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ASSESSMENT OF SYSTEM RELIABILITY AND CAPACITY-RATING OF COMPOSITE STEEL BOX-GIRDER HIGHWAY BRIDGES

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This paper develops practical and realistic reliability models and methods for the evaluation of system-reliability and system reliability-based rating of various types of box-girder bridge super-structures. The results of the reliability evaluation and rating of existing bridges indicate that the reserved reliability and capacity rating at system level are significantly different from those of element reliability or conventional methods especially in the case of highly redundant box-girder bridges.

Key Words : *interactive limit state function, system reliability, probabilistic system redundancy factor, probabilistic system reserve factor, capacity-rating.*

1. INTRODUCTION

The optimal decision on the bridge maintenance and rehabilitation involves tremendous economic and safety implication, and heavily depends upon the results of the evaluation of residual safety and carrying-capacity of the existing bridges. However, in spite of the remarkable advances in modelling and numerical or analytical techniques for response or behavior analysis and ultimate strength or stability analysis of redundant bridge system such as steel box-girder bridges, it is still extremely difficult to evaluate realistic residual system-safety or system carrying-capacity especially when these bridges are deteriorated or damaged to a significant degree.

It has to be noted that the cross beams and diaphragms as well as concrete decks of box-girder bridge significantly contribute to transverse load distribution and redundancy, and also effectively provide the resistance to eccentric loads. Thereby, the capacity rating and safety evaluation of box-girder bridges based on system performance and system reliability are very important for the realistic prediction of residual carrying capacity of these highly redundant bridge system.

Recently, some practical system reliability models and methods for I-girder bridges[1),8),10)] have been suggested with the emphasis on the system-level reliability rather than the element level. Also some practical approaches for reliability based safety assessment and capacity rating have been made by Cho and Ang[2)].

Various approaches to system redundancy measure have been proposed by several researchers[5)]. However, no established practical approaches to the assessment of system redundancy or system reliability are available especially for continuous, composite and multi-box steel girder bridges.

This paper develops practical reliability models and methods for the evaluation of system reliability and proposes a new method for the system reliability-based redundancy and capacity-rating that incorporate the concept of non-codified equivalent system-capacity of existing bridges.

2. LIMIT STATE MODEL

(1) Element Limit State Model

Since this study is concerned with the ultimate carrying-capacity of existing steel box-girder bridges, the ultimate limit state functions need to be considered in the modelling. The strength limit state models at element level for box-girder bridges suggested in the paper are based on the basic flexural strength as well as the strength interaction equations which simultaneously take into account flexure, shear and torsion.

a) Bending Strength Limit State Model

When the primary bending failure alone is considered, the linear bending strength equation becomes,

$$g(\cdot)=M_R-(M_D+M_L) \quad \cdots\cdots (1)$$

where M_R=true moment strength ; M_D, M_L=true applied moment. A realistic safety assessment or rating of existing bridges requires a rational determination of the degree of deterioration or

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damage. Therefore, the true moment strength M_R in Eq.(1) must incorporates some random variates to reflect such deterioration/damage and the underlying uncertainties. System identification techniques[13] may be used for the precise evaluation of the damage factor, D_F, proposed in the paper. However, in lieu of such elaborate techniques as system identification, the resistances of the deteriorated or damaged members are made to be estimated on the basis of visual inspection and/or simple in-situ tests with a practical analysis utilizing the FFT techniques, supplemented with the engineering judgement. The true moment strength M_R may be modelled as,

$$M_R = M_n N_M D_F \cdots\cdots (2)$$

where M_n=nominal moment strength specified in the code ; N_M=the correction factor adjusting any bias and incorporating the uncertainties involved in the assessment of M_n and D_F (=$MFPD$), in which, M= material strength uncertainties ; F=fabrication and construction uncertainties ; P=prediction and modelling uncertainties ; D=the uncertainties involved in the assessment of damages and/or deterioration ; D_F=the damage factor, which is the ratio of current stiffness, K_D, to the intact stiffness, K_I, i.e.$K_D/K_I(\cong \omega_D^2/\omega_I^2)$ in which ω_D and ω_I are the fundamental natural frequencies of the damaged and the intact structures.

