교량내하력 값에 기초한 초과하중 확률 계산에 관한 연구

A Study on the Computation of Overload Probability Based on Bridge Load Rating Factor

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Abstract

In order to rate current bridge load carrying capacity, typically two methods are used. These are Allowable Stress Rating (ASR) and Load Factor Rating (LFR). Using the rating factors, there are many attempts to make a connection between rating factors and probability concept. The main purpose of the paper is computing the probability of overload using rating factors and probability concept. In this paper, the load rating methods are briefly explained, and the probability concept is connected to rating factors by using live load from Weigh-in-Motion (WIM). Based on the live load model and rati ng factor, the computation procedure of the probability of overload is explained.

요 지

교량의 현 내하력을 평가하기 위해서 사용되는 방법으로 허용응력 평가법(ASR), 하중계수평가법 (LFR) 등의 방법 등이 사용되고 있다. 현재, 교량 평가 값을 이용하여 이 값을 확률이론에 연결시키려는 시도들이 많이 연구되고 있다. 본 논문의 주목적은 교량의 내하력 평가값 (Rating Factor)과 확률이론을 이용하여, 과하중 확률을 구하는데 있습니다. 본 논문에서는 이러한 평가 방법들을 요약 설명하고, Weigh-in-Motion 에 의해 얻은 활하중 모델을 도입하여, 교량 평가 값을 확률이론에 연결 시켰다. 활하중 모델과 교량 내하력 값을 토대로, 초과 하중 확률을 계산하고, 그 방법을 설명하였다.

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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 (FHWA 1989). 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 computation (AASHTO's method) based on available drawings and idealized boundary conditions. Load raconsider damage and deteriorated section discovered during field inspection.

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.

The load rating of bridge is performed according to the AASHTO manual (AASHTO 1983 ; AASHTO 1994). 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 .

In AASHTO's maintenance manual (AASHTO 1983 ; AASHTO 1994), the inventory rating is the load which can safely utilize an existing structures for an indefinite period and generally corresponds to the customary design level of stresses. 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 (AASHTO 1983), the operating rating is a maximum permissible load to which a structure may be subjected, and allowing unlimited numbers of vehicles to use the bridge at operating level may shorten the life of the bridge.

In this paper, the load rating methods are explained, and live load data are collected from Weigh-in-Motion (WIM). Based on WIM data, the probability of overload is computed for the bridge which has the rating values.

2. Load Rating Procedure

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

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

The live load factors (γ_L) and dead load factors (γ_D) used in general rating equation are in Table 1 for allowable stress design and load factor design 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

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. A cross section of a concrete-slab-steel-girder is shown in Fig. 1.

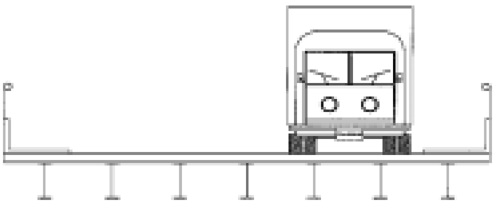

Fig. 1 The Cross Section of Concrete-Slab-Steel-Girder

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 (AASHTO 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 or more, the distribution factor, is

$$DF = \frac{S}{5.5} \quad (2)$$

S = Girder spacing in feet

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

Using ASD distribution factor, the equation for maximum live load moment in a girder due to HS20 truck is expressed in Eq. (3).

$$M_{LL} = \frac{M_{HS20}}{2} \times DF \quad (3)$$

2.2 Allowable Stress Design Rating

The AASHTO maintenance manual (Manual 1983) provides the guideline for load rating. There is a procedure for ASD.

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. As an example, the equations of inventory and operating moment strength are in Eq. (4) for concrete slab steel girder bridge with non-composite section.

Allowable Stress Design Rating

↓

Calculation of inventory and Operating Strength by Using Manual(AASHTO 983. AASHTO 1994)

↓

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

↓

Calculation of Rating Factors

Fig. 2 ASD Rating Procedure

$$M_{inv} = S_{non} \times f_{inv}$$
$$M_{ope} = S_{non} \times f_{ope} \quad (4)$$

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 (AASHTO 1983 ; AASHTO 1994)

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

When the dead loads are calculated, the unit weights of materials, which are specified in current AASHTO specification (AASHTO 1992), are used. As an example, the cross section shown in Fig. 3 is used.

