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An Experience of Practical Reliability-Based Safety Assessment and Capacity Rating

By Hyo-Nam Cho*, Hyun-Ho Choi**, Jung-Ho Kim***, and Young-Min Choi****

Abstract

Nowadays, reliability-based methods have been widely used for the safety and capacity rating of deteriorated and/or damaged bridges. This paper is intended to suggest practical reliability-based assessment models and methods for the safety assessment and capacity rating of various kinds of actual existing bridges. And the application of recently developed a new approach, called equivalent system-strength, for reliability-based capacity rating is illustrated using the data obtained from actual bridges. This paper also summarizes various approaches to reliability-based safety assessment and capacity rating, and investigates their application to various existing aged bridges. It will be systematically shown that the proposed bridge reliability models and methods could be effectively used in practice for the safety assessment and capacity rating of existing old bridges in conjunction with static and dynamic field load tests, nondestructive tests, and visual inspections.

Keywords: *existing bridges, reliability methods, safety assessment, capacity rating*

1. Introduction

In many countries it is reported that a number of bridges are seriously damaged mainly due to excessively overloaded heavy freight vehicles and lack of proper maintenance. In order to ensure the safe and reliable operation of bridges a reasonable and cost effective maintenance technology is required. The optimal decision on bridge maintenance and rehabilitation using the integrity assessment involves tremendous economic and safety implication, and heavily depends upon the assessment of reserve safety and load carrying capacity. The reliability-based concept of structural maintenance and rehabilitation for the possible extension of the service life and assuring the structural safety of existing bridges is very attractive. Much progress in the application of reliability methods to the safety assessment has been made following the application to limit state design code calibrations. Unlike design code calibration, however, safety assessment and rating are more complex decision processes that require the assessment of site-specific data. It is no doubt that realistic safety assessment of existing bridges is a challenging task.

For the realistic prediction of reserve safety and capacity of redundant bridges it may be necessary to assess the safety and capacity rating of actual large bridges based on system performance and system reliability. However, it is still extremely difficult to evaluate realistic reserve safety and load carrying capacity, especially when bridges are highly redundant and deteriorated or damaged to a significant degree.

This paper presents a practical system reliability-based approach for the safety assessment and load carrying capacity evaluation of various bridges, and emphasizes the model and approach of the system reliability-based safety assessment and rating compared to those of the element. These differences are illustrated through numerical calculations in terms of the safety index and capacity rating of the example bridges.

Moreover, since the precise prediction of reserved carrying capacity of bridge as a system is extremely difficult, this paper demonstrates recently developed approach called equivalent system-strength (Cho *et al*., 1993) for the evaluation of reserved system carrying capacity of bridges, which may be defined as a bridge system-strength corresponding to the system reliability of the bridge.

2. Safety Assessment and Rating Methods for Existing Bridge Structures

2.1. Conventional Approaches

Since many existing bridge structures are complex, it is a challenging task to accurately assess the current condition of a particular bridge. Researches on this subject have been intensified in recent years because a number of new technologies are becoming available. Many experienced researchers or expert engineers are capable of assessing the condition of existing structures using available documents, data from visual inspection, field and laboratory testing, health or condition monitoring, and computational structural analysis. In spite of the fact that the sensory and measurement technologies have been highly developed, rational strategies and established methods for structural safety assessment and rating from measured data are still not available in practice, especially in the case of complex and large bridge systems. Moreover, at the time of assessing a bridge, there are many uncertainties, particularly regarding materials, environmental effects and loading. However, in general, the conventional safety assessment and rating methods have been developed mainly based on design rules, which is a process of trial and error through the years. The volume of specifications, standard, and codes is increasing, with only slow improvement in the assessment application. Therefore, so far conventional approaches which are largely based on the WSR (Working Stress Rating) or LFR (Load Factor Rating) criterions have failed in the assessment and rating of actual bridge systems because

*Professor, Department of Civil and Environmental Engineering, Hanyang University, Korea (E-mail: ryfid@hanyang.ac.kr)
**Doctor, Department of Civil and Environmental Engineering, Hanyang University, Korea (E-mail: padre@ihanyang.ac.kr)
***Senior Researcher, Korea Institute of Construction Technology, Korea (E-mail: jhkim@kict.re.kr)
****Research Professor, Ph.D. Department of Civil and Environmental Engineering, Hanyang University, Korea (E-mail: choijaram@hanafos.com)

they have been usually performed without properly considering various uncertainties involved in environmental loading effects, degree of deterioration or damage of existing bridge structures. Namely, the conventional capacity rating of a bridge does not systematically take into account any information on the uncertainties of strength and loading, the degree of damage or deterioration and the characteristics of actual response to the specific bridge to be rated.

2.2. Reliability-based Approaches

Due to rapid developments in the theory of structural reliability for the past two decades, the reliability-based methods are becoming a powerful tool for analysis, design, and assessment of engineering systems in practice. However, it is still not so advanced as it would be desirable and also needed. Nevertheless, the methodology has disseminated into various areas (Schuëller, 1997; Sundararajan *et al*., 1995): for example, reliability analysis of special type structures such as nuclear power plant, pressure vessels and piping, aircraft structures, offshore structures, etc.; codified design (EUROCODE, LRFD, etc.) of general type structures; and optimum structural design. And it is also used in the reliability assessment of existing structures (Cho *et al*., 1993; Farhey *et al*., 1997), re-qualification procedures, system identification and structural control, stochastic dynamics, life prediction, and risk studies, etc.

In the safety evaluation and capacity rating of bridges system performance and system reliability are very important for the realistic prediction of residual carrying capacity of highly redundant bridge system. Recently, some practical system reliability models and methods for girder bridges have been suggested with the emphasis on the system-level reliability rather than the element level (Nowak, 1995; Tabsh and Nowak, 1991). Also some practical approaches for reliability-based safety assessment and capacity rating have been made by (Cho, 1996; Cho *et al*., 1998; Cho *et al*., 1997; Cho *et al*., 2001).

