신뢰성응용연구 신뢰성기술
제13권, 제4호. pp.241 – 252, 2013

강거더 교량의 신뢰성해석을 위한 저항모델 개발

엄준식

상지대학교 건설시스템공학과

Resistance Model for Reliability Analysis of Existing Steel Girder Bridges

Jun Sik Eom

Dept. of Civil Engineering, Sangji Univ.

Abstract

Because of financial and safety concerns, there are needs for more accurate prediction of bridge behavior. Underestimation of the bridge load carrying capacity can have serious economic consequences, as deficient bridges must be repaired or rehabilitated. Therefore, the knowledge of the actual bridge behavior under live load may lead to a more realistic calculation of the load carrying capacity and eventually this may allow for more bridges to remain in service with or without minor repairs. The presented research is focused on the reliability evaluation of the actual load carrying capacity of existing bridges based on the field testing. Seventeen existing bridges were tested under truck load to confirm their adequacy of reliability. The actual response of existing bridge structures under live load is measured. Reliability analysis is performed on the selected representative bridges designed in accordance with AASHTO codes for bridge component (girder). Bridges are first evaluated based on the code specified values and design resistance. However, after the field testing program, it is possible to apply the experimental results into the bridge reliability evaluation procedures. Therefore, the actual response of bridge structures, including unintentional composite action, partial fixity of supports, and contribution of nonstructural members are considered in the bridge reliability evaluation. The girder distribution factors obtained from the tests are also applied in the reliability calculation. The results indicate that the reliability indices of selected bridges can be significantly increased by reducing uncertainties without sacrificing the safety of structures, by including the result of field measurement data into calculation.

Keywords : Bridge Reliability, Field Testing, Resistance Model

논문접수일 : 2013년 10월 30일 논문수정일 : 2013년 12월 11일 게재확정일 : 2013년 12월 13일

1. Introduction

In the United States, there are currently about 600,000 bridges erected on public highways. Of these, about 30 percent are classified as structurally or functionally deficient by Federal Highway Administration (2009). In addition to the deterioration of the bridge structures, the actual live loads on bridges have also increased considerably. For example, in 1950 the maximum observed gross vehicle weight (GVW) of a truck was only about 500 kN in the state of Michigan (Laman (1995)). However, 45 years later during weigh-in-motion studies on several highways in Michigan, the maximum GVW of 1,110 kN was recorded (Laman (1995)). The deficient bridges should be posted for limited traffic, repaired, or replaced. The disposition of bridges involves clear economical problems. To avoid high cost of repair or replacement of existing bridges, it is important to evaluate the present load carrying capacity of the bridges and any further changes in the capacity in the applicable time span.

Traditionally, bridges have been designed in deterministic methods. The deterministic approach assumes that the resistance and load are known and then relies on a prescribed safety factor to ensure that the resistance is sufficiently greater than the load. Approximate analysis has led considerable overestimation of load effects because of the simplified and idealized assumptions. Furthermore, most parameters in resistance and loads are random variables. However, the current deterministic analysis is based on the much lower strength than the mean of the actual material strength. In addition, the results of field measurement are again stochastic. Therefore, probabilistic methods allow better design of new bridges as well as evaluation of existing bridges than deterministic analysis. Kwon et al. (2009) mentioned that civil infrastructures are directly exposed to harsh natural environments. This can cause the structural corrosion that can drastically reduce the service life of the structures. Therefore, actual bridge test data can be used to improve the accuracy of load and resistance models by reducing the uncertainty caused by the idealized assumptions used in analysis.

In this paper, it is aimed to develop the realistic resistance model for reliability analysis of currently existing bridges by including the field test data.

Measured strains are compared to the calculated values, and the reasons for small measured strains are identified. Also, efforts have been directed to more realistic values of girder distribution factors (GDF) for the reliability analysis of existing bridges.