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 M_D, M_L may be expressed in terms of respective random variate as follows,

$$M_D = m_D D_n N_D \cdots\cdots (3a)$$

$$M_L = m_L L_n K N_L \cdots\cdots (3b)$$

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

b) Interactive Failure Limit State Model

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 and Japanese Standard Bridge Codes are based on the maximum distortion energy theory[11]. Thus, the nonlinear limit state function may be stated as follows,

Table 1 Statistical uncertainties

Variables	$\bar{X}/X_n$	Ω_x	Dist.
Resistance	1.08	0.15	log-normal
Dead load effect	1.05	0.10	normal
Live load effect	1.15	0.19	log-normal

$$g(\cdot) = 1.0 - \left\{\left(\frac{\sigma_D + \sigma_L}{\sigma_R}\right)^2 + \left(\frac{\tau_D + \tau_L}{\tau_R}\right)^2\right\} \cdots\cdots (4)$$

where σ_R, τ_R=true ultimate or critical bending and shear stress ; σ_D,σ_L, τ_D, τ_L=true bending and shear stress, respectively.

Also in the similar form as Eq.(2), σ_R and τ_R may be given as follows,

$$\sigma_R = \sigma_n N_\sigma D_F \cdots\cdots (6a)$$

$$\tau_R = \tau_n N_\tau D_F \cdots\cdots (5b)$$

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

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

$$\sigma_D = \underset{\sim}{\sigma}_D D_n N_{D\sigma} \cdots\cdots (6a)$$

$$\sigma_L = \underset{\sim}{\sigma}_L L_n K N_{L\sigma} \cdots\cdots (6b)$$

$$\tau_D = \underset{\sim}{\tau}_D D_n N_{D\tau} \cdots\cdots (6c)$$

$$\tau_L = \underset{\sim}{\tau}_L L_n K N_{L\tau} \cdots\cdots (6d)$$

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

(2) Mechanism Failure Limit State Model

For the system reliability analysis based on the collapse mechanism of box girder bridges, the following limit state of failure mechanisms may have to be used,

$$g_i(\cdot) = \sum_j c_{ij} M_{Rij} - \sum_k (b_{Dik} S_{Dik} + b_{Lik} S_{Lik}) \cdots\cdots (7)$$

where M_{Rij}=true moment strength of j th section in i th mechanism ; S_{Dik}, S_{Lik}=true applied load effects of k th loading in i th failure mechanism ; c_{ij}, b_{Dik}, b_{Lik}=coefficients that describe a collapse mode. The true resistance M_{Rij} and applied load effects S_{ik} may be modelled in the same way as Eq.(2) and (3).

(3) Statistical Uncertainties

In the paper, all the uncertainties of the basic random variables of resistance and load effects described above are obtained from data available in the literature[4),12)]. The mean-nominal ratio and COV of resistance and load effects with the

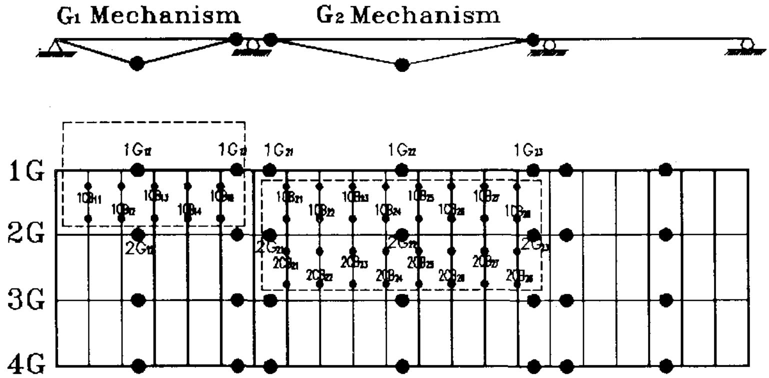

Fig.1 Collapse failure state of box-girder bridge

assumed distributions are summarized in **Tabie 1.**

3. SYSTEM RELIABILITY

(1) Reliability Analysis

The structural reliability may be conceptually measured or numerically evaluated by the failure probability, P_F. However practically, relative reserve safety of a structural element or system may be best represented by the corresponding safety index β, i.e.,

$$\beta = -\Phi^{-1}(P_F) \quad \cdots\cdots (8)$$

where P_F=probability of failure ; Φ^{-1}=inverse of the standard normal distribution function.

