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# Load-Carrying Capacity Assessment of Deteriorated Rural Bridge

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**Abstract** □ Most of rural bridges have passed 30 years of age since they were built, which have to support unexpected overload caused by changed design load and excessive amount of transportation. For these rural bridges, repairs and replacements are needed. Even though there have been attempt to estimate the safety of existing bridges deteriorated with major defects, those approaches must rely on the observable damage and subsequent decisions are made subjectively. To avoid the high cost of rehabilitation, the bridge rating must correctly represent the present load-carrying capacity. Rating engineers use a methods such as 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 method is suggested instead of the bridge replacement.

**Keywords** □ Bridge rating, Load carrying capacity, Load test.

## I. Introduction

Most of rural bridges have passed 30 years age since they are built, which have to support unexpected overload.[12] This overload was not considered at design, because of the reason why those are caused by changed design level and truck sizes based on new specifications. new construction of these bridges is considered for the safety of regional transportation. However, new construction is very expensive and funds for the rehabilitation of the bridges are restricted within limits of provincial finance.

Generally, it can be seen that conservative design assumption results in reserve load capacity.[5] Physical load testings are desired to provide the required information and, in turn, to produce a reliable load rating. Those are referred to a diagnostic load testing and proof load testing. The relationship between the load and response can be established and can be used to confirm or deny the assumption being questioned. Especially, since a successful proof load test can demonstrate that the resistance of a bridge is greater than the proof load, it can be applied to verify the existing or increased load rating taking into account possible bridge deterioration for aging bridges. However, it is necessary for reduction of test risk to determine an appropriate intensity of proof load based on reliable load model and time-dependent reliability.[7] Otherwise the owner may not be willing to accept a capacity based on unintended composite action. Therefore, for field-testing to be successful, it is imperative that the reliable contributions to an experimentally determined load capacity rating should be removed.

Load rating computation has a trend that the load capacity of existing bridges is under -estimated. Test data[15] were used to know whether the bridges acted compositely or not. The strain gages were attached at bottom flange of each girder. It was shown whether the bridges acted compositely or not. From the Beal's load test,[6] it is demonstrated that a load test can be used to quantify this reserve strength on the existing bridges. Although bridge is designed as a simple supported structure, because of these reserve strength, the limit weight could be increased or the posted limit would be unnecessary.

In this paper rural bridge built 30 years ago was studied. 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. However, there is no appropriate rehabilitation methods to avoid excessive expenditure to rebuild bridges. The load test is conducted to validate load capacity of this bridge, and detailed analysis is performed. It is tried to propose a way how to increases the load carrying capacity of the deteriorated rural bridge with low-cost maintenance plan.

## II. Bridge Descriptions

Fig. 1 shows the profile and plain view of the bridge. The bridge has not been significantly rehabilitated since it was constructed[12] (early 1970s). The bridge has 7.2m simple span and a type is a slab bridge passed over main irrigation channel.

Reinforcement concrete substructure support the bridge with fixed bearing at two ends. Field measurements were performed to obtain nece-

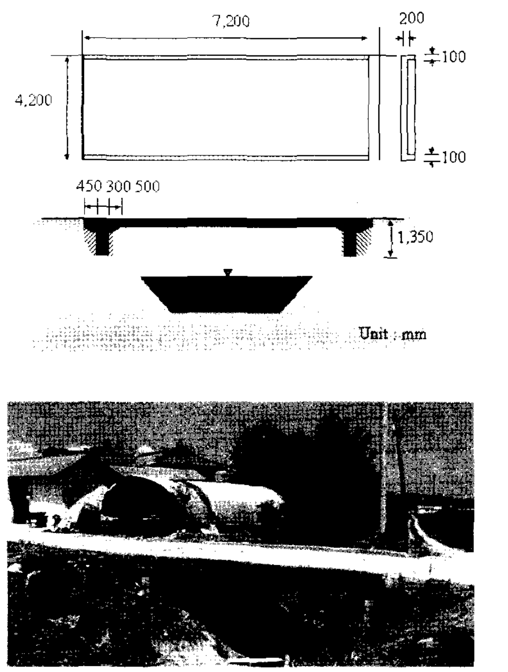



Fig. 1 Plain view and profile of the rural bridge (photo by Kim[12])

ssary information for bridge load rating. The bridge is a typical slab bridge for transporting agricultural 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 type 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 100mm diameter core removed from the slab deck. The measured average compressive strength of slab and pier obtained were 150 $kgf/cm^2$ ~ 160 $kgf/cm^2$ and 216 $kgf/cm^2$, respectively.[12] Allowable stress of reinforcing bar of the slab was assumed 1,500 $kgf/cm^2$ based on Korea Bridge Design specification[14] because the bridge was constructed in 1972.

