action. Chajes et al. (1997) made a finite element models and calculated the section properties by using test results. The strain gages were attached at top and bottom of flange at all girders. The test results showed that the bridge acted compositely although the bridge was built as non-composite section. So, the first task was to find out the neutral axis. The neutral axes were calculated by using measured top and bottom flange strains assuming plane strain. Plots of the location of a sections neutral axis as the truck passed over the bridge were made for each set of transducer locations. By averaging results from all plots, the neutral axis locations were determined for interior and exterior girder. For example, there is a cross section to explain the Chajes's method (Chajes et al. 1997) that was used to calculate the neutral axis.

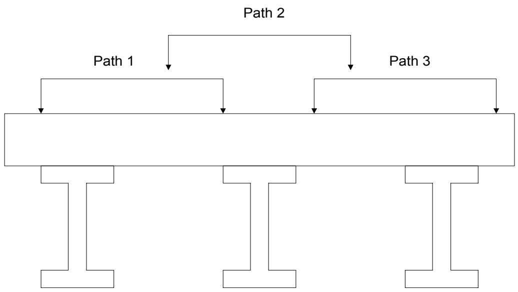

Fig. 5 Cross Section and Loading Paths

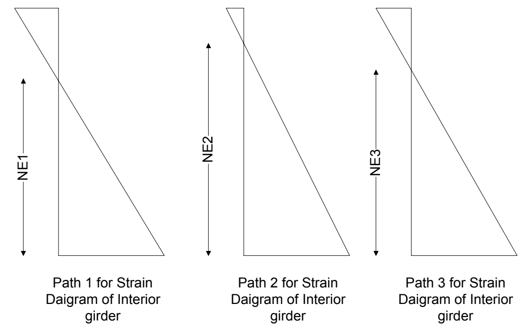

Fig. 6 Strain Diagram for Interior Girder

During three loading paths, the strain diagrams are measured for interior girder as following Fig. 6.

The neutral axis is calculated as following superposition in Eq.(11).

$$\text{Neutral Axis from Bottom Flange} = \frac{NE1 + NE2 + NE3}{3} \tag{11}$$

Where

NEi = Distance of neutral axis from bottom flange for ith path

After the neutral axis was determined, assuming that the effective width was girder spacing, the effective depth of concrete slab was determined from equilibrium of internal forces. By transforming the concrete slab to equivalent steel area, the section properties for composite section were established. Moment of inertia of test data is usually bigger than calculated one, because the asphalt is classified into dead load in calculation of rating by AASHTO method. It assumed that the asphalt doesn't resist any load. But, test result showed that the asphalt resisted the load with slab. The moment of inertia was calculated as fully composite section including asphalt thickness.

For Yoo and Stallings (1993) study, the strain gages were applied at both top and bottom of each girder. From the measured top flange strains, it was identified whether the bridge acted compositely or not. The small top flange strains indicate that the neutral axis is near the top flange as composite section. When the bridge acted compositely, the section was identified to composite section.

For Moses et al. (1985) study, the test data were used to know whether the bridges acted compositely or not. The strain gages were attached at bottom flange of each girder. By comparing computed stresses with measured stresses, it was known whether the bridges acted compositely or not.

3.4.4 Computing Support Restraint from Test Data

Although bridge is designed as a simple supported structure, many tests showed that there were resisting moments at supports. Some of researchers ignored the support restraint and some of researchers considered the support restraint to rate the bridge.

For the case of Chajes et al. (1997) load test, the bridge showed the support restraint although the bridge was simply supported. In the report, calibrated support restraints were reported. For Fu et al. (1997) case of load test, the rating value is used to find out why the test rating value was bigger than that of AASHTOs rating changing boundary conditions. Although the rating factors from AASHTO were calculated with idealized boundary conditions, in addition to this, one more computation was done. This was that one end of the bridge was simple and the other one was fixed end. The rating values from one-simple and the other one-fixed gave the similar rating values to that of test rating. Therefore, it was assumed that the bridge had support restriction.

3.4.5 Computing Rating Factor from Test Data

Although the test trucks were not HS20 trucks, most of rating results were reported for HS20 truck. This means that the rating factors were scaled to HS20 truck.

Kissane et al. (1980) scaled the rating factor of test truck to HS20 truck as following Eq.(12)

$$RF_{HS20} = \frac{f_i - f_{iDL}}{(1+I)\,\sigma_{iTruck}}\,\frac{M_{truck}}{M_{HS20}} \tag{12}$$

Where

RF_{HS20} = HS20 rating factor

σ_{iTruck} = Measured stress during the load test

f_i = Allowable bending stress

f_{iDL} = Computed dead load stress

I = Impact factor from test result

M_{truck} = Moment due to test truck

M_{HS20} = Moment due to HS20 truck

Other researchers also scaled test results to rating conditions using the ratio of bending moments (Fu et al. 1997, and Yoo and Stallings 1993).

4. Result of Rating and Reinforcement

4.1 Preliminary Rating

Since DB 13.5 was a design truck of the bridge, the bridge was rated by using the same design truck as shown in Fig. 7. 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 ASDR.

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.