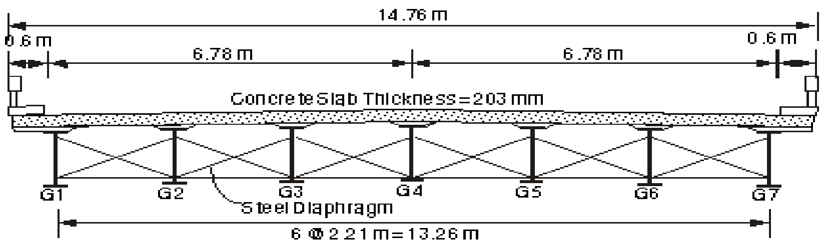
2. Resistance Model

2.1 Selected Bridges for Field Testing and Reliability Analysis

This study is focused on steel girder bridges with simply supported spans from 10 to 45 m. More than a hundred bridges were inspected to check their feasibility for load test. The parameters considered include accessibility for testing equipment, traffic volume (Average Daily Traffic, ADT, less than 15,000), skewness (no more than 30o), and existence of non-typical features. Finally, 17 bridges were selected for this study. All the selected bridges carry two lanes of traffic. The details are shown in the <Table 1>. A typical cross-section of selected bridges is illustrated in <Figure 1>.

<Table 1> Selected Bridges

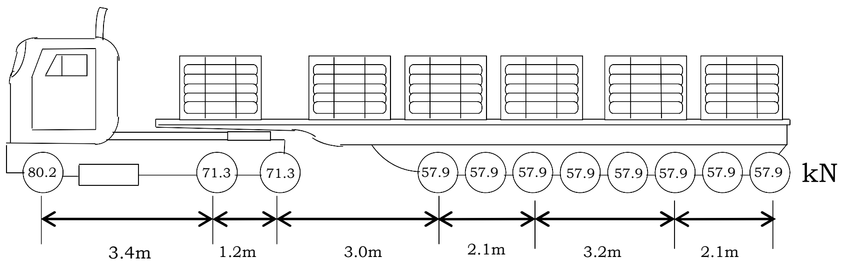
No.	Span (m)	No of Girders	Girder spacing (m)	Year Con.	Skew	ADT
1	9.9	12	1.36	1922	10	5,000
2	10.6	10	1.4	1948	15	3,300
3	10.6	9	1.57	1949	0	4,000
4	11.7	10	1.42	1929	0	4,900
5	13.7	10	1.32	1935	30	970
6	13.7	9	1.46	1939	20	12,000
7	15.2	9	1.57	1947	0	2,500
8	16.7	8	1.79	1953	10	4,400
9	16.8	11	1.44	1932	0	13,000
10	18.8	6	1.9	1965	11	3,500
11	20.4	7	1.44	1933	0	9,600
12	21.3	11	1.37	1936	0	5,600
13	22.8	9	1.22	1928	0	3,500
14	26.4	10	1.37	1932	0	4,200
15	29.8	5	2.82	1970	0	800
16	38.4	7	2.21	1972	0	2,000
17	42.6	6	2.85	1986	0	12,000



<Figure 1> Typical Cross-section of Selected Bridges (Bridge 16)

2.2 Loads Used in Field Tesing

In Michigan, the maximum mid-span moment in medium span bridges is caused by 11-axle trucks, with gross vehicle weight (GVW) up to 730 kN depending on axle configuration. This is almost twice the allowable legal load in other states. Most states in U.S. allow a maximum GVW of 356 kN with up to 5 axles per vehicle. The vehicles used in these tests were fully loaded 11-axle trucks, with a length from front to rear axle of up to 18 m. A typical side-view of test trucks is shown in <Figure 2>. Strain data were taken from the bottom-flanges of girders at midspan. Strain data were obtained under passes of 11-axle trucks with known weight and axle configuration. Trucks were run at crawling speed and also at high speed to measure the dynamic effect.



<Figure 2> Typical Side-View of Trucks used in the Analysis

2.3 Resistance Model Based on Field Testing

The main objective of the field testing program was to verify the analytical results and to obtain more realistic bridge resistance model. The calculated strain, εc, at the bottom flange of the girder under test loading can be expressed in terms of the moment based on the linear elastic theory as follows:

$$\varepsilon_C = \frac{M_T}{SE} \tag{1}$$

where MT is the maximum moment due to the applied load during the test, S is section modulus, and E is modulus of elasticity.

