교량평가법에 의한 내하력 비교에 관한 연구

A Study on the Comparison of Load-carrying Capacity by the rating Methods of Bridges

한 상 철[1] · 양 승 이[2]

Han, Sang Chul Yang, Seung Ie

요 약 : 현존 교량의 절반 가량이 성능이 저하되어 보수 보강이 요구되고 있는 실정인데 이들 교량은 제 기능을 발휘하지 못하거나 소요강도에 미달되는 경우가 있다. 필요이상의 보수 보강비용 또는 재건설의 비용의 투입을 피하기 위하여 현재 상태의 내하력이 정확히 평가되어져야 한다. 교량의 평가방법으로 허용응력 평가법 (ASD), 하중계수평가법 (LFD), 하중저항계수평가법 (LRFD)등이 현재 사용 되고 있다. 본 논문에서는 허용응력평가법 (ASD), 하중계수평가법 (LFD), 하중저항계수평가법 (LRFD)등이 비교되고, 실제 교량에 적용된 하중실험의 자료들을 모았다. 그리고 하중실험의 교량평가 결과와 이론에 의한 교량평가 결과의 차이점에 대해 연구 하였다. 그리고 기존에 존재하는 교량에 ASD, LFD 그리고 LRFD 방법을 적용 비교하여 어느 수준에 해당되는지를 비교 검토하였다.

ABSTRACT : About half of bridges in United States are considered to be deficient and therefore are in need of repair or replacement. Half of these are functionally obsolete, and others do not have required strength. For these 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 bridge load test data are collected. The reasons that make the difference between test results and analytical results are explained for each bridge load test. And, load rating methods are applied to real bridge. The rating factors from each method are compared.

핵 심 용 어 : 내하력, 교량평가, 허용응력평가, 하중계수평가, 하중저항계수평가

KEYWORDS : load carrying capacity, bridge rating, allowable stress rating, load factor rating, load and resistance factor rating

1) 정회원, 서울산업대학교 토목공학과 교수
2) 미국 콜로라도대학교 토목환경공학과 박사과정

본 논문에 대한 토의를 2002년 4월 30일까지 학회로 보내주시면 토의 회답을 게재하겠습니다.
본 논문은 서울산업대학교 학술문화연구재단 학술연구비 지원에 의하여 연구되었습니다.

1. Introduction

About half of bridges in United States are considered to be deficient and therefore are in need of repair or replacement. Half of these are functionally obsolete, and others do not have required strength [*Status* 1991]. For these bridges, repairs and replacements are needed. In order to avoid the high cost of rehabilitation, the bridge rating must correctly report the present load-carrying capacity. The bridge rating is performed by AASHTO's method [*Manual* 1983, *Condition* 1994].

Load ratings may also be determined from diagnostic load tests, often by extrapolation of stresses observed at a test load to stresses that may exist at a rating load. From many diagnostic load test results, it is observed that the actual load-carrying capacity of a bridge is usually higher than the computed strength.

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 [*Manual* 1983], 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 [*Manual* 1983], the operating rating is a maximum permissible load to which a structure may be subjected".

In this paper, the load rating methods are introduced and bridge load test data are collected. The reasons that make the difference between test results and analytical results are explained. And, as an example, AASHTO's load rating methods are applied to real bridge.

2. Load Rating Procedure

The bridges are rated by the following general equation for moment.

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

Where

RF = Rating factor

M = The moment strength of the controlling member of a bridge. Computing these is different for ASD and LFD.

γ_L = Dead load factor

M_{Dead} = The dead load moment on the member.

M_{LL} = The live load demand moment on the member with distribution factor

I = Impact Factor

γ_L = Dead load factor

γ_L = Dead 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.

Table 1 Live and Dead Load Factors

	ASD		LFD	
	Inventory	Operating	Inventory	Operating
Dead Load Factors, γD	1.00	1.00	1.30	1.30
Live Load Factors, γL	1.00	1.00	2.17	1.30

In addition, the load and resistance factor design rating method (LRFD), which was proposed in 1989 [*Guide* 1989], can be used to rate the bridges. The live load, dead load, and resistance factors will be explained in later section.

2.1 Beam Line Analysis in AASHTO

For multi-beam bridges of moderate span, AASHTO allows the use of factors based on beam spacing to apportion traffic loads among beams. A cross section of a concrete slab steel girder is shown in Fig. 1.

If a truck is moving over the bridge, the truck load is transmitted from the deck to the girders and then to the substructure. Girders immediately under the truck carry the most loads.