Based on the experiences in the various types of bridges, it may be argued that the available conventional methods for safety assessment and capacity rating of bridges provide in most cases nothing but notional results which are no better than an expert's guess. As expected, the reliability-based approach renders rational and realistic results, which are consistent in most cases with the actual condition of the bridges in question and with the results of visual inspection, nondestructive tests (NDTs) and field load tests.

2.3. Reliability Assessment of Actual Bridges

Table 1 shows a summary of various types of actual bridges, that had been assessed. As listed in the Table 1, most of the bridge assessment projects were carried out based on an extensive array of field investigations such as static and dynamic load tests, NDTs as well as close-up visual inspections. Moreover, the safety and carrying capacity of the bridges have been evaluated by the methods based on both the reliability-based and conventional approaches for comparison. For each bridge assessment, therefore, a comparison was made in order to demonstrate the rationality and applicability of the reliability-based approach.

3. Reliability-based Safety Assessment Models and Methods

Reliability methods can be effectively used for evaluating the condition of existing bridge structures. The study of structural reliability is concerned with the calculation and prediction of the probability of limit state violation for an engineered structural system at any stage during its life. In particular, the study of structural safety is concerned with the violation of the ultimate limit state for the structures. Limit state models have to be defined for each limit state. This might include limit state for bending strength, shear strength, local buckling, lateral buckling, fatigue, corrosion, etc. Various effective reliability methods can be applied to evaluate the structural reliability for these failure limit states.

3.1. Limit State Models

3.1.1. Strength Limit State Models for Various Types of Bridges

Based on the bridge assessment with actual applications as shown in Table 1, strength limit state models for various types of bridges are briefly presented. These models may be used in practice as effective tools for the reliability and reliability-based capacity rating of any type of bridges with the various dominant limit state failures. Moreover, the proposed limit states for various failure modes can be used to predict more realistic reserve safety and capacity of deteriorated and/or damaged bridges.

(1) Strength Limit State Model for RC Box Girder Bridges:

The linear strength limit state model at the element level for RC box girder bridges as shown in Eq. (1) is dominantly used for the basic bending and shear when single dominant failure mode is considered.

$$g(\cdot) = R - \sum Q_i \tag{1}$$

where R is true resistance; and Q_i is i-th load effect.

A realistic safety assessment or rating of existing bridges requires a rational determination of the degree of deterioration or damage. Therefore, the true resistance R in Eq. (1) must incorporates some random variables to reflect such deterioration/damage and the

Table 1. Statistics of the Bridge Assessment Conducted by the Author

Assessment	RC T-Beam	RC Slab	PSC Beam	RC Box Girder	Segmental PSC Box Girder	Steel Truss	Steel I-Beam & Plate Girder	Steel Box Girder	Steel Cable-Stayed
Diagnosis	8	7	11	2	3	2	15	6	1
Inspection	7	7	10	2	2	2	14	6	1
NDT	7	7	10	2	2	2	13	6	1
Static/Dynamic Load Test	6	6	4	2	2	1	12	4	1
Rating	6	6	4	2	2	1	12	4	1
Element Reliability	7	5	4	2	3	1	12	4	1
System Reliability	1	0	1	1	0	0	5	2	1

underlying uncertainties. SI techniques may be used for the precise evaluation of damage factor, D_F. However, in lieu of such elaborate techniques as SI, the resistance of the deteriorated or damaged members is made to be estimated on the basis of visual inspection and/or various in-situ nondestructive tests, supplemented with the engineering judgment in case of short span bridges. Then, the true resistance R such as moment may be modeled as follows:

$$R = M_R = M_n N_M D_F \tag{2}$$

where M_R is nominal moment strength specified in the code; N_M is the correction factor adjusting any bias and incorporating the uncertainties involved in the assessment of M_n and D_F (=MFPD), in which, M is material strength uncertainties; F is fabrication and constructions uncertainties; P is prediction and modeling uncertainties; D is the uncertainties involved in the assessment of damages and/or deterioration; and D_F is damage factor, which is the ratio of current stiffness K_D to the intact stiffness K_I i.e., K_D/K_I, or approximately that of the fundamental natural frequency of the damaged ω_D to the intact one ω_I i.e., ω_D^2/ω_I^2 in case of simple bridges.

It may be noted that the nominal moment strength M_n specified in the code becomes either yield moment M_y in case of tension failure or buckling moment M_{cr} in case of compression failure.

Also, true applied moments may be expressed in terms of respective random variables as follows:

$$M_D = m_D D_n N_D \tag{3a}$$
$$M_L = m_L L_n K N_L \tag{3b}$$

where m_D, m_L are the influence coefficients of moment for dead and truck loads, respectively; D_n, L_n are the nominal dead and truck loads, respectively; K is the response ratio (=K_s(1+I)); and N_D, N_L are the correction factors(=AQ), in which, K_s is the ratio of the measured stress to the calculated stress; I is the impact factor; A is response random variables corresponding to dead or truck load; and Q is random variable representing the uncertainties involved in dead or live loads.

(2) Strength Limit State Model for PSC Girder Railway Bridges:

Similarly, for PSC girder railway bridges the same model at the element level as shown in Eq. (1) can be used for the limit state of bending, shear, and torsion when single failure mode is considered.