Various available numerical methods can be applied for the reliability analysis of the bridges at either element or system level using the limit state models proposed in the paper. For the evaluation of element reliability, the AFOSM method[4] incorporating the equivalent-normal transformation and Hasofer-Lind's iterative algorithm is used for bending limit state function, but an improved IST (Importance Sampling Technique) algorithm which is implemented as a computer code developed by the author[3] is used for the system reliability and interactive failure limit state analysis of the proposed models. The proposed IST algorithm for the computation FMA(Failure Mode Approach) of system failure (10a) probability $P_{F(sys)}$ $(=\int_X I[X]\,[f(x)/h(x)]$ concerned with $h(x)$ $dx)$ is based on the following proposes importance samling scheme,

$$h(x) = \sum p_i h_i(x) \quad \cdots\cdots (9a)$$

$$p_i(x) = \frac{f_i(x^*)}{\sum f_i(x^*)} \quad \cdots\cdots (9b)$$

where x=basic random variables ; x^*=MLF(Maximum Likelihood Failure) point ; $I[X]$=indicator function ; $f(x)$=joint probability density function of x ; $h(x)$=sampling density function proposed as a form of the composition density function ; p_i=probability or relative weight associated with $h_i(x)$, the i-th sampling density of i-th failure mode. It may be noted that the introduction of $h(x)$ is to locate more samples on more important regions of x which could make significant contributions of $P_{F(sys)}$. As it is demonstrated in the references[7] the main advantage of using the IST may be attributed to the fact that the difficulties associated with the intractable problems such as interactive non-linear limit state functions, correlated non-normal variates and complexity or generality of reliability model do not appear to exist with the IST simulation and thus do not seem to significantly affect the convergence rate or result.

(2) Practical System Reliability Assessment for Box-Girder Bridges

Obviously, the failure of a single element or member does not constitute a system failure of highly redundant bridge system such as steel box-girder bridges. As such, 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. Various descriptions for system failure or system resistance are possible based on either theoretical or practical approaches. Nowak[10] 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 he used an incremental nonlinear analysis of grid model. However, in this study for the practical but efficient numerical solution of system reliability without involving extensive nonlinear structural analysis, it is 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 modelling 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 as shown in **Fig.1**. And thus, the system reliability problem of box-girder super-structure is formulated as parallel (mechanism)-series models obtained from the FMA (Failure Mode Approach) based on major failure mechanisms. **Fig.1** shows a typical illustrative example of some failure mechanisms of grid model for a 3-span continuous box-girder bridge.

The system modelling of steel box-girder bridge can be made either by considering or neglecting the contribution of cross beams to ultimate strength of mechanisms. Also, it may be noted that at element reliability level the load effect of each girder is obtained from the linear elastic analysis of grid model for box-girder bridges.

(3) Probabilistic System Redundancy and Reserve Safety

The collapse of a bridge occurrs by taking multiple failure paths involving various intermediate damage or failure state after initial failure. Thus, the definition of system redundancy requires a measurement of the ultimate capacity of a bridge to resist collapse failure following the initial element failure.

Thereby, the results of system-reliability analysis in terms of system reliability index may be effectively used either as a probabilistic measure of system redundancy or reserve safety of existing bridges. So far, various approaches to the definition for the measure of the system redundancy or reserve safety have been suggested by several researchers[5),9)].

The following definitions for system redundancy and reserve safety in terms of reliability indices are adopted in the paper as a measure of system redundancy and reserve safety[6)].

$$PSRF=\beta_s/\beta_i \quad \text{(10a)}$$

$$PSReF=\beta_s/\beta_e \quad \text{(10b)}$$

where $PSRF$ = Probabilistic System Redundancy Factor ; $PSReF$ = Probabilistic System Reserve Factor ; β_s = system reliability index ; β_i = reliability index of initial-failure element ; β_e = element reliability index.