There are several methods to compute the load the specific girder carries. AASHTO's method is called Beam Line Analysis. Beam line analysis estimates the biggest load on one girder by using distribution factors (DF). AASHTO specification [*Standard* 1992] provides wheel load distribution factors for various combinations of decks and girders. The wheel-load distribution factors for interior girders are function of girder spacing. For example, concrete slab steel girder bridges with two traffic lanes

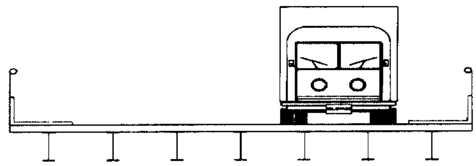


Fig. 1 The Cross Section of Concrete Slab Steel Girder

or more, the distribution factor, is

$$DF = \frac{S}{5.5} \tag{2}$$

Where

S = Girder spacing in feet

The live load bending moment of exterior girders is determined by applying to the girder reaction of the wheel load obtained by assuming the deck to act as a simple span between girders. This method is called "Lever Arm".

The HS-20 load is a the application of either a single truck in each traffic lane, or a distributed loading in each traffic lane, whichever produces the greatest live load moment. The HS-20 load is used in ASD and LFD design provisions. The live load moment demand is calculated as following.

$$M_{LL} = \frac{M_{HS20}}{2} \times DF \tag{3}$$

2.2 Allowable Stress Design Rating

The AASHTO maintenance manual [*Manual* 1983] provides the guideline for load rating.

Fig. 2 shows a procedure for ASD load rating. In allowable stress design rating, the each material (steel, concrete, timber, etc) has specified allowable stresses for each of two rating levels, inventory and operating. The inventory and operating strengths are computed by using these allowable stresses. For example, the equations of inventory and operating moment strength are in equation 4 for concrete slab steel girder bridge with non-composite section.

$$\begin{aligned} M_{inv} &= S_{non} \times f_{inv} \\ M_{ope} &= S_{non} \times f_{ope} \end{aligned} \tag{4}$$

Where

M_{inv} = Moment strength in inventory level

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*]

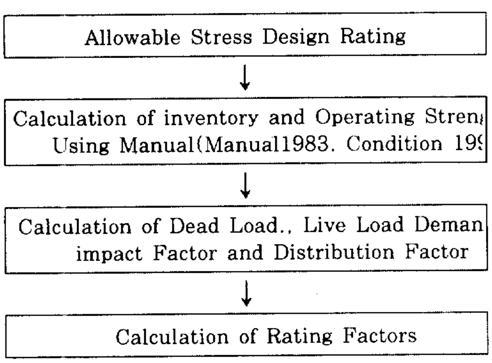


Fig. 2 ASD Rating Procedure

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. As an example, the cross section shown in Fig. 3 is used.

In order to calculate dead load for interior girder, tributary width for interior girder is determined as following.

$$Tributary\ Width = \frac{S_1 + S_2}{2} \tag{5}$$

After tributary width of concrete deck is decided, the following cross section is obtained.

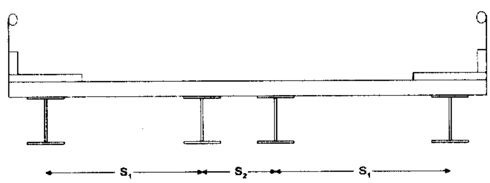


Fig. 3 Cross Section of the Bridge

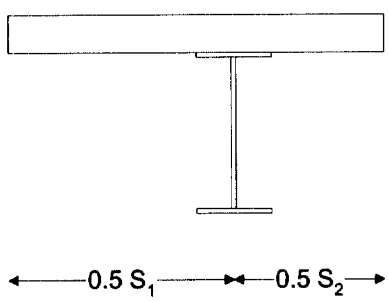


Fig 4 Cross Section with Tributary Width

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]. 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, there is an equation for impact factor in AASHTO specification [*Standard* 1992] and this is in equation 6.

$$I = \frac{50}{125 + L} \quad (6)$$

Where

I = Impact factor (maximum 30 %)

L = Length in feet 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 equation 1.

2.3 Load Factor Design Rating Method

LFD load ratings follow the strength design provisions in the AASHTO design specification [*Standard 1992*]. There is a procedure for load rating using LFD.

The moment strength of steel bridge is summarized in table 2.

Load Factor Design Rating

↓

Calculation of Maximum Strength by Using

↓

Calculation of Dead Load., Live Load Deman
impact Factor and Distribution Factor

↓

Calculation of Rating Factors

Fig. 5 LFD Rating Procedure

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

At inventory level for steel girders, the overload limitation specified in article 10.57 of AASHTO specification [*Standard* 1992] must be checked. Overloads are defined as loads that can be allowed on structures infrequently without causing permanent damage. Overload provision checks for yielding in steel. For overload, the live load factor and dead load factor for inventory rating computation are 1.67 and 1.00, respectively. So, there are two rating factors at inventory level. For the operating rating computation, the live load factor and dead load factor are 1.00 and 1.00, respectively. The moment strengths for overload limitation are summarized in table 3, respectively.

For reinforced concrete, moment strength is computed as the ultimate moment strength. There is a table for yield stresses of reinforcing steel.