However, the CEB model code (Thürlimann, 1979a, 1979b) requires 3-mode interaction equations for shear and torsional design of concrete bridges derived from space truss analogy for members subjected to bending, shear, and torsion. When the strength limit state in terms of the interactive combined load effects has to be considered, a nonlinear limit state model expressed as an implicit function may be suggested as follows (Cho *et al*., 1998):

Mode 1:

$$g(\cdot) = 1 - \left[\frac{F_{yu}}{F_{yl}}\left\{\rho_B\left(\frac{T_D+T_L}{T_R}\right)^2 + \left(\frac{V_D+V_L}{V_R}\right)^2\right\} + \left(\frac{M_D+M_L}{M_R}\right)\right] \tag{4a}$$

Mode 2:

$$g(\cdot) = 1 - \left[\rho_M\left(\frac{T_D+T_L}{T_R}\right)^2 + \left(\frac{V_D+V_L}{V_R}\right)^2 - \frac{F_{yl}}{F_{yu}}\left(\frac{M_D+M_L}{M_R}\right)\right] \tag{4b}$$

Mode 3:

$$g(\cdot) = \frac{1}{2}\left(\frac{F_{yl}}{F_{yu}} + 1\right) - \left[\rho_M\left(\frac{T_D+T_L}{T_R}\right)^2 + 2\left(\frac{T_D+T_L}{T_R}\right)\left(\frac{V_D+V_L}{V_R}\right)\sqrt{\frac{2h}{u}} + \left(\frac{V_D+V_L}{V_R}\right)^2\right] \tag{4c}$$

where M_R, V_R, T_R = true plastic bending, shear, torsion strength of girder, respectively; M_D, V_D, T_D = actual vending moment, shear force, and torsion due to dead load, respectively; M_L, V_L, T_L = actual bending moment, shear force, and torsion due to live load, respectively; F_{yu}, F_{yl} = yielding force of the half of the upper and lower stringer, respectively; ρ_b, ρ_T, ρ_M = perimeter ratio of bottom and top deck; H = depth of shear wall (distance between the longitudinal reinforcements enclosed by the stirrups); u = perimeter connecting the longitudinal stringers of the cross section.

(3) Strength Limit State Model for Combined Bending & Shear Stress of Steel Box Girders:

When the interaction type of combined failure limit state function needs to be considered, the interaction stress or strength failure limit state in terms of bending, shear and torsion may be used in the form of the code specified interaction equation without applying the safety factors. The interaction failure criteria specified in both Korean Standard Bridge Codes are based on the Maximum distortion energy theory. Thus, the nonlinear limit state function may be stated as follows (Cho, 1999):

$$g(\cdot) = 1.0 - \left\{\left(\frac{\sigma_D+\sigma_L}{\sigma_R}\right)^2 + \left(\frac{\tau_D+\tau_L}{\tau_R}\right)^2\right\} \tag{5}$$

where σ_R, τ_R are ultimate bending and shear stress, respectively; σ_D, σ_L are bending stress due to dead and live loads, respectively; and τ_D, τ_L are shear stress due to dead and live loads, respectively.

Also, σ_R and τ_R may be given as follows:

$$\sigma_R = \sigma_n N_\sigma D_F \tag{6a}$$

$$\tau_R = \tau_n N_\tau D_F \tag{6b}$$

where σ_n, τ_n are nominal ultimate bending and shear stress specified in the code, respectively; N_σ, N_τ are correction factors; and D_F is damage factor.

Again, σ_D, σ_L, τ_D and τ_L may be expressed, respectively, as,

$$\sigma_D = \tilde{\sigma}_D D_n N_{D\sigma} \tag{7a}$$

$$\sigma_L = \tilde{\sigma}_L L_n K N_{L\sigma} \tag{7b}$$

$$\tau_D = \tilde{\tau}_D D_n N_{D\tau} \tag{7c}$$

$$\tau_L = \tilde{\tau}_L L_n K N_{L\tau} \tag{7d}$$

where D_n, L_n are nominal dead and live loads, respectively; $\tilde{\sigma}_D$, $\tilde{\sigma}_L$, $\tilde{\tau}_D$, $\tilde{\tau}_L$ are the influence coefficients of bending and shear stress for dead and live loads, respectively; $N_{D\sigma}$, $N_{L\sigma}$, $N_{D\tau}$, $N_{L\tau}$ are correction factors.

3.1.2. Fatigue Limit State Model

In the case of steel girder bridges, the fatigue fracture limit state may be dominant over any other limit states. Therefore, for the evaluation of the fatigue reliability and remaining fatigue life or fatigue fracture risk of steel bridges, a simple fatigue reliability model was applied (Cho *et al*., 2001).

In the conventional S-N approach, the fatigue failure probability is used as the measure of the fatigue failure risk for the more rational assessment of the bridge collapse cause. Then, the fatigue failure probability PF may be approximately evaluated by the following formula proposed by the Ang-Munse (Cho *et al*., 2001) well suited for practical applications:

$$P_F = F_N(n) = 1 - R_N(n) \\ = 1 - \exp\left[-\frac{365 \times ADTT \times Yr}{n}\Gamma(1+\Omega_N^{1.08})^{\Omega_N^{1.08}}\right] \quad (8)$$

where Yr = expected fatigue life(year); $\Gamma(\cdot)$ = Gamma function; Ω_N = C.O.V. of the fatigue life which is chosen as 0.74 by Ang-Munse in the paper; $R_N(n)$ = reliability level at fatigue cycles n; and $F_N(n)$ = risk function equal to 1-$R_N(n)$ with respect to fatigue cycles n in terms of CDF(cumulative distribution function).

In the linear elastic fracture mechanics (LEFM) reliability approach, Zhao and Haldar (1994) developed a fracture reliability model considering crack growth. The limit state function of the model can be derived as follows:

$$g(\cdot) = \int_{a_0}^{a_c}\frac{da}{\left[F\left(\frac{a}{b}\right)\sqrt{\pi a}\right]^m} - C\left(\frac{2}{\sqrt{\pi}}\bar{S}\right)^m \Gamma\left(\frac{m}{2}+1\right)(N-N_0) \le 0 \quad (9)$$

All the basic uncertainties of the random variables of resistance and load effects described above can be obtained from data available in the literature (Ellingwood, 1980). However, these basic uncertainties should be updated or improved based on the site specific information such as visual inspections, measurements, NDTs, etc.