## III. Approach Method

### 1. Load Rating Procedure Using Allowable Stress Design

In this section, the bridge load rating procedure is explained for allowable stress design rating (ASDR).[2),4)] It is illustrated how to use the load test data that is used to verify load carrying capacity of rural bridges.

The bridges are rated by the following general equation[4)] for moment in Eq. (1).

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

Where, RF = Rating factor, M = The moment strength of the controlling member of a bridge. Computing these is different for ASD (Allowable Stress Design) and LFD (Load Factor Design). $M_{DL}$ = The dead load moment on the member. $M_{LL}$ = The live load demand for moment with distribution factor on the member. I = Impact Factor. $\gamma_D$ = Dead load factor. $\gamma_L$ = Live load 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)]

Table 1 Live and Dead Load Factors

| Type of Load Factor | ASD | | LFD | |
|---|---|---|---|---|
| | Inventory | Operating | Inventory | Operating |
| Dead Load Factors( $\gamma_D$ ) | 1.00 | 1.00 | 1.30 | 1.30 |
| Live Load Factors( $\gamma_L$ ) | 1.00 | 1.00 | 2.17 | 1.30 |

The AASHTO maintenance manual[2),4)] provides the guideline for load rating procedures. Allowable stresses of each material (steel, concrete, timber, etc.) specified in the maintenance manual[2),4)] for two rating levels such as inventory and operating are used for rating computation. The inventory and operating strengths are computed by using these allowable stresses. For example, moment strength of inventory and operating states of members are in Eq. (2).

$$\begin{bmatrix} M_{inv} = S_{non} \times f_{inv} \\ M_{opr} = S_{non} \times f_{opr} \end{bmatrix} \quad (2)$$

Where $M_{inv}$ = Moment strength at inventory level, $M_{opr}$ = Moment strength at operating level, $S_{non}$ = Non-composite section modulus of cross section, $f_{inv}$= Allowable bending stress of inventory level from AASHTO Manual,[2),4)] $f_{opr}$ = Allowable bending stress of operating level from AASHTO Manual.[2),4)]

In this study load-carrying capacity assessment of rural bridges using ASD rating method was used because rating factors of other methods can be translated as shown in Fig. 2.[1)]

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,[15)] are used. The cross section of internal and external girders calculating dead load of girders are used its tributary width (TW) determined by Eq. (3).

$$TW = \frac{S_1 + S_2}{2} \tag{3}$$

Where $S_1$ is distance from external girder to internal girder, $S_2$ is distance between internal girders.

After tributary width of concrete deck is decided, with this cross section, dead load moment for effective section can be computed. The typical live load for bridge rating is either the standard HS20 truck or HS20 lane load as defined in the AASHTO specification.[3)] The live load that produces the larger bending moment is chosen. In order to calculate the moment in a girder, the moment calculated by HS20 truck or HS20 lane load should be multiplied by the wheel-load distribution factor (DF) for the girder or slab bridge.[3)] To account the dynamic effect of moving load, there is an equation for impact factor in AASHTO specification[3)] and this is in Eq. (4).

After moment strengths, dead load moment demand, and live load moment demand are computed, the rating factors can be calculated by using impact factor of Eq. (1). It is required for the bridge rating to measure a maximum strains and calculate impact factors from the load test results. Three researchers performed dynamic load tests to compute the impact factor based on