At the inventory level, it is required to check the cracking serviceability limit state for prestressed concrete. The cracking moment is the moment that produces net tensile stresses equal to the rupture modulus of concrete. The live load factor and dead load factor are 1.00 and 1.00, respectively. As a result, there are two rating values at inventory level.

Table 3 Moment Strength for Overload Limitation Check

Type of cross section	Moment strength
M for Non-composite	$0.80f_yS$
M for Composite	$0.95f_yS$

Table 4 Yield Stresses for Reinforcing Steel

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

In rating calculation, the live load factor and dead load factor for inventory level are 2.1667 and 1.3, respectively. The live load factor and dead load factor are 1.3 and 1.3 for operating rating.

2.4 Load and Resistance Factor Load Rating

Load and resistance factor rating was proposed in an AASHTO guide specification in 1989 [*Guide* 1989]. The LRFD approach produces a single bridge rating. Beam strengths are ultimate strengths. Load factors are set according to the frequency of trucks on the bridge, and the possibility of uncontrolled overweight loads. Resistance factors are adjusted to reflect deterioration in bridges. Distribution factors are the same as ASD and LFD distribution factors. There is figure to explain the LRFD procedure in Fig. 6.

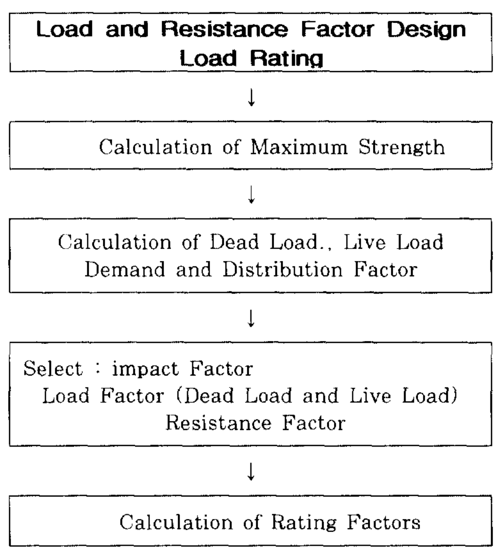


Fig. 6 LRFD Rating Procedure

Although in both ASD and LFD rating there are two levels of rating (inventory and operating), in LRFD a single load rating is produced. The LRFD rating factor may exceed the operating rating factor computed by using LFD and ASD for the bridges which are in good condition, receive frequent qualified inspection, have adequate maintenance programs and loads corresponding to reasonable levels of traffic and enforcement. Conversely, the bridges which do not maintain these conditions or have non-redundant critical components have their rating falling close to inventory levels or even lower.

There are important differences between ASD/LFD ratings, and LRFD ratings. The impact factor for ASD and LFD, I, is calculated by the equation 6. But in LRFD, impact factor is selected as following.

Table 5 Impact Factors

	Impact Factor
Smooth approach and deck conditions	0.1
Rough wearing with bumps	0.2
Under extreme adverse condition of high speed, span less than 40 ft and highly distressed pavement and approach condition	0.3

If such a judgment cannot be made, the impact factor is related to the condition of wearing surface. Explanation is table 6.

Table 6 Impact Factor

Condition of Wearing Surface		Impact factor
Good Condition	No repair required	0.1
Fair Condition	Minor deficiency, item still functioning as designed	0.1
Poor Condition	Major deficiency, item in need of repair to continue functioning as designed	0.2
Critical Condition	Item no longer functioning as designed	0.3

The dead load factor, γ_D, is 1.2. The live load factor, γ_L, is a function of the average daily truck traffic and of the regulation of overloads. The live load factors vary from 1.3 for a low volume roadway (ADTT $<$ 1000) with controls of overloads, to 1.8 for a heavy volume roadway (ADTT $>$ 1000) with significant sources of overloads without effective enforcement of overload.

Resistance factors depend on the material, on the condition of girders, on redundancy of structures, on maintenance and on the nature of inspections. For redundant structures in good condition, resistance factors range from 0.95 for steel girders and prestressed concrete girders to 0.90 for reinforced concrete girders. For non-redundant structures in heavily deteriorated condition, resistance factors are 0.55 for steel girders, prestressed concrete girders, and reinforced concrete girders. If section dimensions are measured during inspections, resistance factors may be increased by 0.05, but can not be greater than the values for girders in good condition. If maintenance activity corrects the deficiencies which may lead to further section loss, the resistance factor is increased by 0.05. If maintenance may not correct the deficiencies, the resistance factor is decreased by 0.05. But, the maximum value of resistance factor cannot exceed the values for girders in good condition.

The moment strength computation is the same as that of LFD.

Dead loads for LRFD ratings are computed based on tributary areas for members. Live loads differ. LRFD uses three rating vehicles, and a distributed lane load for long spans. The live load for calculating moment demand shall be the three AASHTO trucks shown in Fig. 7.