3.1.3. Mechanism Failure Limit State Model

The system reliability of bridges is formulated as a parallel-series model obtained from the FMA(Failure Mode Approach) based on the major failure mechanisms. For instance, for a system reliability analysis of steel box-girders, it can be assumed that the system failure state of box-girder bridge may be defined as the realization of collapse mechanism of major girders with or without considering the contribution of deck and cross beams. For this approach, an assumption is also made for the modeling of limit state such that approximate pseudo-mechanism analysis is possible by taking the critical buckling moment in compression failure zone or the yield moment in tension failure zone as the ultimate moment of box-girder section. In this study, the system modeling of steel box-girder superstructure is formulated as parallel (mechanism)-series models obtained from the FMA based on major failure mechanisms. The system modeling of steel box-girder bridge can be made either by considering or neglecting the contribution of cross beams to ultimate strength of mechanisms. Similarly, for RC and PSC box-girders, it can be assumed that in the case of bending failure modes, the system-failure state of box-girder bridge may be defined as the realization of collapse mechanism of major girders with or without considering the contribution of slabs and diaphragms, whereas in the case of shear-failure modes, the system failure is defined as the realization of the shear failure of all the webs at each negative critical sections of box-girder near supports. Thus, the system reliability of the bending or shear limit state of box-girder superstructure may be, respectively, formulated as parallel (mechanism)-series models obtained from the FMA based on major failure mechanisms.

For the system reliability analysis based on the collapse (or pseudo) mechanism of box girder bridges, the following limit state of failure mechanisms may have to be used (Cho *et al*., 1993) .

$$g(\cdot) = \sum_j c_{ij}M_{Rij} - \sum_k (b_{Dik}S_{Dik} + b_{Lik}S_{Lik}) \quad (10)$$

where M_{Rij} is moment strength of j-th section in i-th mechanism; S_{Dik}, S_{Lik} are applied dead and live load effects of k-th loading in i-th failure mechanism, respectively; and c_{ij}, b_{Dik}, b_{Lik} are coefficients that describe a collapse mode, respectively.

3.2. Reliability Methods

3.2.1. Reliability Models for Bridge Structures

Realistic reliability assessment of existing bridges may not be an easy problem because, in most case, it depends upon various sensitive uncertainty parameters and errors associated with models and methods. This apparently implies the most efficient reliability models and methods for each different type of bridges should be identified and developed for practical application. This paper employs practical and efficient models and methods for reliability assessment of various types of bridges.

It is important that the development of methods to efficiently and accurately quantify the reliability of bridge components and systems under static and dynamic loading including the effects of structural deterioration and/or damage. The safety of an existing bridge structure may be rated on the basis of specified target reliability. This requires the evaluation of either the element reliability or the system reliability of the bridge. Obviously, the failure of a single element or member does not constitute a system failure of highly redundant bridge system. As such, the collapse failure of a bridge-system is significantly different from element failures. The element reliability must be evaluated based on a dominant limit state from the various limit state models shown in Eqs. (1)~(10). However, the system reliability is obviously more desirable for realistic safety assessment (Bruneau, 1992; Fu and Moses, 1989; Tabsh and Nowak, 1991). For the more precise assessment of deteriorated and/or damaged bridges in critical conditions, the system reliability models must be used based on a practical and rational approach with limit states as strength, mechanism or fatigue fracture.

In the case of the strength limit state, the realization of collapse failure of an existing bridge may be defined as the limit state of system performance. Various descriptions for system failure or system resistance are possible based on either theoretical or practical approaches.

For the fatigue system reliability analysis of a steel bridge, theoretical formulation for the identification of all the system failure modes is extremely difficult. In this case an upper bound modeling may be desirable in practice by assuming that the failures of 2-3 adjacent critical details may trigger the system failure and that thus may be defined as the approximate system failure for the fatigue reliability analysis. Moreover, if one is concerned with the in-service fatigue reliability, it may also be possible to assume that the failure of a critical detail requires an immediate repair of the bridge when the first crack is observed. Then, the fatigue reliability of a steel bridge may be approximated by a series system, which drastically reduces the computational effort.

3.2.2. Element Level Reliability Analysis

The structural reliability may be numerically evaluated by the failure probability, P_F. However, for the application in practice, the relative reserve safety of a structural element or system may be best represented by the corresponding safety index $\beta = -\phi^{-1}(P_F)$, where ϕ^{-1} is inverse of the standard normal distribution function.

Various available numerical methods, appropriate methods for the limit state models were applied to the reliability analysis of bridges at either the element or the system level. In this paper, for the evaluation of element reliability, the advanced first order second moment (AFOSM) method usually with the Rackwitz-Fiessler (1978) normal tail approximation for non-Gaussian independent variates and with direct iteration for correlated variates is used for the element limit state function. Also, an improved IST algorithm (Cho and Kim, 1991) is used for the evaluation of the element reliability with combined interaction failure limit states as well as of the system reliability with the FMA modeling.

3.2.3. Practical System Reliability Assessment

The failure of a single element or a member does not constitute a system failure of highly redundant bridge system because the collapse failure of a bridge-system is significantly different from element failures. The realization of collapse failure state of an existing bridge may be defined as the limit state of system performance (Bruneau, 1992). Various descriptions for system failure or system resistance are possible based on either theoretical or practical approaches. Tabsh and Nowak (1991) defined the system failure of girder type bridges as the attainment of either a prescribed large amount of permanent deformation or unstable singular system stiffness matrix, for which they used an incremental nonlinear analysis with grid models of girder bridges. In this study, for a practical evaluation of the system reliability without involving extensive nonlinear structural analyses, the system failure state of a girder bridge may be defined as the realization of collapse mechanism of major girders. For this approach, an assumption is also made for the modeling of limit state such that an approximate pseudo-mechanism analysis in the limit state form as shown in Eq. (10) is possible by taking the critical buckling moment in compression failure zone or the yield moment in tension failure zone as the ultimate moment of box-girder sections. The system reliability problem of bridge superstructures can then be formulated as parallel (mechanism)-series models obtained from the FMA-based on dominant failure mechanisms. Also, for the system reliability analysis of high redundant bridge system the event tree analysis (ETA) model can be effectively applied by considering the major failure paths including combined failures of bridge components.

In most applications, all the uncertainties of the basic random variables of resistance and load effects to conduct the bridge reliability analysis may be obtained partly from data available in Korea and partly from some similar uncertainty data in the literature (Bruneau, 1992; Cho *et al*., 2001; Nowak, 1995; Nowak, 1990; Rackwitz, 1978; Tabsh and Nowak, 1991).