For all bridges tested in this study, the measured strain obtained from field tests, εT was much smaller than analytically predicted strain, εc, calculated even with the girder distribution factors determined in the tests. These results indicate that the actual load carrying capacity of individual girders is much higher than analytically predicted. Based on Eq. (1), Lichtenstein (1993) used a term called “apparent section modulus”, or Sapparent, calculated based on the measured moment and defined as:

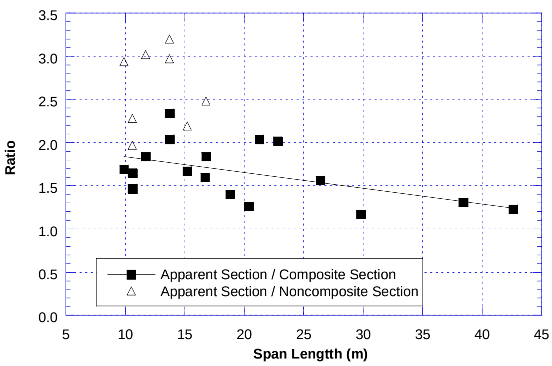
$$S_{apparent} = \frac{M_T}{\varepsilon_T E} \tag{2}$$

where εT is the maximum member strain measured during the load test.
For each tested bridge, the section modulus is calculated and shown in <Table 2>. For non-composite bridges, both composite section modulus and non-composite section modulus are calculated, since these bridges can be considered as composite, as explained in the introduction. The analytical strains, εc, are calculated based on the truck loads applied and the measured girder distribution factors, and they are compared with measured strains, εT. The calculated values of Sapparent, are shown in <Table 2>. As expected, there are considerable differences between the theoretical section moduli and apparent section moduli.

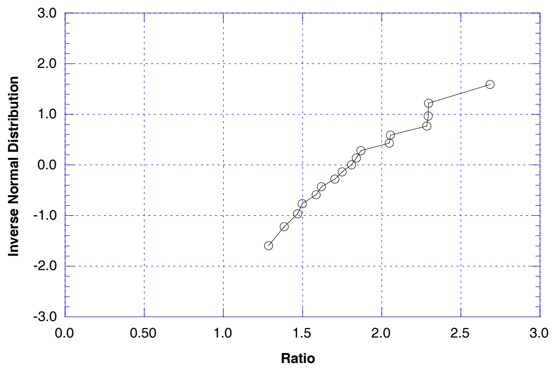
<Table 2> Section Moduli Calculated for Tested Bridges

Bridge No	Section Modulus S (x106mm3)		ɱc (x10-6)	ɱT (x10-6)	Sapparent (x106mm3)
	Comp.	Non-comp.			
1	6.3	4.5	386	102	16.9
2	11.0	7.9	245	86	22.5
3	5.2	2.9	240	70	9.8
4	7.3	4.8	371	107	16.7
5	6.9	4.0	310	88	14.2
6	12.5		263	162	20.3
7	14.3		310	207	21.4
8	17.5		220	96	40.1
9	24.5		511	348	36.1
10	14.5		276	120	33.2
11	21.0		196	115	35.8
12	9.0	6.6	239	130	16.5
13	4.5	3.1	357	137	8.2
14	10.9		227	130	19.0
15	46.1		208	150	63.9
16	6.2	4.4	256	115	9.8
17	48.2		249	194	62.0

Bridges No.1, 2, 3, 4, 5, 12, 13, and 16 were designed as noncomposite superstructures. This means that to estimate the maximum stresses, only steel girders (without a concrete slab) were taken into account. However, field tests by Schultz et al. (1995) proved that even with no shear connectors, there is still bonding between the concrete and steel at the interface and it is usually enough to resist shear forces induced by dead and live loads. Therefore, it is assumed in <Figures 3> and <Figure 4> that the noncomposite girders behave as composite beams. <Figure 3> shows the ratio of the apparent section modulus to composite section modulus regardless of whether the bridge is designed as noncomposite or composite, and the test results are compared with composite section modulus on normal probability paper in <Figure 4>. The mean value of the ratio is 1.70, and the coefficient of variation (C.O.V.) is 0.216. For comparison, the ratios of apparent section modulus vs. noncomposite section are also shown in the figure for those bridges designed as noncomposite,. It is observed that the ratio decreases as the span length increases, because longer spans are less affected by the partial fixity of supports.



<Figure 3> Ratio of Apparent Section Modulus to Calculated Section Modulus of Tested Bridges, compared with Span Length



<Figure 4> Ratio of the Apparent Section Modulus to the Composite Section Modulus of Tested Bridges plotted on the Inverse Normal Probability Paper.

2.4 Girder Distribution of Loads for Bridge Resistance Model

To determine the effect of live load acting on each girder, a girder distribution factor (GDF) has to be determined. In AASHTO Standard (2002), GDF's are defined as a function of girder spacing. GDF's specified in AASHTO LRFD (2013) consider other parameters, such as span length, bridge skew, in addition to girder spacing, which is the only parameter in AASHTO Standard (2002).