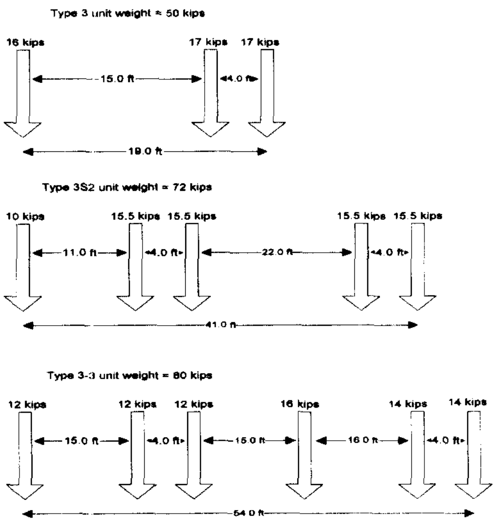
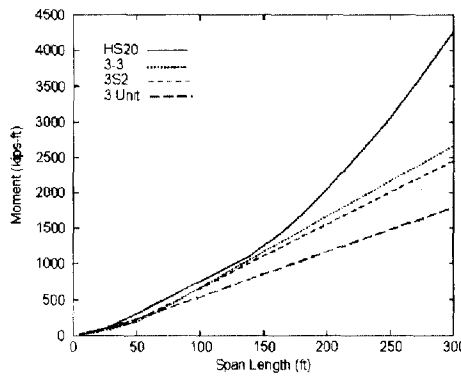


Fig 7 Axle Weight of AASHTO Rating Trucks in LRFR (adapted from Guide 1989)

One of them, which produce the greatest live load moment, is used in rating calculation.

The moment due to these three trucks and HS20 are computed and plotted in Fig. 8 for simple span.



For longer spans the lane-type loading shown in Fig. 9 governs the evaluation (up to 300 ft).

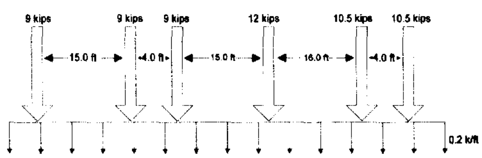


Fig 9 Axle Weight and Lane Loading (adapted from Guide 1989)

Table 7 Collection of Load Test Results

Reference Name (Rating Truck)	Span length (ft)		Rating Factor of AASHTO's ASD		Rating Factor of AASHTO's LFD		Rating Factor from Test Result	
			Inventory Level	Operating Level	Inventory Level	Operating Level	Inventory Level	Operating Level
Chajes [et al. 1997] (HS20)	64.0	Non	0.760	1.430	-	-	1.520	3.090
		Com	1.430	2.360	-	-		
Kissane [et al. 1980] (HS20)	39.5		1.310	-	-	-	3.800 (A)	-
	37.5		1.414	-	-	-	3.400 (A)	-
	32.0		1.723	-	-	-	4.700 (A)	-
	12.5		2.372	-	-	-	7.700 (A)	-
	27.0		1.567	-	-	-	3.100 (A)	-

Moses [et al. 1985] (HS20)	81.0	Non	0.864	1.404	-	-	1.018 (A)	1.751 (A)
		Com	1.543	2.330	-	-	1.877 (A)	2.943 (A)
	50.0	Non	1.289	1.844	-	-	1.122 (A)	1.627 (A)
		Com	2.039	2.856	-	-	1.762 (A)	2.491 (A)
	70.0	Non	1.059	1.594	-	-	1.363 (A)	2.128 (A)
		Com	1.657	2.406	-	-	2.279 (A)	3.400 (A)
	73.0	Non	1.382	2.065	-	-	2.174 (A)	3.241 (A)
		Com	2.283	3.283	-	-	3.687 (A)	5.284 (A)
	85.0	Non	-	-	-	-	-	-
		Com	1.930	2.827	-	-	2.327 (A)	3.603 (A)
Fu [et al. 1997] (HS20)	53.0		0.700	-	1.71	-	1.430 (A) 1.710 (L)	-
Beal and Loftus [1988] (H20)	34.0		-	0.900	-	-	-	2.100 (A)
	54.0		-	1.100	-	-	-	1.700 (A)
	44.0		-	1.600	-	-	-	2.650 (A)
	77.0		-	2.050	-	-	-	3.450 (A)
	34.0		-	1.400	-	-	-	2.950 (A)
	44.0		-	1.500	-	-	-	2.700 (A)
	49.0		-	1.350	-	-	-	3.000 (A)
	Continuous span		-	1.950	-	-	-	2.600 (A)
Beal and Loftus [1988] (H20)	120.0		0.520	1.030	0.790	1.320	0.890 (A) 1.360 (L)	1.760 (A) 2.270 (L)
	100.0		0.750	1.260	0.950	1.580	1.150 (A) 1.440 (L)	1.920 (A) 2.410 (L)
	70.0		0.400	1.410	0.810	1.340	0.510 (A) 1.020 (L)	1.800 (A) 1.710 (L)
	50.0		0.490	1.290	0.780	1.300	1.220 (A) 1.950 (L)	3.230 (A) 3.250 (L)
	60.5		0.560	1.600	0.920	1.530	1.040 (A) 1.710 (L)	2.980 (A) 2.850 (L)
	60.0				0.583	1.000	0.806 (L)	1.250 (L)

Where
Non = Non-composite section
Com = Composite section
(A) = ASD rating factor from test results
(L) = LFD rating factor from test results

3. Collection of Load Test Data

In this section, the results of load test are presented for each researcher. Table 7 shows the collection of load test data.