Because, this study is primary concerned with the capacity rating of existing box-girder bridges PSReF will preferably be used in the numerical application.

4. SYSTEM RELIABILITY-BASED RATING

The conventional capacity rating of a bridge have been largely based on the WSR (Working Stress Rating) or LFR (Load Factor Rating) criterions which do 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 or redundancy specific to the bridge to be rated. Thus, unfortunately the nominal rating load or reserve capacity evaluated by the conventional code-specified formula, in general, fails to predict realistic carrying capacity or reserve capacity of deteriorated or damaged bridges. Recently, Moses[9)] and the authors[12)] have suggested reliability index to be used as a rating criterion to predict realistic relative reserve safety by incorporating actual bridge conditions and uncertainties.

For more realistic capacity rating utilizing the reserve safety of redundant bridge such as box-girder bridges, the system reliability index is suggested to be used as a β-rating criterion. Also, this paper proposes a practical but rational approach for the evaluation of capacity-rating load (P_n) or rating factor (RF) in the form of the equivalent system-capacity rating, which is derived based on the concept of FOSM form of system reliability index. For a comparative study with the codified LRFR (Load and Resistance Factor Rating) criteria previously developed by the author[12)], the LRFR criteria will also be given herein.

Table 2 Load and resistance factor for rating of steel girder bridges

Target Rel.	ϕ'	γ_D'	γ_L'
β_{o1}=3.0(SLR)	0.85	1.20	2.0
β_{o2}=2.5(MOR)	0.95	1.20	1.7

(1) Codified LRFR Criteria

The following general LRFR criterion may be developed corresponding to a specified target reliability index[2)].

$$P_n=\frac{\phi' D_F R_n-\gamma_D' C_D D_n}{\gamma_L' C_L K} \quad \text{(11)}$$

$$RF=\frac{P_n}{P_r} \quad \text{(12)}$$

where P_n = the nominal load carrying capacity ; P_r = the standard design or rating load ; ϕ', γ_D', γ_L' = the norminal resistance, dead load and live load factors, respectively ; R_n = nominal strength specified in the code, which becomes M_n of Eq.(2) in the case of flexural limit state ; c_D, c_L = unit mean dead and live load effects, which are also identical with m_D, m_L of Eq.(3a, b), in the case of flexural limit state.

The results of calibration for steel girder bridges

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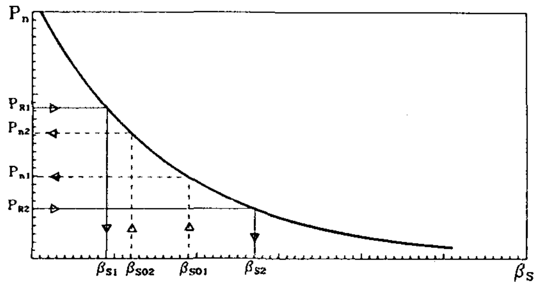

Fig.2 β_s versus P_n

corresponding to SLR (Service Load Rating, $\beta_{co1}=3.0$) and MOR (Maximum Overload Rating, $\beta_{co2}=2.5$) are shown in **Table 2.**

(2) Non-Codified System-Reliability-Based Equivalent-Capacity Rating

The system reliability index β_s may be conceptually expressed as the ln-ln model of the FOSM form of 2 nd moment reliability methods in the following way,

$$\beta_s \cong \frac{\ln(\bar{R}_s/\bar{Q}_s)}{\sqrt{\Omega_{Rs}^2+\Omega_{Qs}^2}} \cdots\cdots\cdots (13)$$

where $\bar{R}_s$=mean system resistance ; and $\bar{Q}_s$=mean system load effects, Ω_{Rs} and Ω_{Qs}=the COV of system resistance and load effects. And $\bar{Q}_s$ may be expressed in terms of system mean dead and live load effects ($\bar{Q}_s=\bar{c}_{Ls}P_n+\bar{c}_{Ds}D_n$), Ep.(13) may be solved for P_n as follows ;