For the bending moment in an interior girders, the AASHTO Standard (2002) specifies GDF's as follows. For steel girder and prestressed concrete girder bridges, with more than one lane, GDF is:

$$GDF = \frac{S'}{3.36} \tag{3}$$

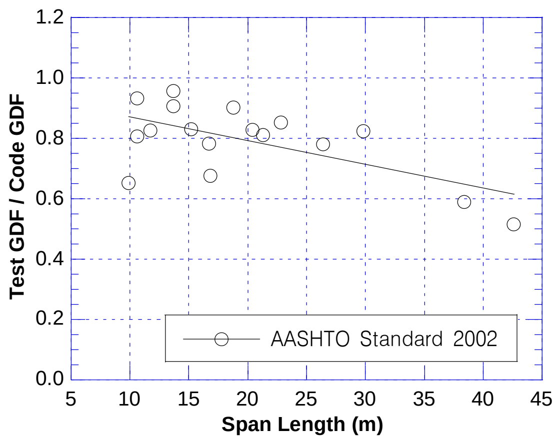
where S′ is girder spacing (m).

The AASHTO LRFD Code (2013) specifies GDF as a function of girder spacing, span length, stiffness parameters, and bridge skew. For bridges with skew less than 30 degrees, the GDF's are specified as follows. For the bending moment in interior girders with more than one lane loading, the GDF is:

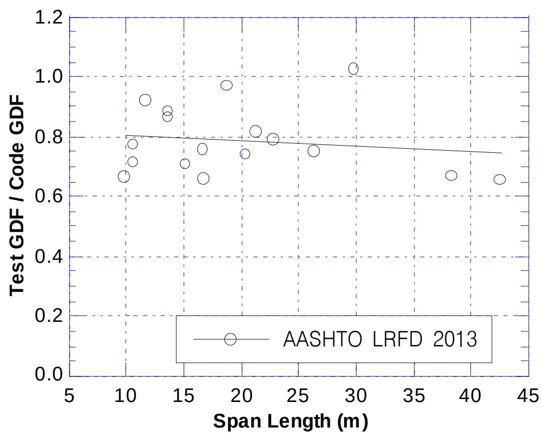
$$GDF = 0.075 + \left(\frac{S'}{2900}\right)^{0.6}\left(\frac{S'}{L}\right)^{0.2}\left(\frac{K_g}{Lt_s^3}\right)^{0.1} \tag{4}$$

where S′ is girder spacing (mm); L is span length (mm); Kg is n (I+Aeg2); ts is thickness of concrete slab (mm); n is modular ratio for the girder and slab materials; I is moment of inertia of the girder (mm4); A is cross section area of the girder (mm2); and eg is distance between the centers of gravity of the girder and slab (mm).

<Figure 5> and <Figure 6> compare the GDF's obtained from the tests with those specified in the codes. In <Figure 5>, test values are compared with those in AASHTO Standard (2002), and in <Figure 6> with AASHTO LRFD (2013).



<Figure 5> Ratio of Test GDF to AASHTO Standard GDF (2002), with linear interpolation



<Figure 6> Ratio of Test GDF to AASHTO LRFD GDF (2013), with linear interpolation

In <Figure 5>, there is a clear trend that the AASHTO Standard values become conservative for bridges with longer span. spacing. However, AASHTO LRFD GDF's do not differ significantly depending on the span length, as shown in <Figure 3>. However, the scatter is still very large. For each case, the mean value, standard deviation, and coefficient of variation are calculated and shown in <Table 3>. As expected, AASHTO Standard (2002) GDF's have the highest variation. The AASHTO LRFD GDF's (2013) also have high level of variation even though the formulas are very complicated compared to the AASHTO Standard (2002).

In <Table 3>, it is also shown that the code specified GDF values are very conservative. When the code values are compared with test, test results is only about 80 percent of what codes specifies.

<Table 3> Mean and C.O.V. for the Ratios of Test GDF's to Code Specified GDF's

GDF Ratio	Mean Value	Coefficient of Variation
Test / AASHTO Standard (2002)	0.79	0.152
Test / AASHTO LRFD (2013)	0.78	0.142

3. Reliability Analysis

3.1 Iterative Numerical Procedure used for reliability analysis

An iterative procedure was developed by Rackwitz and Fiessler (1978), based on normal approximations of non-normal distributions at the design point. The design point is the point of maximum probability on the failure boundary (limit state function).