3.3. Safety Assessment

Fu (1989) and Cho and Ang (1989) have suggested that the reliability index be used as a rating criterion, as shown in Table 2, to predict a realistic reserve safety by incorporating actual bridge conditions and uncertainties. The reliability index β at either the element level (β_e) or the system level (β_s) may be used as a safety rating criterion. For instance, safety rating criterion in terms of β was often used in the application of bridge rating as a guide for safety assessment of actual existing bridges for maintenance purposes.

Table 2. Safety Rating Criterion [28]

β-criterion	Capacity	Maintenance
$\beta \geq \beta_o$	Normal (Safe)	Regular visual inspection
$2.0 \leq \beta < \beta_o$	Limited (SLR & MOR to be evaluated)	Repair with weight-limit to be posted
$1.0 \leq \beta < 2.0$	Seriously limited (SLR & MOR to be evaluated)	Repair, rehabilitation with weight limit to be posted
$\beta < 1.0$	Completely lost	Strengthening or replacement

*Target reliability $\beta_o = 3.0$ for the SLR(Service Load Rating), 2.0 for the MOR(Maximum Overload Rating)

The nominal safety factor n' was also invariably used as a safety measure in the safety assessment of actual bridges. The uncertainty-based safety factor n' can be expressed in the following form corresponding to the FOSM reliability index β evaluated by a reliability analysis:

$$n' = \frac{\eta_Q}{\eta_R}\exp(\beta\sqrt{(\Omega_R^2 + \Omega_Q^2)}) \tag{11}$$

where η_R, η_Q are the mean-nominal ratio of resistance R and load effect Q, respectively; and Ω_R, Ω_Q are the coefficient of variation (COV) of resistance and load effects, respectively.

It may be stated that the uncertainty-based nominal safety factor n' should be distinguished from the conventional safety factor n($=R_n/Q_{\max}$ or $\sigma_R/\sigma_{\max}$), which is traditionally used as the notional safety in engineering practice. In this paper, the nominal safety factor n' is used as a rational concept of structural safety together with the safety index β in practice.

4. Capacity Rating Methods for Existing Bridges

An interaction-type rating formula is presented for the codified rating at the element level. For more realistic capacity rating utilizing the reserve safety of redundant bridge such as box-girder bridges, the system reliability index is used as a β-rating criterion. In addition, it is also demonstrated that a practical and rational approach for the evaluation of reserve system carrying capacity - system rating load (P_{ns}) or rating factor (RF) - in the form of the equivalent system-strength proposed by Cho *et al.* (1993), which is derived based on the inverse fitting of the conceptual FOSM form of system reliability index. For a comparative study with the codified load and resistance factor rating (LRFR) criteria previously developed by Cho and Ang (1989), the LRFR criteria is also given herein.

Various types of the rating equations shown in the following sections have been developed for the capacity rating of steel bridges. Based on experiences with the extensive applications, it may be positively stated that the reliability-based safety and capacity evaluation methods presented in the paper are far more realistic and rational than the conventional methods.

4.1. Codified Capacity Rating

The current practice for the bridge capacity rating in Korea is primarily based on the conventional WSR because most engineers still prefer to use the working stress concept in design and capacity rating. Noting that the RF is defined as a ratio of the actual reserve load carrying capacity, P_n, to the standard rating or design load, P_r, the WSR has many serious drawbacks as a tool for realistic bridge capacity rating, because it does not systematically take into account any information on the degree of deterioration or damage and the uncertainties specific to the bridge to be rated. In order to remove some of the above drawbacks of the conventional WSR equation, instead, an improved advanced working stress rating (AWSR) was often used in the bridge capacity rating. As a rational alternative to the conventional WSR method, a reliability-based LRFR criterion incorporating the response factor K and damage factor D_F of the bridge was developed and used for more realistic and consistent assessment of the safety and capacity of a bridge by Cho and Ang (1989).

(1) LRFR for strength limit states:

$$P_n = \frac{\phi' D_F R_n - \gamma'_D c_D D_n}{\gamma'_L c_L K}, \quad RF = \frac{P_n}{P_r} \tag{12}$$

in the Eq. (12), R_n is nominal strength specified in the code; D_F is damage factor; D_n is nominal dead load; c_D, c_L are the influence coefficients of dead and live load effects; P_r is the standard design or rating load; and $\phi', \gamma'_D, \gamma'_L$ are the nominal resistance, dead and live load factors, respectively.

(2) LRFR for combined strength limit states:

Since combined load effects of compression and bending can not be evaluated in the conventional WSR, LFR and LRFR formula, an improved LRFR criterion is proposed based on the code-specified LRFD interaction Eq. (1) considering axial and bending effects for more precise capacity rating of steel bridges. It may be seen as follows (Cho *et al*.,1997):

For $P_u \geq 0.2\phi P_n$

$$\frac{\gamma'_d P_d + \gamma'_l P_l K_c RF}{\phi P_n D_{Fc}} + \frac{8}{9}\left(\frac{\gamma'_d M_d + \gamma'_l M_l K_b RF}{\phi'_b D_{Fb} M_n}\right) = 1 \tag{13a}$$

For $P_u < 0.2\phi P_n$

$$\frac{\gamma'_d P_d + \gamma'_l P_l K_c RF}{2\phi P_n D_{Fc}} + \left(\frac{\gamma'_d M_d + \gamma'_l M_l K_b RF}{\phi'_b D_{Fb} M_n}\right) = 1 \tag{13b}$$

where P_d, P_l are nominal compressive forces due to dead and live load effects, respectively; P_{cr} is compressive buckling strength; M_d, M_l are nominal bending moment due to dead and live load effects, respectively; M_n is nominal bending strength; ϕ'_c , ϕ'_b are strength reduction factor for compression and bending; γ'_d , γ'_l are dead and live load factors, respectively, which should be calibrated but can be used in practice as the values specified in the code; K_c, K_b are response ratio for compressive and for bending stress, respectively; and D_{Fc}, D_{Fb} are damage factors for the compression and bending, respectively. Note that the rating factor RF can be easily obtained by solving Eq. (13) for the RF.