$$P_n=Z_m\mathrm{EXP}(-\beta_s\Omega_s)-\eta_sD_n \cdots\cdots\cdots (14)$$

where Z_m=parameter that conceptionally represents the mean resistance safety ratio ($\bar{R}_s/\bar{c}_{Ls}$) ; $\Omega_s=\sqrt{\Omega_{Rs}^2+\Omega_{Qs}^2}$=parameter that conceptionally represents the system uncertainties ; η_s=ratio of unit system mean dead and live load effects($=\bar{c}_{Ds}/\bar{c}_{Ls}$). As shown in **Fig.2** the relationship between P_n and β_s can be graphically represented by the exponential curve corresponding to Eq.(14).

Thus, the unknown parameters Z_m and Ω_s of Eq.(13) can be evaluated when the two distinct rating points(P_{Ri}, β_{si}) are substituted into Eq.(14). Note that these are 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.(14) becomes

$$P_{R1}=Z_m\mathrm{EXP}(-\Omega_s\beta_{s1})-\eta_sD_n \cdots\cdots\cdots (15a)$$

$$P_{R2}=Z_m\mathrm{EXP}(-\Omega_s\beta_{s2})-\eta_sD_n \cdots\cdots\cdots (15b)$$

From Eq.(15a) and (15b) the parameters Z_m and Ω_s can be derived as follows ;

$$Z_m=\left[\frac{(P_{R2}+\eta_sD_n)^{\beta_{s1}}}{(P_{R1}+\eta_sD_n)^{\beta_{s2}}}\right]^{\frac{1}{\Delta\beta}} \cdots\cdots\cdots (16)$$

$$\Omega_s=-\frac{1}{\Delta\beta}\ln\left(\frac{P_{R1}+\eta_sD_n}{P_{R2}+\eta_sD_n}\right) \cdots\cdots\cdots (17)$$

Table 3 Comparisons with Nowak's evaluation

Approach	β_e	β_s
Nowak	3.80	4.95(ρ=1) 6.20(ρ=0)
Proposed	3.80(Ext.G) 5.23(Int.G)	5.86

Table 4 Bridge data

Bridge	Sang-il	Ham-an
Type	Continuous box with 3 span	Continuous box with 3 span
Span Length	30.3+45.5+30.3 = 116.12 m	29.95+35+29.91 = 95 m
Design Load	DB-24, DB-18	DB-24
Girder Space	6.9m	5.0m
No. of Girder	4	2
$\sigma_{test}/\sigma_{cal}$	0.4826	0.6847
Material Type	SWS 50	SWS 50
Impact Factor	1.250(1,3 span) 1.211(2 span)	1.250(1,3 span) 1.232(2 span)
Damage Factor	1.0	1.0

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

Finally, substituting Eq.(16) and (17) into Eq.(14), P_n may be derived in the following form ;

$$P_n=\frac{(P_{R2}+\eta_sD_n)^{\Delta\beta_1/\Delta\beta}}{(P_{R1}+\eta_sD_n)^{\Delta\beta_2/\Delta\beta}}-\eta_sD_n \cdots\cdots\cdots (18)$$

where P_n=nominal load carrying capacity ; P_{R1}, P_{R2}=upper and lower rating loads, respectively ; $\Delta\beta_1=\beta_{s1}-\beta_{s0}$;$\Delta\beta_2=\beta_{s2}-\beta_{s0}$, in which, β_{s1} and β_{s2} =system reliability indices corresponding to P_{R1} and P_{R2} respectively ; β_{s0}=target reliability.

The equivalent system-rating factor proposed in the paper is a new concept which enables the bridge engineers to easily understand the reserve carrying capacity of a bridge superstructure as system in terms of the maximum system-capacity rating load P_n at SLR ($\beta_{s01}=3.0$) or MOR ($\beta_{s02}=2.5$).