This method requires knowledge of the distributions of all involved random variables. The mathematical representation of the failure boundary is the limit state function equal to zero, g = R-Q = 0. The design point, denoted by (R*, Q*), is located on the failure boundary, so R*=Q*.

Let F_R and f_R be the cumulative distribution function (CDF) and the probability density function (PDF) of R, respectively. Similarly, F_Q and f_Q are the CDF and PDF of Q, respectively. Also, let $R^{'}$ and $Q^{'}$ be the mean value of the approximating normal distribution of R* and Q*, respectively. The method starts by guessing an initial value of (R*, Q*). Next, F_R and F_Q are approximated at the design point by normal distributions $F_{R'}$ and $F_{Q'}$ such that

$$F_{R'}(R^*) = F_R(R^*) \tag{5}$$

$$F_{Q'}(Q^*) = F_Q(Q^*) \tag{6}$$

$$f_{R'}(R^*) = f_R(R^*) \tag{7}$$

$$f_{Q'}(Q^*) = f_Q(Q^*) \tag{8}$$

The standard deviations of R' and Q' are computed from:

$$\sigma_{R'} = \frac{\phi\{\Phi^{-1}[F_R(R^*)]\}}{f_R(R^*)} \tag{9}$$

$$\sigma_{Q'} = \frac{\phi\{\Phi^{-1}[F_Q(Q^*)]\}}{f_Q(Q^*)} \tag{10}$$

where ϕ is PDF of the standard normal random variable, and Φ is CDF of the standard normal random variable.

The means of R' and Q' can now be evaluated using the following expression:

$$\mu_{R'} = R^* - \sigma_{R'}\Phi^{-1}[F_R(R^*)] \tag{11}$$

$$\mu_{Q'} = Q^* - \sigma_{Q'}\Phi^{-1}[F_Q(Q^*)] \tag{12}$$

The reliability index is

$$\beta = \frac{\mu_{R'} - \mu_{Q'}}{\sqrt{\sigma_{R'}^2 + \sigma_{Q'}^2}} \tag{13}$$

Next, a new design point can be calculated from the following equation

$$R^* = Q^* = \mu_{R'} - \beta\frac{\sigma_{R'}^2}{\sqrt{\sigma_{R'}^2 + \sigma_{Q'}^2}} \tag{14}$$

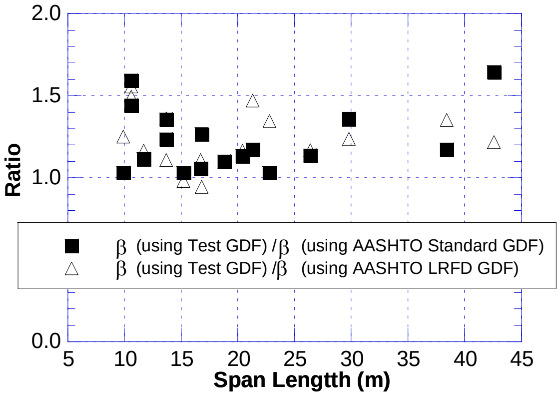
Then the second iteration begins: the approximating normal distributions are found for F_R and F_Q at the new design point. The reliability index is calculated by using Eq. (13) and the next design point is found from Eq. (14). Calculations are continued until R* and Q* do not change in consecutive iterations. Rackwitz-Fiessler method (1978) is used in the study for the calculation of the reliability indices. The assumptions and parameters used in the study are explained in the following sections.

3.2 Reliability Analysis Results

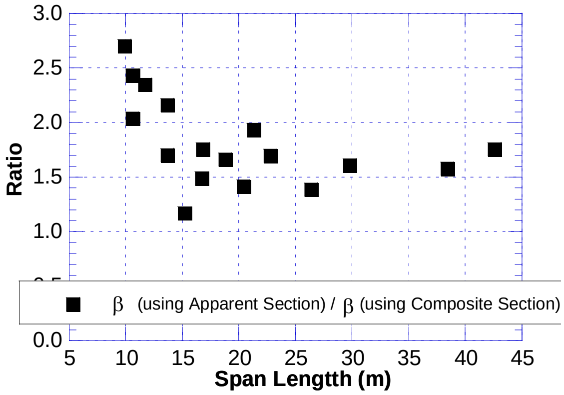
Based on the considered GDF's and section moduli, reliability analysis is performed and the results are presented in <Table 4>. In the table, the reliability indices are calculated using AASHTO Standard GDF, AASHTO LRFD GDF. The values are compared with the reliability indices calculated using the test results. In each case, reliability indices are calculated based on noncomposite, composite, and apparent section modulus. <Figure 7> presents the increase of reliability indice depending on the considered girder distribution factors. <Figure 8> shows the level of reliability index increase when the apparent section modulus is used. The increase of reliability is greater for shorter span bridges. This is because short span bridges are more affected by factors not considered in analysis, such as partial support fixity and structural contribution of nonstructural members.