4.2. Non-codified Equivalent-capacity Rating

An analytical prediction of the reserve load carrying capacity of an aged bridge as a system is almost impossible in general. Even a numerical evaluation is quite difficult particularly when the structure is highly redundant and significantly deteriorated and/or damaged. Therefore, Cho (1993) proposed a practical and rational approach for the evaluation of realistic load carrying capacity of existing bridges as a system in terms of the equivalent system strength. In this paper, the key concept of non-codified equivalent-capacity rating is simply described for convenience.

The system reliability index β_s may be conceptually expressed as the ln-ln model of the FOSM form of second moment reliability methods in the following way:

$$\beta_s \cong \frac{\ln(\overline{R_s}/\overline{Q_s})}{\sqrt{\Omega_{Rs}^2 + \Omega_{Qs}^2}} \tag{14}$$

where $\overline{R_s}$ is mean system resistance; and $\overline{Q_s}$ is mean system load effects; and Ω_{Rs}, Ω_{Qs} are the COV of system resistance and load effects. Then, noting that mean system load effects $\overline{Q_s}$ may be expressed in terms of system mean dead and live load effects ($\overline{Q_s} = \bar{c}_{Ls} P_{ns} + \bar{c}_{Ds} D_n$ in which $\bar{c}_{Ls}$, $\bar{c}_{Ds}$ are average unit system mean dead and live load effects, respectively, D_n is nominal dead load effects).

Therefore, Eq. (14) may be solved for P_{ns} as follows:

$$P_{ns} = Z_m \mathrm{EXP}(-\beta_s \Omega_s) - \eta_s D_n \tag{15}$$

where Z_m is a parameter that conceptually represents the mean resistance safety ratio ($(\overline{R}_s/\bar{c}_{Ls})$; Ω_s is parameter that conceptually represents the system uncertainties ($=\sqrt{\Omega_{Rs}^2 + \Omega_{Qs}^2}$); and η_s is ratio of unit system mean dead and live load effects($=\bar{c}_{Ds}/\bar{c}_{Ls}$). The relationship between P_{ns} and β_s can be represented by the exponential curve corresponding to Eq. (15).

Thus, the unknown parameters Z_m and Ω_s can be evaluated when the two distinct rating points (P_{R1}, β_{s1}), (P_{R2}, β_{s2}) are substituted into Eq. (18). Note that these may be obtained as the system reliability indices β_{s1} and β_{s2} corresponding to the upper and lower standard rating load P_{R1} and P_{R2}, respectively. Thus, Eq. (15) becomes:

$$P_{R1} = z_m EXP(-\Omega_s \beta_{s1}) - \eta_s D_n \tag{16a}$$

$$P_{R2} = z_m EXP(-\Omega_s \beta_{s2}) - \eta_s D_n \tag{16b}$$

The system reliability-based capacity evaluation in the paper may be conceptually represented in the Fig. 1.

From Eqs. (16a) and (16b) the parameters Z_m and Ω_s can be derived as follows:

$$Z_m = \left[\frac{(P_{R2} + \eta_s D_n)^{\beta_{s1}}}{(P_{R1} + \eta_s D_n)^{\beta_{s2}}}\right]^{\frac{1}{\Delta\beta}} \tag{17}$$

$$\Omega_s = -\frac{1}{\Delta\beta}\ln\left(\frac{P_{R1} + \eta_s D_n}{P_{R2} + \eta_s D_n}\right) \tag{18}$$

where $\Delta\beta = \beta_{s1} - \beta_{s2}$.

Finally, substituting Eq. (17) and Eq. (18) into Eq. (15), P_{ns} may be derived in the following form:

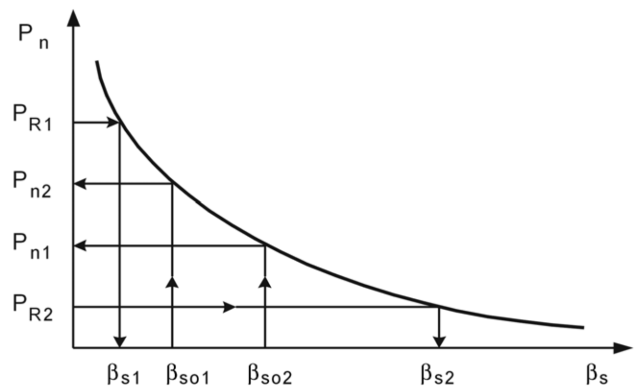


Fig. 1. β_s versus P_n

$$P_{ns} = \frac{(P_{R2} + \eta_s D_n)^{\Delta\beta_{s1}/\Delta\beta_{so}}}{(P_{R1} + \eta_s D_n)^{\Delta\beta_{s2}/\Delta\beta_{so}}} \eta_s D_n \tag{19}$$

where P_{ns} is nominal equivalent system-strength; P_{R1}, P_{R2} are upper and lower rating loads, respectively; $\Delta_{\beta s1} = \beta_{s1} - \beta_{so}$ and $\Delta\beta_{s2} = \beta_{s2} - \beta_{so}$, in which β_{s1}, β_{s2} are system reliability corresponding to P_{R1} and P_{R2}, respectively; and β_{so} is target reliability.

Hence, system reliability-based load carrying capacity may be evaluated in terms of equivalent system-strength either by curve fitting on the Fig. 1 or by the calculating formula in Eq. (19). Also for the capacity rating at the element level with implicit or interactive limit state, it may be noted that the similar concept can be applied to the derivation of the equivalent element-strength, P_{ne}.

5. Applications

In order to demonstrate the applicability of the proposed models and methods for reliability-based safety assessment, rating, and system redundancy evaluation of existing bridges, some real applications are demonstrated, and the results are briefly presented and comparatively discussed with conventional methods.

5.1. Safety Assessment and Rating of Existing Bridges

In the last 10 years, the safety assessments and ratings of a number of short and medium span bridges of various types such as RC T-beam, RC slab, RC box girder, PSC beam, steel I-beam, plate girders, steel box girder, and steel truss bridges were carried out based on inspections, NDTs and systematic static/dynamic field load tests. The general information and response test data of the selected existing bridges are presented in Table 3~4 and some of the essential results of safety evaluation and capacities rating for some of those bridges are summarized in Table 5~6.