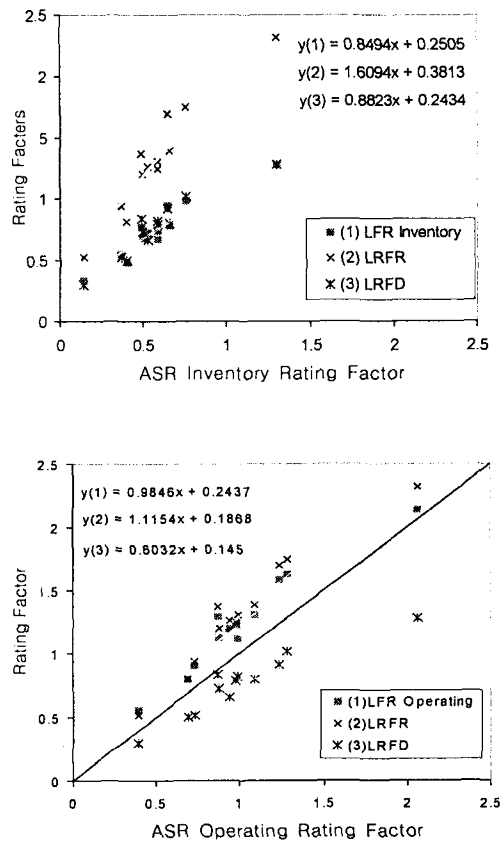



Fig. 2 Relationship between ASR inventory/operating rating factor and other factors

the load test[12),15),16)] used the ratio of a strain recorded due to the moving truck and stationary truck. Although dynamic load tests were done in Stallings and Yoo,[13),16)] the impact factors from the results were not used to rate the bridge. Instead, impact factors of the AASHTO were used.

$$I = \frac{50}{125 + L} \quad \text{or} \quad I = \frac{15}{40 + L1} \tag{4}$$

Where I = Impact factor (<= 0.3), L = Length in feet (ft) or L1 (in meter) of the portion of the span that is loaded to produce the maximum stress in the member

## 2. Data Analysis For Load Rating

Stresses are calculated from these strains that were measured during the load tests. It is illustrated how the measured stresses are used to calculate section property, distribution factor, support restriction, and impact factor.[5),7),10),11)]

*A. Computing Distribution Factor from Test Data*

Chajes *et al.*[7)] used test data to calibrate FEM model and to compute 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 the ratio of $M_{max}$ and $M_{wheel}$.

When we measured strains of all griders or components then we can calculate DF using Eq. (5). For Moses' *et al.*,[15)] 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. (5). Otherwise, in Beal and Loftus's[6)] load test, 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. (6). Yoo and Stalling[16)] computed the distribution factor, which is calculated by the ratio between bottom flange strain at the $i_{th}$ girder and sum of strain of typical interior girders.

$$DF_i = \frac{\varepsilon_i}{\sum_{j=1}^{n} \varepsilon_j} \tag{5}$$

$$DF_i = \frac{\sigma_{max} \times S}{M_{wheel}} DF_{AASHTO} \tag{6}$$

Where $\varepsilon_i$ = bottom flange strain of $i_{th}$ girder, $\sigma_{max}$ = 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[15),16)] give the same result if the cross sections of interior girder and exterior girder are same. Though most of bridges have different section properties for interior and exterior girders, most of them have more safe exterior girder section.[7] However, in this study DF is applied with value of specification because of simple and slab bridge.

*B. Computing Section Properties and Restraints Effects from Test Data*

Most of researchers didn't try to calculate section properties by using test data, since there are several unknown effects from the construction, material, and maintenance activity. Therefore, they identified the sections as either fully composite or non-composite depending on degree of unintended composite action. Chajes *et al.* made and updated a finite element models (FEM) by using test results to calculate the section properties.[7] The strain gages were attached on the 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.

It is essential for the computation of distribution factor and section modulus of real girder elements to find out the neutral axis. The neutral axes were calculated by using measured top and bottom flange strains assuming plane strain. By averaging results from all location of neutral axis as the truck passed over the bridge, the neutral axis locations were determined for interior and exterior girder, respectively. During the loading paths, the strain diagrams should be measured for interior girder. 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 for an equivalent steel area and the section properties of composite section. 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 was assumed that the asphalt does not resist any load. Since test result showed that the asphalt resisted the load with slab, the moment of inertia was calculated as fully composite section including asphalt thickness.

Because 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. For the case of Chajes *et al.*[7] load test, the bridge showed the support restraints although the bridge was simply supported, and for Fu *et al.*[9] case of load test, the rating value is used to find out why the test rating value was bigger than that of AASHTO's rating changing boundary conditions. However, some of owners or managers ignored the support restraints because they didn't want to be used with unauthorized loads.