Kissane [et al. 1980] tested four concrete girder bridges and one slab bridge.

Chajes [et al. 1997] calibrated finite element models (FEM) using test data. Although the bridge was simply supported and constructed as non-composite, test result revealed that the bridge showed both fully composite action and unintended support restraint. These effects were considered in computer analysis. In rating calculation, AASHTO's impact factor was used. Yoo and Stallings [1993] tested five bridges. In some bridges, the load tests were conducted on more than one span in the bridge. Static and dynamic tests were done and rating results were scaled to several standard truck types. In table 7, just H20 rating factors are presented.

In the case of Moses' [et al. 1985] test, weigh-in-motion methods were extended to the evaluation and rating of existing bridges. From the test data, the distribution factor and impact factors were computed. By comparing stresses measured during weigh-in-motion, it was shown that all bridges built as non-composite section acted compositely. In rating calculation, the support restraints were not considered. The ratings were conducted for both non-composite and composite sections.

Fu [et al. 1997] tested a concrete slab steel girder bridge which was built as composite section. The AASHTO rating calculations were conducted with different boundary conditions. The first one was that the bridge was simply supported. The second was that one end was simple and the other end was fixed. And then, the rating factors calculated by AASHTO's method were compared with that of test result. The test rating factor was scaled to HS20 truck.

Beal and Loftus [1988] tested five bridges which included two truss bridges and three concrete slab steel girder bridges. For truss bridges, the critical members were deck girders. When bridges acted compositely, the cross sections were identified as fully composite sections. To calculate the rating factors, the test load and design section modulus were scaled to H20 truck and section modulus the bridges had at load test time, respectively. Test results showed that AASHTO distribution factors for five bridges were conservative.

3.1 Findings

In literature survey, the rating factors from load tests were compared with AASHTO's method. In AASHTO's method, ASD and LFD are usually used.

Most of rating factors in table 7 of section 3 are HS20 rating factor although the test trucks used in the test were not HS20 truck. The rating factors from the test truck were scaled by using equation 7, which was used by Kissane [et al. 1980], Yoo and Stallings [1993], Fu [et al. 1997], and Stalling and Yoo [1993].

$$RF_{HS20} = \frac{f_i - f_{iDL}}{(1+I)\sigma_{iTruck}} \frac{M_{truck}}{M_{HS20}} \qquad (7)$$

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

From test results, distribution and impact factors were calculated and compared with that of AASHTO. The comparisons revealed that AASHTO's distribution and impact factors were conservative for most cases.

As a result of comparison of rating factors, rating factors from test are larger than that of AASHTO's methods except one case. In the one exception case, weigh-in-motion was extended to evaluate the bridge. In this site, distribution factor of AASHTO was not conservative comparing with that of test result because the girder spacings was small (5.75 ft).

So, it is possible to say that the actual load-carrying capacity of a bridge is usually higher than the analytical strength (the results of AASHTO).

From literature survey, bridges showed unintended-composite action. This kind of action makes the bridges stiffer. The figure shown in Fig. 10 is used to explain unintended composite action.

The bridge is simply supported and built as a non-composite section. If the arbitrary three-axle truck is on the bridge, the maximum stress due to this load by beam line analysis method is following.

$$f = \frac{M_{max}}{2S_{non}} \tag{13}$$

Where

f = Stress due to applied truck

M_{max} = Maximum Moment due to truck

S_{non} = Section modulus of steel girder

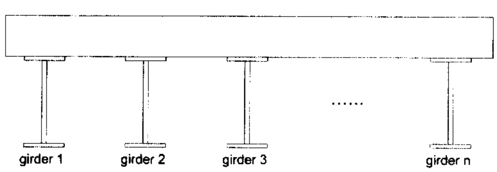


Fig 10 The Cross Section of Concrete Slab Steel Girder Bridge

If the bridge is built as composite section and truck is applied, the stress due to this load is calculated by using equation 13 except non-composite section modulus. Because the composite section modulus is bigger than non-composite section modulus, stress calculated by composite section modulus is smaller than that of non-composite section modulus. In Fig. 11, moment diagrams are shown for comparison

In rating calculation for non-composite bridges, AASHTO's method uses non-composite section modulus and test results involve composite actions because bridges show unintended composite action.