Based on the test result, the bridge is reinforced with steel girders as shown in Fig. 8. Rehabilitation is performed such that the target rating operating value is at least one. Steel girders of H200-200-4-4 (KS Standard) were used to make composite bridge. The section properties are in Table 4.

4.2 Rating Calculation after Reinforcement

4.2.1 AASHTO Rating

For determining the load capacity of the bridge after rehabilitation, AASHTO inventory rating factor for ASDR is calculated by Eqs. (1), (2), (3), and Eq. (4). The girder and concrete activate as composite members because they are built by epoxy injection.

4.2.2 Load Test Rating

The bridge was loaded in two separate stages. The first load was applied very slowly. The second load was applied with high speed to consider the dynamic load effects. The vertical deflection of a center girder was measured during the load test. The result is plotted in Fig. 9. The horizontal axis is time and the vertical axis is displacement. In this figure, two displacements (deflection before rehabilitation and deflection after rehabilitation) are shown. Their results are measured by LVDT at the bottom of a center girder.

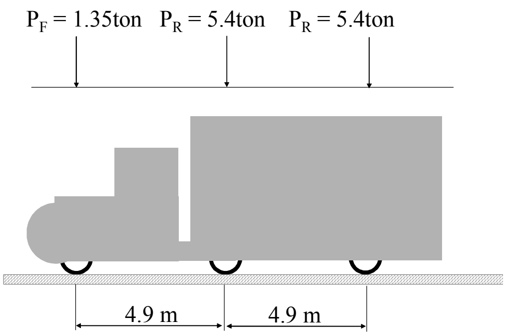

Fig. 7 Design Truck for DB 13.5

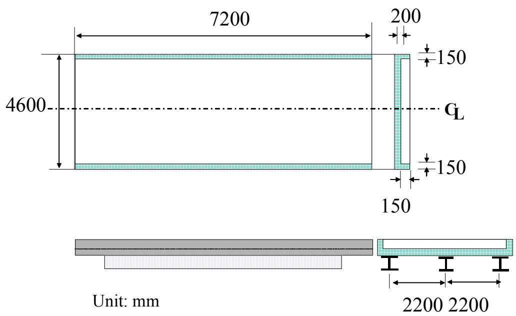

Fig. 8 Composite concrete/steel girder bridge : 3 girders reinforced and epoxy injection

Table 4 Section properties of reinforced superstructures with steel girder

Girder section area (cm^2)	Internal girder			External girder		
	Inertia moment (cm^4) (I)	Section modulus of Top (cm^3) (S_t)	Section modulus of Bottom (cm^3) (S_b)	Inertia moment (cm^4) (I)	Section modulus of Top (cm^3) (S_t)	Section modulus of Bottom (cm^3) (S_b)
23.68	16910.54	1395.63	606.48	19129.07	719.23	673.48

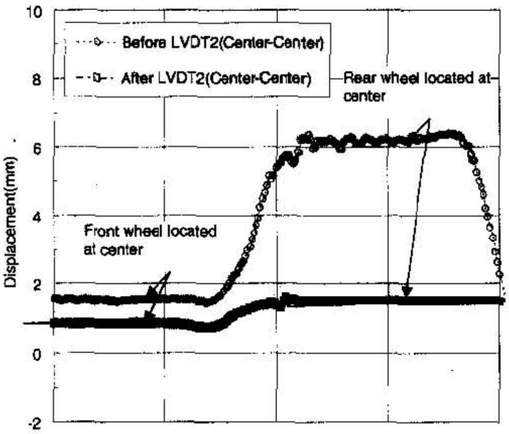

Fig. 9 Vertical deflection of centered girder

Since data of load-strain was lost, experimental rating factor cannot be calculated. Usually, rating factor from load test is bigger than that of AASHTO's because of distribution factor, impact factor, unintended support restraints, etc (Beal and Loftus 1988, Chajes et al. 1997, Fu et al. 1997, Ghosn et al. 1986, Kissane et al. 1980, Moses et. al. 1985).

Because AASHTO rating factor shown in Table 5 after rehabilitation exceeds the target operating rating value, it can be concluded that rating factor from test is bigger than the target operating rating factor.

5. Conclusions

1) Load test was performed on a bridge in rural area. Usually, the bridge types in rural area are slab and the spans are short. Based on AASHTO and test results, the rating computations were tried.
2) One of main purposes of this paper was to collect the load test data and show how to use them. In this paper, several load test data was collected and showed. By using literature survey, this paper presented how to use test data to compute distribution factor, impact factor, and neutral axis.
3) It was designed for the bridge to exceed the operating rating vales one after rehabilitation. After rehabilitation, the displacement of the bridge decreased from 6.4 mm to 1.8 mm and AASHTO operating rating increased from 0.66 to 1.39. Since rating value from test always has higher value of AASHTO (analytical value), the rehabilitation was successfully performed.

Table 5 Comparison of rating factor of slab bridge with reinforced concrete/steel girder

Type	Slab bridge	Composite concrete/steel girder bridge	
		Internal Girder	External Girder
Inventory	0.22	0.81	0.87
Operating	0.66	1.39	1.65

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