*C. Conversion Rating Factor with Test Truck Load*

Although the test trucks were not HS20 trucks, most of rating results were reported for HS20 truck. It means that the rating factors were scaled to HS20 truck in order to assess with same criterion. Kissane *et al.*[13] scaled the rating factor of test truck to HS20 truck as following Eq. (7). Other researchers also scaled test results to rating conditions using the ratio of bending moments.[9),16]

$$RF_{HS20} = \frac{f_i - f_{iDL}}{(1 + I)\sigma_{iTruck}} \left( \frac{M_{Truck}}{M_{HS20}} \right) \quad (7)$$

where $RF_{HS20}$ = HS20 rating factor, $\sigma_{iTruck}$ = measured stress during the load test, $f_i$ = allowable bending stress, $f_{iDL}$ = computed stress owing to dead load (DL), I = impact factor from test result, $M_{Truck}$ = moment due to test truck, $M_{HS20}$ = moment due to HS 20 truck.

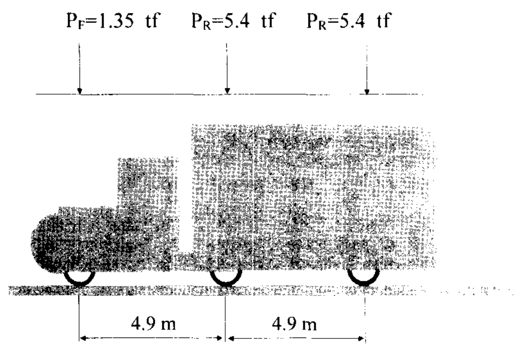



Fig. 3 Design truck for DB 13.5[12]

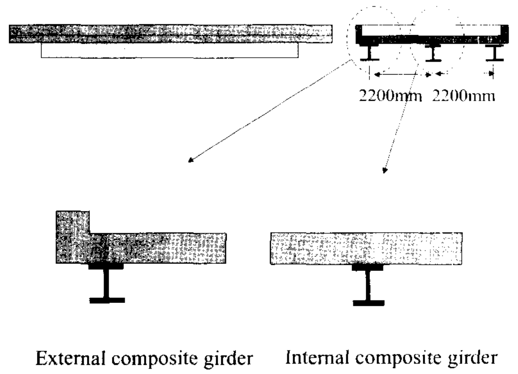



Fig. 4 Composite concrete/steel girder bridge: 3 girders reinforced and epoxy injection[2]

## Ⅳ. Result of Rating and Reinforcement

### 1. Preliminary Rating

Since DB 13.5 was a load of design truck of the bridge,[14] the bridge was rated by using the same design truck as shown in Fig. 3 Rating values of slab bridge calculated as previous work based on ASDR method. Inventory rating value is 0.22 and operating rating value is 0.66. The operating rating values are less than 1.0, which it means that ultimate load will be bigger than the design load.

Based on the test result, the bridge is reinforced with steel girders as shown in Fig. 4. Reinforcement is performed to improve the target rating operating value at least one. Steel girders of H200-200-4-4 of KS Standard were used to make composite bridge. The section properties are in Table 2.

### 2. Rating Values Of Reinforcing Element

*A. AASHTO Rating*

After reinforcement, AASHTO inventory rating factor for ASDR[10] is calculated to determine the load-carrying capacity of the deteriorated bridge by using Eq. (1), (2), (3), and

Table 2 Section properties of reinforced superstructures with steel girder

| Girder section area ($cm^2$) | Internal girder | | | External girder | | |
|---|---|---|---|---|---|---|
| | Inertia moment ($cm^4$)($I_i$) | Section modulus of Top ($cm^3$) ($S^i\_t$) | Section modulus of Bottom ($cm^3$) ($S^i\_b$) | Inertia moment ($cm^4$)($I_e$) | Section modulus of Top ($cm^3$) ($S^e\_t$) | Section modulus of Bottom ($cm^3$) ($S^e\_b$) |
| 23.68 | 16,910.54 | 1,395.63 | 606.48 | 19,129.07 | 719.23 | 673.48 |

Eq. (4). In this study the girder and concrete assume that they are activated as composite members built by epoxy injection between the members.

Table 3. 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 |

*B. Load Test Rating*

The bridge was loaded in two separate stages. The first, load was applied primarily quasi-static load with very slow speed. The second, load is applied with higher speed to consider the dynamic load effects. Testing load truck is shown in Fig. 5. Because the distance between front wheel and rear wheel is shorter than the rating truck, measured vertical deflection at a middle of girder will be bigger than expected value during the load test. The result is plotted in Fig. 6. This figure shows two deflections made by same load before and after reinforcement. Deflections are measured while the truck are approaching on the bridge deck. As we can see b) of Fig. 6, bridge are shown well-reinforced manner.