Also, bridges showed unintended support restraints from test results reported in section 3. In rating of AASHTO's method, the bridge is considered to have ideal boundary conditions. For example, there is no moment at supports where the bridge is simply supported. But, test results showed that there were moments at support although

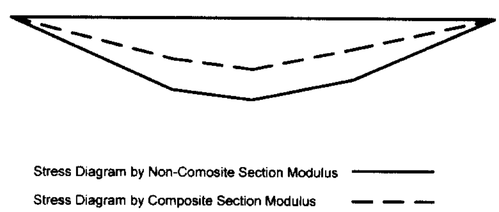


Fig. 11 Comparison of Stress Diagram

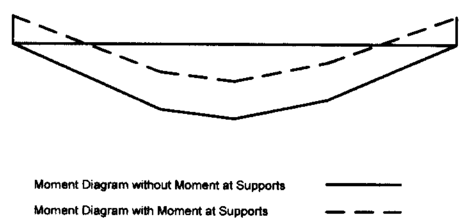


Fig. 12 Comparison of Moment Diagram

the bridges are simply supported. It is called unintended support restraints. The following figure explains the unintended support restraint.

Moments at support decrease the moments along the span. Because the moment at mid-span is used to rate the bridges, the rating factor is larger than that of AASHTO's method.

The rating data in table 7 are plotted on the normal probability paper to compute the mean and standard deviation of rating factors.

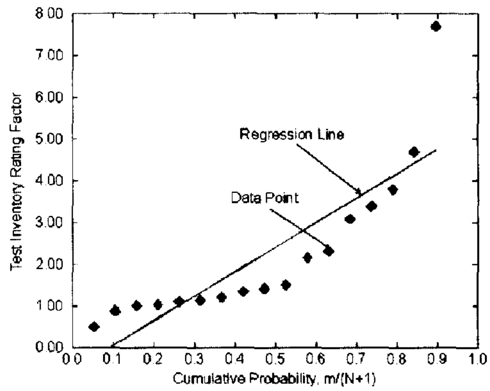


Fig. 15 Normal Probability of Test Rating Factors for Inventory Rating (ASD)

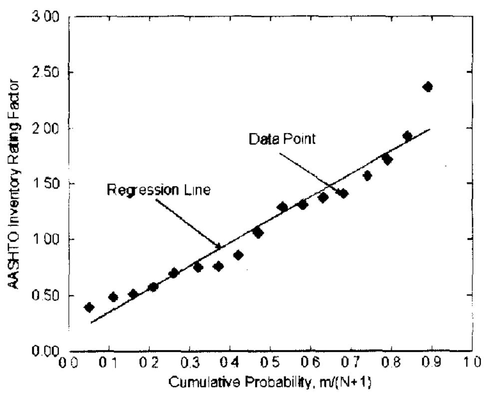


Fig. 13 Normal Probability of AASHTO's Rating Factors for Inventory Rating (ASD)

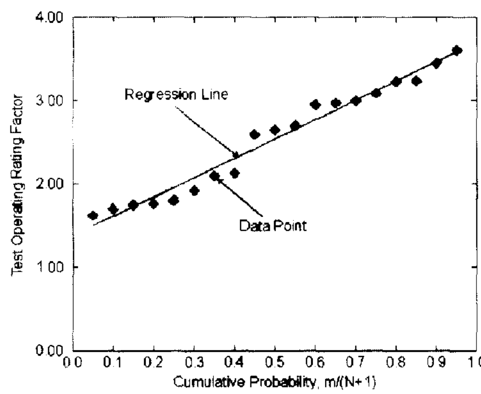


Fig. 16 Normal Probability of Test Rating Factors for Operating Rating (ASD)

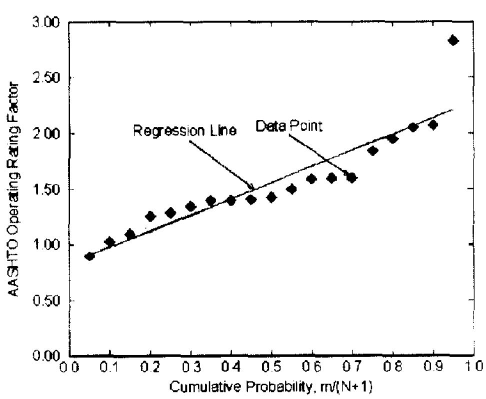


Fig. 14 Normal Probability of AASHTO's Rating Factors for Operating Rating (ASD)

The means and standard deviations of AASHTO's rating factors and test rating factors from normal probability papers are in following table.

Table 8 Mean and Standard Deviation

		Inventory	Operating
AASHTO	Mean	1.179	1.558
	Stand. Dev.	0.701	0.492
Test	Mean	2.417	2.541
	Stand. Dev.	1.998	0.790

Probability density functions are shown in following figures.