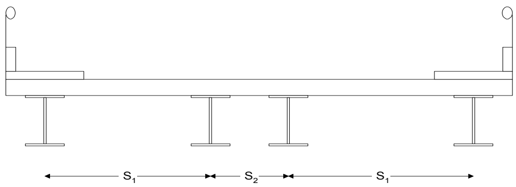

Fig. 3 Cross Section of the Bridge

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

$$Tributary\ Width = \frac{S_1 + S_2}{2} \quad (5)$$

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

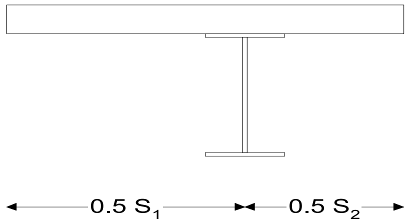

Fig. 4 Cross Section with Tributary Width

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 (AASHTO 1992).

To account the dynamic effect of moving load, there is an equation for impact factor in AASHTO specification (AASHTO 1992) and this is in Eq. (6).

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

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

2.3 Load Factor Design Rating Method

LFD load rating follows the strength design provisions in the AASHTO design specification (Standard 1992). There is a procedure for load rating using LFD.

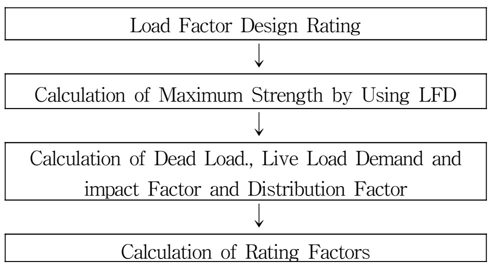

Fig. 5 LFD Rating Procedure

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	$f_y \times Z_s$
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

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

3. Live Load Data Collection

In order to collect the live load, two ways can be used. One way is to use the stationary scales. The other way is Weigh-in-Motion (WIM). In using the stationary scales, the measurements are taken at stationary scales. Trucks are entering the scales at low speed (5 - 10 MPH). For each truck the measured parameters are the date, number of axles, gross truck weight, axle loads, and axle spacings. However, the resulting data may be biased because very heavy truck drivers can avoid the stationary scales by using detours.

Table 3 Yield Stresses for Reinforcing Steel

Reinforcing steel	Yield stress, f_y, (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

The other way is Weigh-in-Motion (WIM). WIM data indicates what loads are showing up at a bridge. In this method, the strain gages are attached on the girders under the bridge. After the strain gages are attached, the bridge is opened to public traffic. So, trucks which pass on the bridge are recorded. Truck driver cannot avoid the bridges at which WIM is performed. So, the data from WIM is not biased. There is a schematic of a typical installation in Fig 6. A pair of tape-switches is placed in each lane to automatically record axle crossings. Reusable strain transducers are clamped on each girder along a line perpendicular to the axis of the bridge. The data from the tape-switches and strain signals are sent to the equipment van and recorded on the computer.

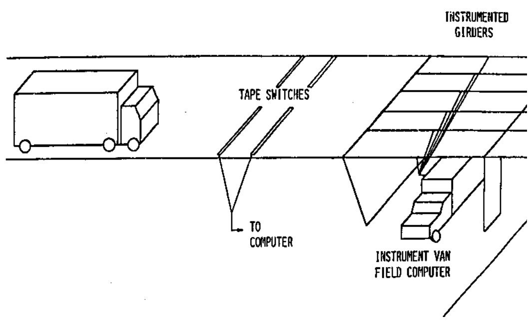

Fig. 6 General Test Setup of WIM (adapted from Moses 1985)

In this paper, WIM data from Nowak and Nassif (1991) is used to compute the probability of overload. Nowak and Nassif collected live loads from three bridges (US-23 over Huron River, US-23/M-14 over railroad, and I-94 over Jackson Road). There was no useful data on US-23. For Bridge US-23, the equipment did not work properly and a large amount of data is not reliable. 6719 trucks were measured from Bridge I-94.

The obtained trucks were run over influence lines to compute maximum moment and shears for span lengths 30 ft, 60 ft, 90 ft, and 120 ft. The results were plotted on normal probability paper for moments and shears. The vertical scale is an inverse normal distribution function and the horizontal scale is the ratio of calculated moments (or shears) from obtained trucks and the moments of bridge design load HS20.

The data on figures (Inverse normal distribution vs. the ratio of moment from obtained truck to HS20 moment) from the paper (Nowak and Nassif 1991) is replotted and shown in Fig. 7 to Fig. 10.

The solid line is the plot of WIM data and the dashed line is linear regression line. The mean and standard deviation of the ratio (live load moment to HS20 moment) for each span length are summarized in Table 4.