As shown in Table 4, it may be found that the reliabilities (β_e=0.61~2.83) for most of the short and medium span bridges are lower than the target reliability (β_o=3.0 for SLR). These results are consistent with the current condition of short span bridges seriously deteriorated and/or damaged, which may be attributed to the heavy truck traffic beyond the design load. As it can also be seen in Table 5, in the case of steel I-beam bridges, reliability indices at the system level (β_s=2.16, 3.00, respectively) is significantly higher than those at

Table 3. The General Information of Selected Existing Bridges

Bridge Type	Bridge Name	Span Length (m)	No. of Girder	Girder Spacing (m)	Girder Depth (m)
RC T-Beam	Kyo-pyung	12	4	2.1	1.0
	Seo-si	12	6	1.8	1.1
RC Slab	Taeha-pum	10	1	-	0.5
	Hyun-bang	11	1	-	0.5
RC Box	Sum-jin	34	3	3.40	2.20
	Sa-su II	35	1	-	2.40
PSC Beam	San-jung	20.5	5	1.8	1.4
	Won-hyo IC	30	6	2.0	2.2
Steel I-Beam	Kum-chuck	15	5	2.1	1.5
	Shin-duck	12	5	1.3	0.8
Steel Box	Ham-an	35	2	5.0	1.7
	Sang-ill	45.5	4	6.9	1.9

Table 4. Test Response Data of the Bridges

Bridge Type	Bridge Name	Rating Load	Design Load	DF	K	1+I	Fundament frequency (Hz)
RC T-Beam	Kyo-pyung	DB24	DB13.5	0.78	0.98	1.12	11.5
	Seo-si	DB24	DB18	0.41	0.69	1.23	6.5
RC Slab	Taeha-pum	DB24	DB18	0.7	2.1	1.15	8.6
	Hyun-bang	DB24	DB24	0.8	1.11	1.11	10.5
RC Box	Sum-jin	DB24	DB24	0.7	0.77	1.23	2.7
	Sa-su II	DB24	DB24	0.8	1.0	1.31	2.9
PSC Beam	San-jung	DB24	DB18	0.99	0.90	1.17	7.8
	Won-hyo IC	DB24	DB18	0.71	1.24	1.27	3.0
Steel I-Beam	Kum-chuck	DB24	DB18	0.94	0.53	1.37	6.4
	Shin-duck	DB24	DB24	0.77	0.88	1.07	9.0
Steel Box	Ham-an	DB24	DB24	1.0	0.68	1.24	3.1
	Sang-ill	DB24	DB24	1.0	0.48	1.21	2.8

Note: Values are obtained by applying Korean Standard Design Truck Load DB-18 equivalent to AASHTO HS-20, and values in parentheses are obtained by applying Korean Standard Design Truck Load DB-24 equivalent to 1.33times of AASHTO HS-20.

Table 5. Reliability Index and Safety Factor of the Bridges

Bridge Type	Bridge Name	β_e	β_s	Nominal safety factor (n')	Conventional safety factor (n)
RC T-Beam	Kyo-pyung	0.61	-	1.17	1.57
	Seo-si	2.02	-	1.67	3.76
RC Slab	Taeha-pum	1.26	-	1.31	3.66
	Hyun-bang	2.41	-	1.72	2.44
RC Box	Sum-jin	1.34	1.80	1.29	1.32
	Sa-su II	1.75	-	1.67	2.22
PSC Beam	San-jung	1.54	-	1.41	1.48
	Won-hyo IC	3.68	-	2.37	4.13
Steel I-Beam	Kum-chuck	1.60	2.16	1.51	1.34
	Shin-duck	2.17	3.00	1.89	2.34
Steel Box	Ham-an	4.22	5.20	2.43	4.61
	Sang-ill	2.83	4.86	1.70	2.14

Table 6. Rating Factor of the Bridges

Bridge Type	Bridge Name	WSR	SR	LRFR	Non-Codified (Element)	Non-Codified (System)
RC T-Beam	Kyo-pyung	0.17	-	0.33	-	-
	Seo-si	0.55	-	0.86	-	-
RC Slab	Taeha-pum	0.63	-	0.53	-	-
	Hyun-bang	0.68	-	0.77	-	-
RC Box	Sum-jin	0.00	0.94	0.00	0.11	0.37
	Sa-su II	0.08	-	0.39	-	-
PSC Beam	San-jung	1.08	-	0.64	-	-
	Won-hyo IC	0.23	1.61	1.32	-	-
Steel I-Beam	Kum-chuck	1.35	0.59	0.67	-	-
	Shin-duck	0.91	0.76	0.80	-	-
Steel Box	Ham-an	4.50	2.83	3.41	3.41	4.28
	Sang-ill	3.49	2.42	2.53	2.21	3.26

the element level (β_e=1.60, 2.17, respectively) mainly due to the redundancy provided by multiple girders and cross beams. These observations suggest that system effects may have to be considered in practice for the assessment of residual safety or carrying capacity of redundant bridges in particular.

As mentioned above, the system reliability indices (β_s = 4.86, 5.20, respectively) of steel box girder bridges as shown in Table 5 are considerably higher than the element reliability indices (β_e = 2.83, 4.22, respectively). Based on experiences with the bridge reliability, it may be positively stated that the system reliability should be considered for more precise assessment of the reserve system safety and load carrying capacity of box girder bridges.