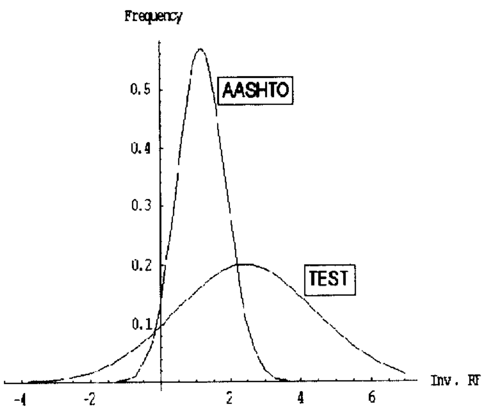


Fig. 17 PDF of Inventory Rating Factors

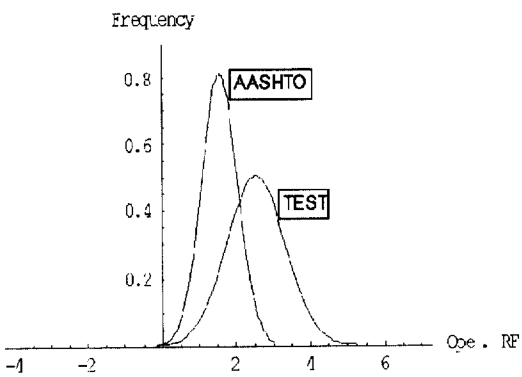


Fig. 18 PDF of Operating Rating Factors

If there is an assumption that the area below than rating value 1 is failure probability, the failure probability for each case is computed.

This table also shows that AASHTO's method is conservative, because test results show the real load carrying capacity of bridges.

Table 9 Failure Probability

		Failure Probability
AASHTO	Inventory	0.399
	Operating.	0.128
Test	Inventory	0.239
	Operating.	0.025

4. Example of AASHTO's Rating Method

For the case of LRFD, two computations are compared. In the one, a bridge in good condition with light traffic is assumed. In the other, poor conditions and heavy traffic are assumed. The followings are assumed for the first computation.

- Wearing surface is in good condition (No repair required).
- The bridge is redundant.
- Average daily truck traffic (ADTT) is less than 1,000.
- The bridge is carefully inspected.
- The bridge is vigorously maintained.
- The bridge has reasonable enforcement and apparent control of overload.
- Wearing condition is good condition

The followings are assumed for the second computation.

- Wearing surface is in critical condition (No longer functioning as designed).
- ADTT is greater than 1,000.
- The bridge is not carefully inspected.
- The bridge is not vigorously maintained.
- The bridge has significant of overloads without effective enforcement.
- The wearing condition is critical condition (no longer functioning as designed).

4.1 Bridge Description

The table 10 contains information needed

Table 10 Bridge Description

Bridge type	Concrete slab steel bridge
f_y	32.92 ksi
f_c'	2.5 ksi
Construction year	1940
Span length	64 ft
Interior girder	W36170
Girder spacing	5 ft
Thickness of slab	8.5 in

to rate the bridge.

4.2 Moment Strength

Allowable inventory and operating bending stresses are 18 ksi and 24.5 ksi from AASHTO maintenance manual [*Manual* 1983], respectively. Moment strengths of ASD are

$$Inv = 18ksi \times 580in^3 = 870kips - ft$$

$$Ope = 24.5ksi \times 580in^3 = 1184kips - ft$$

The interior girder W36×170 satisfies the compact section requirement for AASHTO specifications [*Standard* 1992]. So, the moment strength is plastic moment strength, f_yZ_s for LFD, and LRFD.

$$f_yZ_s = 32.92ksi \times 668in^3 = 1833kips - ft$$

Table 11 Moment Strength for Each Method (Non-Composite)

	ASD		LFD	LRFD	
	Inv	Ope			
Moment Strength	870 kip-ft	1184 kip-ft	1833 kip-ft	1833 kip-ft	
Resistance Factor	-		-	0.95 (1)	0.70 (2)

Where

(1) = Careful inspection, vigorous maintenance, and redundancy

(2) = No careful inspection, no vigorous maintenance

4.3 Dead Load Moment Demand

Although the dead load factor is different for LFD and LRFD, the unfactored dead load moment is same for each method.

The dead load moment demand and dead load factors are in following table.

Table 12 Dead Weight of Each Materials

Weight of concrete	Weight of steel	Weight of asphalt
0.530 kip/ft	0.170 kip/ft	0.15 kip/ft

* Additional 0.1 kip/ft of Girder of Miscellaneous railings, sidewalks, curbs, and so on.

Table 13 Dead Load Factor and Dead Load Moment Demand

	ASD	LFD (Inventory)	LRFD
Dead Load Factor	1.000	1.300	1.200
Dead load moment Demand	$(0.53+0.17+0.15+0.10)\frac{63.98^2}{8} = 486kip-ft$		

4.4 Live Load Moment Demand

Live load factor for each rating method is summarized in table 14.