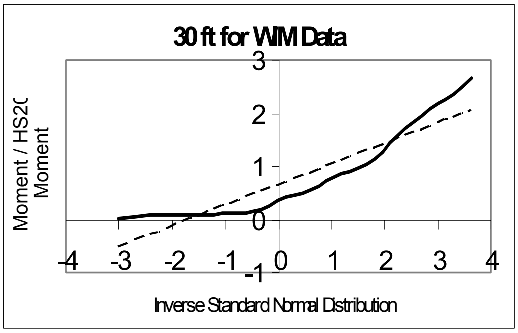

Fig. 7. Inverse Standard Normal Distribution Function of 60 ft (Adapted From Nowak 1991)

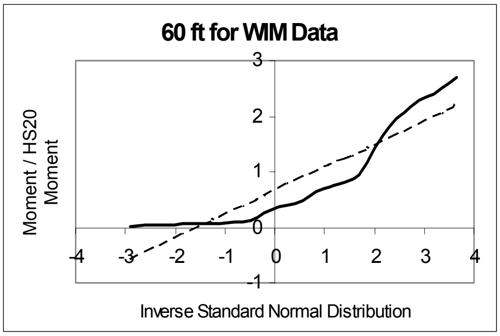

Fig. 8. Inverse Standard Normal Distribution Function of 60 ft (Adapted From Nowak 1991)

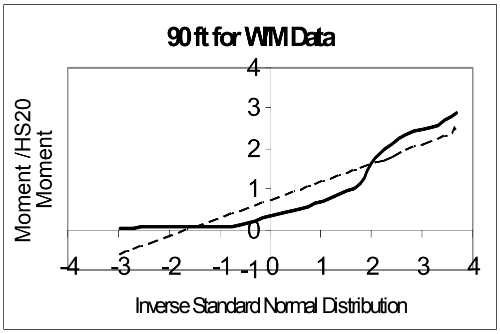

Fig. 9. Inverse Standard Normal Distribution Function of 90 ft (Adapted From Nowak 1991)

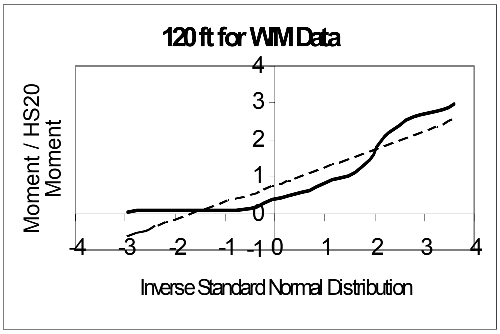

Fig. 10. Inverse Standard Normal Distribution Function of 120 ft (Adapted From Nowak 1991)

Table 4 Mean and Standard Deviation for Each Span Length

span length (ft)	Mean	Standard Deviation
30	0.676	0.387
60	0.678	0.429
90	0.744	0.454
120	0.785	0.484

The probability density function for each span length is plotted in Fig. 11 to Fig. 14.

For all spans lengths, PDF's of moment ratio are shown in Fig. 15.