(3) Deterministic System Redundancy and Reserve Strength

Once the ultimate system-capacity in terms of the equivalent system rating load P_n is obtained from Eq.(18) the deterministic measure of system redundancy and reserve strength corresponding to those of probablistic measure may be defined as follows,

$$\mathrm{DSRF}=P_{ns}/P_{ni} \cdots\cdots\cdots (19a)$$

$$\mathrm{DSReF}=P_{ns}/P_{ne} \cdots\cdots\cdots (19b)$$

where DSRF=Deterministic System Redundancy Factor ; DSReF = Deterministic System Reserve Factor ; P_{ns} = nominal load of ultimate system capacity corresponding to system reliability index ; P_{ne}=nominal load of element capacity corresponding to element reliability index ; P_{ni}=nominal load of initial-failure capacity corresponding to reliability index of initial-failure element.

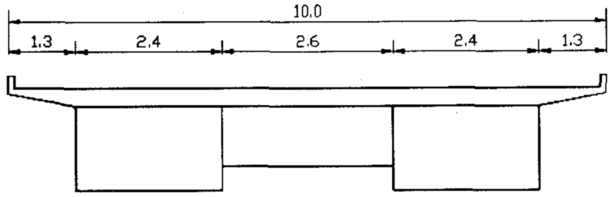

Fig.3 Cross section of Ham-an bridge (unit : m)

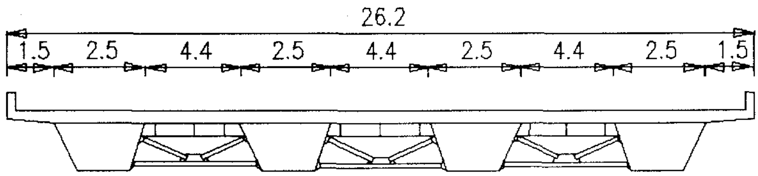

Fig.4 Cross section of Sang-il bridge (unit : m)

Table 5 Reliability of example bridges

Bridge	Element Reliability β_e (P_F)		System Reliability β_s (P_F)		Initial-Failure Reliability β_I (P_F)
	Bending Strength Limit State	Interactive Failure Limit State	Without Cross beam Effect	With Cross beam Effect	
Sang-il (4-Box)	3.11 (0.9354E-3)	2.83 (0.2347E-2)	4.86 (0.5632E-6)	6.73 (0.1023E-10)	2.38 (0.8672E-2)
Ham-an (2-Box)	4.78 (0.9318E-5)	4.22 (0.1222E-4)	5.20 (0.9551E-7)	5.76 (0.4129E- 8)	3.93 (0.4242E-4)

Table 6 Capacity rating of example bridges

Basis-Capacity Rating	Method		Sang-il P_n(t)	Sang-il RF	Ham-an P_n(t)	Ham-an RF
System Reliability -Based	Non-Codified	SLR	140.9	3.26	184.9	4.28
		MOR	229.4	5.31	257.3	5.95
Element Reliability -Based	Non-Codified	SLR	95.5	2.21	147.3	3.41
		MOR	142.2	3.29	182.3	4.21
	LRFR	SLR	109.2	2.53	147.1	3.41
		MOR	155.6	3.60	200.0	4.63
	WSR		150.8	3.49	194.4	4.50
	LFR(AASHTO)	I. R	104.7	2.42	122.1	2.83
		O. R	174.5	4.04	203.5	4.71

Table 7 System redundancy and reserve safety/strength

Bridge		PSRF	PSReF	DERF	DSReF
Sang-il	• Without Cross beam effect	2.04	1.56	2.12	1.61
	• With Cross beam effect	2.83	2.16	3.01	2.03
Ham-an	• Without Cross beam effect	1.32	1.09	1.87	1.41
	• With Cross beam effect	1.47	1.21	2.06	1.61

5. APPLICATION

(1) Applicability of the System Reliability Model

At first, the applicability of the proposed models and methods for system reliability analysis is demonstrated by comparing the results for an illustrative example with those of the Nowak's approach[10]. The example bridge that Nowak used for system reliability evaluation was an 18-m composite steel girder bridge with 5 W 33 × 135 girder spaced at 2.4-m. It may be observed that at the element reliability level, two results are identical as expected, but at the system level, the results of the proposed method fall between the extreme results corresponding to the Nowak's assumed correlation, which apparently indicates the appropriateness of the proposed approach. **Table 3** shows the results of element and system reliabilities evaluated by two approaches.