<Table 4> Reliability Indices For Tested Bridges Based on Apparent Section Modulus

Bridge ID	Reliability Index β						
	AASHTO Standard (2002) GDF		AASHTO LRFD (2013) GDF		Test GDF		Test GDF
	Non Composite	Composite	Non Composite	Composite	Non Composite	Composite	Apparent Section
1	3.21	4.64	2.65	4.06	3.31	4.68	12.55
2	4	5.46	3.86	5.31	5.76	7.23	13.28
3	3.34	7.02	3.42	7.1	5.32	9.04	14.31
4	3.21	5.21	3.07	5.06	3.58	5.53	12.23
5	2.36	5.75	2.89	6.32	3.2	6.54	12.43
6		7.24		6.56		8.94	12.32
7		7.94		8.34		8.19	9.28
8		9.39		8.95		9.91	13.98
9		3.19		4.27		4.04	5.6
10		8.27		8.29		9.09	13.76
11		7.89		7.66		8.92	11.16
12	4.82	6.08	3.84	5.08	5.65	6.89	11.75
13	5.1	6.77	3.91	5.52	5.26	6.86	11.49
14		8.45		8.21		9.6	11.74
15		6.06		6.65		8.23	9.76
16	5.67	8.11	4.92	7.26	6.65	9.07	12.8
17		3.93		5.31		6.46	6.91



<Figure 7> Comparison of Reliability Indices based on AASHTO GDF's



<Figure 8> Comparison of Reliability Indices based on different section moduli

4. Conclusion

Because of financial and safety concerns, there are needs for more accurate prediction of bridge behavior. Underestimation of the bridge load carrying capacity can have serious economic consequences, as deficient bridges must be repaired or rehabilitated. The presented research is focused on realistic reliability evaluation of the actual load carrying capacity of existing bridges based on the field testing. A bridge can be evaluated using either deterministic rating calculation or reliability analysis. Bridges are usually evaluated based on the code specified values and design resistance. However, after the field testing program, it is possible to apply the experimental results into the bridge evaluation procedures. Therefore, the actual response of bridge structures, including unintentional composite action, partial fixity of supports, and contribution of nonstructural members are considered in the bridge evaluation. It is shown that the reliability indices of selected bridges can be significantly increased by reducing uncertainties without sacrificing the safety of structures, by including the result of field measurement data into calculation.

Acknowledgements

The presented study has been supported by Sangji University Sabbatical Support Program.

References

[1] AASHTO LRFD Bridge Design Specifications, American Association of State Highway and Transportation Officials, Washington, D.C., 2013.

[2] AASHTO Standard Specifications for Highway Bridges. American Association of State Highway and Transportation Officials, Washington, D.C., 2002.

[3] FHWA bridge programs material type of structure by state, Federal Highway Administration, 2009.

[4] Y.I. Kwon, S.J. Kim and H.W. Lee (2009), "Service Life Prediction of CFRP bar for Concrete Reinforcement based on Accelerated degradation tests," J. Applied Reliability, Vol.9, No.2, pp71-80.

[5] J. Laman (1995), "Fatigue Load Models for Girder Bridges," Ph.D. dissertation, Department of Civil Engineering, University of Michigan, Ann Arbor, Michigan.

[6] A.G. Lichtenstein (1993), “Bridge Rating Through Nondestructive Load Testing,” NCHRP report No. 12-28(13)A, National Cooperative Highway Research Program.

[7] R. Rackwitz, and B. Fiessler (1978), “Structural Reliability under Combined Random Load Sequences,” Computer and Structures, Vol. 9, pp. 489-494..

[8] J.L. Schultz, B. Commander, G.G. Goble, and D.M. Frangpol (1995), “Efficient Field Testing and Load Rating of Short and Medium Span Bridges,” Structural Engineering Review, Vol. 7, No 3, pp. 181-194.