It may also be seen in Fig. 2 that the results of nominal safety factor n' based on the reliability index are more reasonable than those of conventional safety factor n that provides only pure notional values with unreasonable fluctuation. Moreover, it may be realized that the results of the WSR, the stress rating (SR) and the LRFR comparatively shown in Table 6 are significantly different in some bridges such as Won-hyo IC, which may again indicate that the WSR cannot realistically predict the bridge capacity but rather provide pure notional results. On the contrary, it can be observed that the

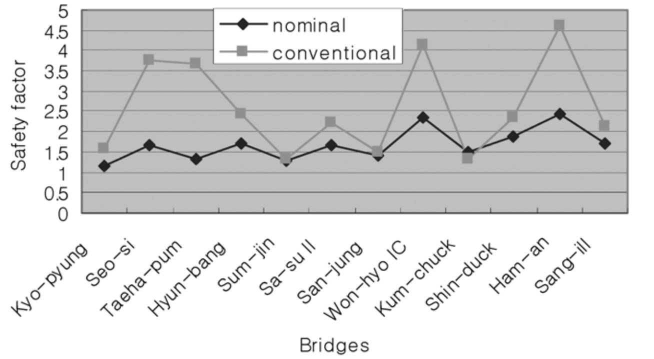


Fig. 2. Comparisons between Nominal and Conventional Safety Factors

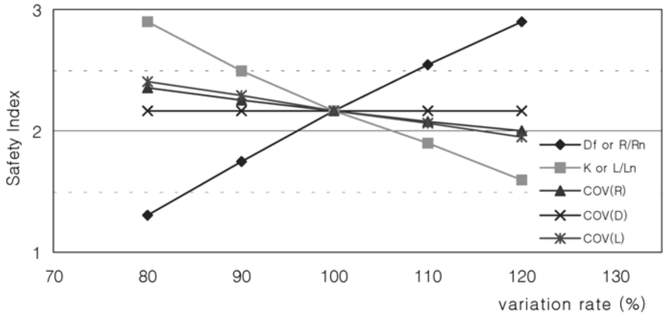
results of the reliability evaluation and the LRFR are reasonably compatible and invariably consistent especially when the bridges are deteriorated and/or damaged. It is observed that the results of codified rating methods show somewhat diverse tendency mainly due to damage effects, bridge response and some inherent differences in the parameters of the rating methods. However, it may be admitted that the LRFR methods in general provide more rational results consistent with actual bridge conditions.

It is also interesting to observe that the codified rating results of Sumjin Bridge, one of the RC box girder bridges listed in the Table 6, have zero values which apparently means that the carrying-capacity of the bridge is completely lost, and which is somewhat consistent with extensive surface cracks due to the critical shear failure of box webs. But at the system level the residual capacity is comparatively high because the failure of a single element or member does not constitute a system collapse of redundant structures as most box girders have cross beams, bracings, diaphragms and deck stiffness. Thus, it may be concluded that the system reliability-based approach is essential to predict residual capacity, especially in the case of seriously damaged or deteriorated redundant bridges.

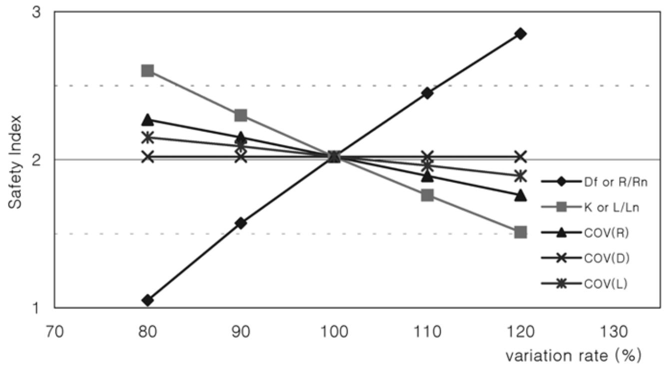
Moreover, the results evaluated by the capacity rating method based on the system (or element) reliability-based, so-called non-codified capacity rating method, are also listed in Table 6. The method could be very effective in practice in the case of highly redundant bridges such as steel box girders multi-cellular RC box girders. As it can be seen in Table 6, since the non-codified rating results at the element level are compatibly consistent with those of the LRFR and the element reliability, the equivalent system capacity rating corresponding to the system reliability index should provide more reasonable results.

5.2. Sensitivity of Parameters and Uncertainties

In order to identify sensitive parameters and uncertainties in the bridge reliability, sensitivity analyses were performed for RC T-beam and steel I-beam bridges. Results of the sensitivity analyses for the parameters that greatly influence the reliability and capacity of bridges, such as the mean-nominal ratio for load and resistance, damage factor D_F, response ratio K, the COV's for load and resistance, are given in Fig. 3. It can be seen in Fig. 3 that the major parameters such as the mean-nominal ratio, damage factor, and response ratio are considerably more sensitive compared with the COV's. Based on the observation, it may be stated that for more realistic and precise reliability and capacity rating, the mean-nominal ratio, damage factor, and response ratio should be estimated more carefully than the COV.



(a) Steel I-Beam Bridges



(b) RC T-Beam Bridges

Fig. 3. Parameter Sensitivities of the Selected Bridges

6. Concluding Remarks

This paper is intended to suggest practical reliability-based assessment models and methods for the safety assessment and capacity rating of various kinds of actual existing bridges. Moreover, the application of recently developed a new approach, called equivalent system-strength, for reliability-based capacity rating is illustrated using the data obtained from actual bridges. This paper also summarizes various approaches to reliability-based safety assessment and capacity rating, and investigates their application to various existing aged bridges.

Based on the observations made in many bridge assessment researches and projects related to the proposed safety assessment and rating methods, the following remarks can be made:

(1) Reliability methods for both safety assessment and rating are important and should be utilized in practice especially for bridge structures. Based on extensive applications to the various types of real bridges, it may be concluded that the reliability-based safety and capacity evaluation methods suggested in the paper provide more realistic assessment than the conventional methods and can be a valuable tool in practice.

(2) Though, in the last five decades, many structural reliability methods have been developed, it still needs more experimental verifications and/or further improvement of these methods based on extensive experiment and field test results. Moreover, for more precise assessment of reserve safety and capacity rating of existing bridges, it is demonstrated that the system reliability-based approach is more rational and realistic than the element reliability-based approach. The system reliability and system reliability-based capacity evaluation can be used as an effective tool in practice for the realistic assessment of reserve safety and the carrying capacity rating of existing bridges.

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