Their results are measured by LVDT and strain gages at the bottom of a center girder. However relationship data of load-strain was lost, experimental rating factor using Eq. (5) can't be calculated. Usually, rating factor from the result of test is bigger than that of AASHTO's because of distribution factor, impact factor, unintended support restraints, etc[6),7),9),10),13),15)] as shown in Fig. 7. Because AASHTO rating factor shown in Table 3 after reinforcement exceeds the target operating rating value, it can be concluded that rating factors from test is bigger than the target operating rating factor.

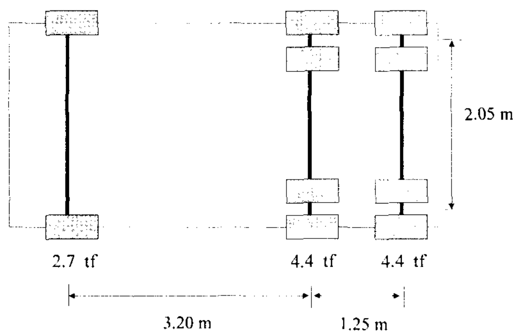
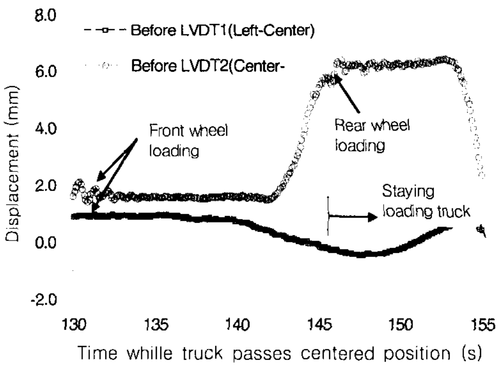
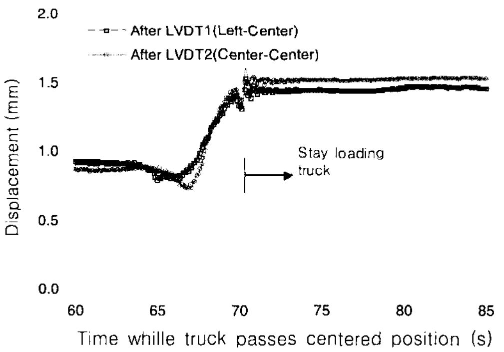



Fig. 5 Plain view of load testing truck




a) Before reinforcement




b) After reinforcement

Fig. 6 Vertical deflection of centered position

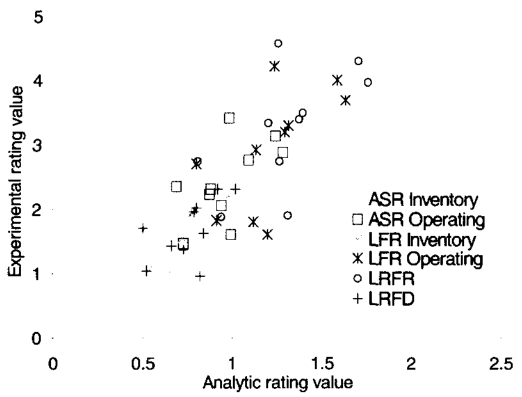


Fig. 7 Relationship between analytic rating values and experimental values

## V. Conclusions

A load carrying capacity assessment of deteriorated rural bridge was used to determine rehabilitation of aged rural bridges. The rating factor method was applied to simple support slab bridge. This method can be effectively applied to any other aged rural bridges.

1. Diagnostic load test was performed on a bridge in rural area. It is assumed that the bridge type in rural area is slab bridge and the spans are short. Based on AASHTO method and test results, the rating computations were tried.

2. It was designed for the bridge to exceed the operating rating values 1.0 after reinforcement. 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.

However, this approach should be made up for the estimation of relationship between their load-capacity and safety estimated by periodical inspection of deteriorated rural bridges. It is required to define the damage state or probability of failure of the bridges and rehabilitation methods increasing its resistance against load with expected occurrences. When it could be provided with useful informations of the structural behavioral properties, proposed approach can be used to evaluate load-carrying capacity of deteriorated bridge and to make a plan how to make management and reinforcement support system.

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