Table 14 Live Load Factor

	ASD	LFD (Inventory)	LRFD	
Live Load Factor	1.000	2.167	1.300 (1)	1.800 (2)

Where

(1) = ADTT is less than 1,000 and the bridge has reasonable enforcement and apparent control of overload.

(2) = ADTT is greater than 1,000 and the bridge has significant of overloads without effective enforcement

ASD, LFD, and LRFD use older wheel-load distribution factors specified in AASHTO [*Standard* 1992].

Table 15 Fraction of a Wheel Load Carried by an Interior Girder

	ASD	LFD	LRFD
Distribution Factor	0.907	0.907	0.907

Although impact factors for ASD and LFD are same, that of LRFD are different. The summary of impact factors is in table 16.

Table 16 Impact Factors

	ASD	LFD	LRFR	
Impact Factor	0.265	0.265	0.1 (1)	0.3 (2)

Where

(1) = The wearing surface condition is good condition (no repair required)

(2) = The wearing surface is critical condition (no longer functioning as designed)

HS20 truck is used for ASD and LFD ratings. For LRFD, the critical live loads are 3S2 truck. The results are in table 17.

Table 17 Live Load Moments in Beams due to Rating Truck

	ASD	LFD	LRFD	
Live Load moment	505 kip-ft	505 kip-ft	340 kip-ft (1)	402 kip-ft (2)

Where

(1) = Live load moment demand with impact factor 0.1

(2) = Live load moment demand with impact factor 0.3

4.5 Comparison of Rating Factors

The rating factors are calculated by using equation 1 and the results are in table 18.

Table 18 Rating Factors

	ASD		LFD		LRFD	
	Inv	Ope	Inv	Ope	(1)	(2)
Rating Factor	0.761	1.383	1.100	1.831	2.621	0.968

where

Inv = Inventory rating factor

Ope = Operating rating factor

4. Conclusion

The rating factor of load test is higher than that of analytical results (AASHTO's method). In the case of inventory level, the mean of test results is 51 % bigger than that of analytical results. For operating level, there is 39 %. And, the probability of failure of AASHTO is bigger than 40 % that of test at inventory level. For operating level, there is 80%. There are several factors which make the difference between rating factor of AASHTO and that of load test results. From literature survey, it is known that these are unintended composite action, unintended support restraint, impact factor, and distribution factor.

The results of AASHTO's rating example

show that LFD rating factors are bigger than ASD rating factors (inventory level: 31 %, operating level: 24 %). As expected in LRFD, when bridge is in good inspection, maintenance, and reasonable level of traffic and enforcement, the rating factor exceeded the operating levels of ASD and LFD. Rating factor for bridge which is not in good inspection, maintenance, and reasonable level of traffic and enforcement, gave almost same level of inventory rating of LFD and LRFD.

Reference

1. *Standard specification for highway bridges* (1992). American Association of State Highway Transportation Officials, Washington, D.C., P 686.
2. *Manual for maintenance inspection of bridges* (1983). American Association of State Highway Transportation Officials, Washington, D.C., P 53.
3. *Guide specification for strength evaluation of existing steel and concrete bridges* (1989). American Association of State Highway Transportation Officials, Washington, D.C., P 23.
4. *AASHTO LRFD Bridge Design Specification* (1994). American Association of State Highway Transportation Officials, Washington, D.C.
5. The status of nation's highway and bridge: condition and performance and highway bridge replacement and rehabilitation program. (1991) U.S Department of Transportation, FHWA., Washington, D.C., P 173.
6. *Manual for Condition Evaluation of bridges* (1994). American Association of State Highway Transportation Officials, Washington, D.C., P 136.
7. Chajes, M. J., Mertz, D. R., and Commander, B., (1997). "Experimental load rating of a posted bridge", ASCE Journal of Bridge Engineering, Vol 2 (n1), PP 1 10.
8. Kissane, R. J., Beal, D. B., and Sanford, J. A. (1980). "Load rating of short-span highway bridges", FHWA/NY/RR-80/79, New York State Department of Transportation, P 35.
9. Moses, F., Ghosn, M., and Gobieski, J., (1985). "Weigh-in-motion applied to bridge evaluation", FHWA/OH-85/012, Case Western Reserve University, P 168.
10. Fu, G., Pezze, F. P., and Alampalli, S., (1997). "Diagnostic Load Testing for Bridge Load Rating", Transportation Research Record, No 1594, PP 125 133.
11. Yoo, C. H., and Stallings, J. M., (1993). "Bridge testing", Alabama Department of Transportation, Auburn University, P 85.
12. Beal, D. B., and Loftus, M. J., (1988). "Load tests on five bridges", Engineering Research and Development Bureau, New York State Department of Transportation, P 19.

(접수일자 : 2001년 7월 18일)