(2) **Example Bridges**

The modelling and analysis methods for the system reliability proposed in this paper are applied to two existing steel-box girder bridges. Some of measured and analysis results with bridge data are given in **Table 4.**

(3) **System Redundancy and Reserve Safety/Strength**

The results of the system redundancy measures based on the reliability assessment for the example bridges are summarized in **Table 5** and **7**. First of all, in **Table 5** it can be clearly observed that considerable differences exist between the element and system reliabilities in the range of $\Delta\beta=2.03\sim3.9$, and thus, in turn, it results in relatively high values in the measures of both probabilistic and deterministic system redundancy and residual safety/strength up to 2.16/2.83 for PSReF/PSRF, 2.03/3.01 for DSReF/DSRF for Sang-il Bridge. These results apparently indicate that the decks, cross beams and secondary members significantly contribute to the system redundancy. Therefore, it may be argued that system reliability approaches may have to be used for the evaluation of realistic reserve safety and capacity of highly redundant bridges. Next, as seen in **Table 5**, it may be noted that the element reliability based on the interactive failure limit state are significantly lower by 10~12% compared to those based on the bending strength limit state.

Furthermore, it can be noted in **Table 7** that the effect of the number of girders on the system redundancy in the case of Sang-il bridge with 4 box-girders renders a lot more significant results with 1.56 for PSRF and 3.01 for DSRF in the case of Ham-an bridge with 2 box-girders. Also it can be seen from **Table 5** that the system reliabilities based on the failure mechanism considering the effect of the cross beams are about 10~28% higher than those without considering it. Again, it may be empahsized that the cross beams significantly contribute to the collapse strength of the structure.

(4) **Rating of Example Bridges**

The results of capacity rating of the example bridges are summarized in **Table 6**. As it has been already shown in the previous section with the discussion of the system reliability and redundancy, it can also be observed that the non-codified capacity ratings based on system reliability is a lot higher by 20~30% than non-codified or LRFR rating (with RF=2.21~3.41 for SLR, 3.29~4.21 for MOR) based on the element reliability. It is also interesting to observe that the results of non-codified and LRFR capacity ratings are about similar. However, it can be seen that, at the element level, the conventional WSR provides unreasonably higher results (with RF=3.49~4.50) than the reliability-based capacity rating, whereas the ASHTO-LFR rating gives almost similar results at the element level (with RF=2.42~2.83).

Based on these comparative observations, it may be suggested that the reliability and capacity-rating evaluated at the system level are significantly different from those evaluated at the element level, especially in the case of highly redundant box-girder bridges. Finally, it may be stated that the non-codified equivalent system-capacity formula derived on the basis of system reliabilities proposed in the paper can be successfully applied to assess the system-redundancy or reserved system-capacity of existing bridges in practice.

6. CONCLUSION

Because of the redundancy and effective load sharing provided by the decks and cross beams in composite steel box-girder bridges, system reliability is significantly defferent from element reliability. And thus it provides a more precise measure for the system safety, redundancy and capacity rating of existing steel box-girder bridges.

The equivalent system reliability-based capacity rating proposed in the paper can be effectively used in practice for the rating of deteriorated or damaged bridges to pick up reserve system-safety or strength for the optimal decisions in maintenance and rehabilitation.

In the case of composite steel box-girder bridges, the interactive failure limit state provides more critical element reliability than bending strength limit state.

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合成鋼箱子桁道路橋の體系信頼性解析および安全度評価

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本論文では，各種合成鋼箱子桁道路橋に対する體系信頼性解析およびここに基づいた耐荷力評価を行うために，実用的で合理的な信頼性模型と解析方法を提案している．既設橋梁に対する耐荷力および安全度評価の結果，全體システムとしての保有安全度と耐荷力は個別要素の場合とくらべると橋梁の不定静性が高いほど，その差が大きくなることが分かった．したがって體系信頼性に基づく提案した耐荷力評価方法は，既設合成鋼箱子桁橋の実際的な耐荷力評価のために合理的に使用できると思われる．