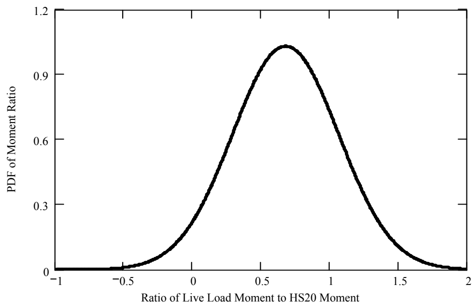

Fig. 11 PDF of Moment Ratio
(Live Load Moment to HS20 Moment) of 30 ft

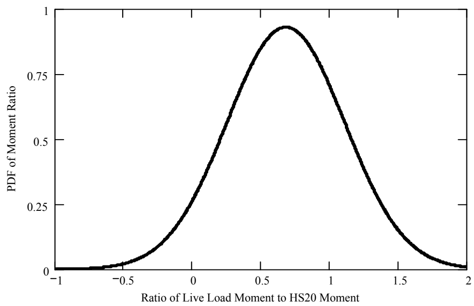

Fig. 12 PDF of Moment Ratio
(Live Load Moment to HS20 Moment) of 60 ft

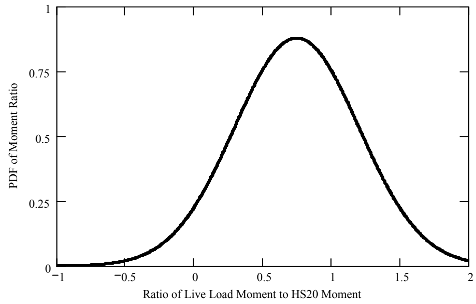

Fig. 13 PDF of Moment Ratio
(Live Load Moment to HS20 Moment) of 90 ft

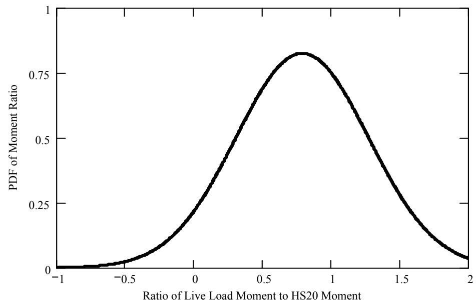

Fig. 14 PDF of Moment Ratio
(Live Load Moment to HS20 Moment) of 120 ft

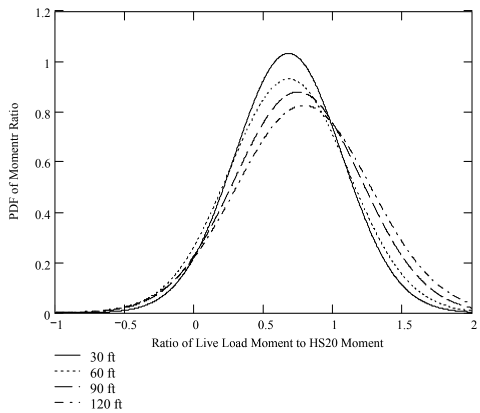

Fig. 15 PDF of Moment Ratio
(Live Load Moment to HS20 Moment) for Each Span Length

4. Bridge Load Test Data

As an example to compute the probability of overload with rating value and show the calculation procedure, it is needed to select the real load test data. Many load test results are reviewed and finally a steel girder bridge which was tested by Chajes (et al. 1997) is selected. He load tested a concrete-slab-steel girder bridge which was constructed as non-composite. The test result revealed that the bridge showed both fully composite action and unintended support restraint.

4.1 Bridge Description

The Table 5 contains information needed to rate the bridge.

Table 5 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	$W36\times170$
Girder spacing	5 ft
Thickness of slab	8.5 in

4.2 Comparison of Rating Factors

The rating factors (ASD and Test) are in Table 6 with test rating results reported in the paper (Chajes et al. 1997). The rating truck was HS20.

Table 6 Rating Factors

	ASD		Test	
	Inv	Ope	Inv	Ope
Rating Factor	0.51	1.43	1.52	3.09

where

I_{nv}=Inventory rating factor

O_{pe}=Operating rating factor

5. Probability of Overload Computation with WIM Data

The rating factor for HS20 truck is the ratio of live load moment strength to HS20 live load demand moment. The WIM data presented in section 3 is the ratio of live load moment demand to HS20 live load. By using WIM data and rating factors, it is possible to compute the probability of overload.

Because the span length of bridge from Chajes (et al. 1997) is 64 ft, the WIM data of 60 ft is used to calculate the probability of overload. Fig. 16 shows the probability of overload for AASHTO inventory rating. Because the inventory rating factor AASHTO is 0.51, the right area at 0.51 in Fig. 16 is the probability of overload. The area of the right side (Probability of Overload with AASHTO Inventory Rating Factor) is

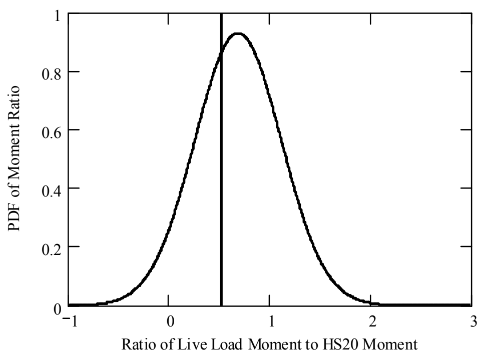

Fig. 16 Probability of Overload with AASHTO Inventory Rating Factor

$$PO_{Inv(AASHTO)} = 0.652 \quad (7)$$

For operating rating factor, the probability of overload is shown in Fig. 17.

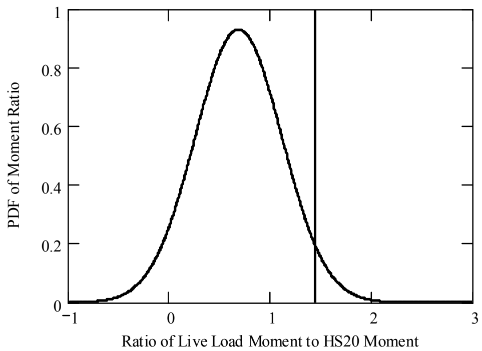

Fig. 17 Probability of Overload with AASHTO Operating Rating Factor

$$PO_{Ope(AASHTO)} = 0.04 \quad (8)$$

For test result, the probability of overload for each level is shown in Figs. 18 and 19.

$$PO_{Inv(test)} = 0.025 \quad (9)$$

$$PO_{Ope(Test)} = 9.417 \times 10^{-9} \quad (10)$$

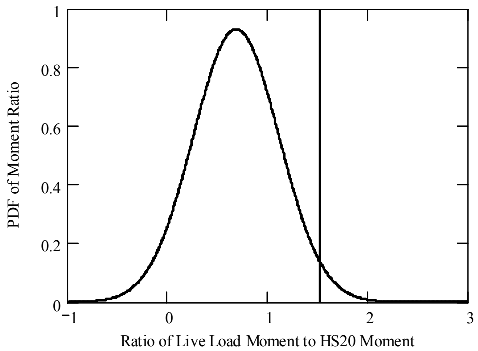

Fig. 18 Probability of Overload with Test Inventory Rating Factor

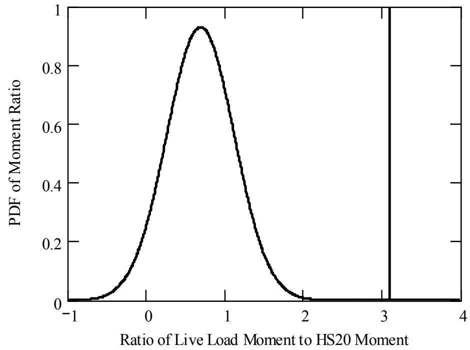

Fig. 19 Probability of Overload with Test Operating Rating Factor

Fig. 20 shows all rating factors (AASHTO Inventory, AASHTO Operating, Test Inventory, Test Operating).

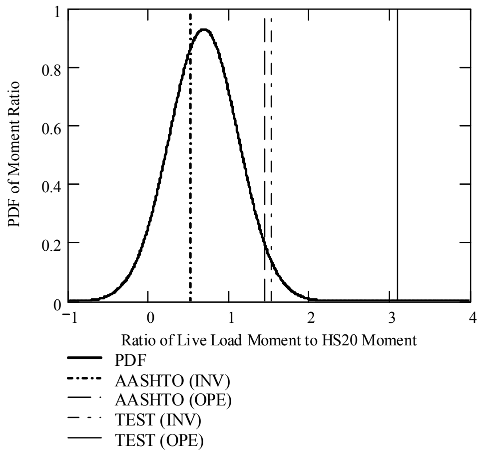

Fig. 20 Probability of Overload with All Rating Factors

6. Conclusion

In this paper, the rating procedure was explained for each rating method. In order to use probability concept using a rating factor, live load models were obtained from the literature survey.

WIM data was decided to be used since WIM data was not biased. As a new try, the probability of overload was calculated using both rating factors and a live load model. By using the live load model, it was possible to make the connection between the probability concept and rating factors, and compute the probability of overload.

Using the methodology presented in this paper, it is possible to uniformly classify and rank the bridges based on rating factors and the probability of overload. And, it is possible to predict the probability of overload and number of trucks by using extremal distribution concepts.

Reference

1. American Association of State Highway Transportation Officials, Standard specification for highway bridges, 1992. Washington, D.C.
2. American Association of State Highway Transportation Officials, Manual for maintenance inspection of bridges, 1983. Washington, D.C.
3. FHWA, The status of nation's highway and bridge: condition and performance and highway bridge replacement and rehabilitation program, U.S Department of Transportation, 1989. Washington, D.C., P 173.
4. American Association of State Highway Transportation Officials, Manual for Condition Evaluation of bridges Officials, 1994 Washington, D.C.
5. Chajes, M. J., Mertz, D. R., and Commander, B., Experimental load rating of a posted bridge, ASCE Journal of Bridge Engineering, 1997. Vol 2 (n1), pp 1-10.
6. Moses, F., Calibration of Load Factors for LRFR Bridge Evaluation, National Cooperative Highway Research Program Report 454, 2001. p 50.
7. Nowak, A. S. and Nassif, H., Effect of truck loading on bridges, Michigan Department of Transportation, University of Michigan at Ann Arbor, 1991. p 173.

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