# SYSTEM RELIABILITY ASSESSMENT OF STEEL GIRDER BRIDGES

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## Abstract

Reliability can be considered as a rational evaluation criterion in assessment of bridge structures. The traditional element-based approach to bridge design and evaluation does not allow for consideration of interaction between the components that form a structural system and, therefore, it can be conservative. Safety of the structural system also depends on the degree of redundancy (load sharing) and ductility. As a result, it has been observed that the load carrying capacity of the whole structure can be much larger than what is determined by the design of individual components. Therefore, this paper is focused on the system behavior. The objective is to formulate a limit state function for the whole bridge, identify the load and resistance parameters, and develop an analysis procedure to assess the reliability of the bridge as a structural system. The major steps of the procedure include selection of representative structures, formulation of limit state functions, development of load and resistance models, development of the reliability analysis method, reliability analysis of selected bridges, and formulation of recommendations for practical bridge assessment. The live load is considered in form of a design truck. The analysis is performed for different values of span length, truck position (transverse and longitudinal), number of vehicles on the bridge (multiple presence), girder spacing, and stiffness of structural members (slab and girders). For each combination of these parameters, the bridge resistance is determined in terms of the weight of a truck (or trucks) causing an unacceptable deflection or instability of the considered bridge. The reliabilities are also calculated for individual components (girders) and compared to system reliabilities of the bridge. The resulting system reliability can serve as a tool in the development of a rational bridge design and evaluation procedure.

**Keywords:** Reliability index, System reliability, Steel girder bridges, Loads, Resistance

## Introduction

There is a growing interest in the development and improvement of efficient procedures for the design, evaluation, repair and rehabilitation of bridge structures. Structural reliability can be considered as a rational measure of performance, as it is a function of uncertainties associated with loads and load carrying capacity (resistance). A traditional deterministic approach is based on the analysis of individual components. The capacity of the structure is reached when any of the components reaches its ultimate capacity. However, in practice, due to ductility and redundancy, there can be redistribution of load resulting in an increased load carrying capacity. The margin safety can be assessed more accurately using the system reliability approach.

Therefore, this paper deals with calculation of the reliability for the whole bridge, rather than individual components (girders). The main objective is to compare the reliability of components (girders) and system for steel girder bridges. The limit state function is formulated for the whole bridge, load and resistance parameters are identified, and the reliability analysis procedure is developed. The developed methodology is demonstrated on representative steel girder bridges. The recommendations are formulated for a practical bridge assessment.



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The ultimate limit states (moment and shear carrying capacities) are considered. The structural performance is determined as a function of load components, strength of material, and dimensions. The reliability analysis is performed for different values of span length, truck position (transverse and longitudinal), number of vehicles on the bridge (multiple presence), girder spacing, and stiffness of structural members (slab and girders). For each combination of these parameters, the bridge resistance (system resistance) is determined in terms of the gross vehicle weight, GVW, of two side-by-side trucks, with axle configuration of the design truck in AASHTO (2004). It is assumed that the ultimate capacity (maximum GVW) is reached when the resulting deflection exceeds the critical limit. The analysis is performed separately for each transverse position of two trucks.

**Structural Analysis Model**

The structural analysis is carried out using a nonlinear model. This allows for stress and load redistribution that can occur as the structure passes from an initial localized yielding in a member, to reaching the ultimate capacity of that member, and finally to a total bridge collapse. Moreover, the use of inelastic analysis allows to consider material nonlinearity, geometric nonlinearity and also boundary nonlinearity. In this study, the analysis was performed using ABAQUS, a commercially available finite element program. Material and other structural parameters are based on the collected information from the literature supplemented with engineering judgment.

For the purpose of finite element analysis, the geometry of bridge superstructure can be idealized in many different ways. For this study, a three-dimensional finite element method was applied to investigate the structural behavior of composite bridges. A concrete slab is modeled using isotropic, eight node solid elements, with three degrees of freedom at each node. The girder flanges and webs are modeled using three-dimensional, quadrilateral, four node shell elements with six degrees of freedom at each node. The reinforcement is represented by uniformly distributed layers of steel. The model assumes a complete connection between girders and concrete slab with no slip. Secondary elements were excluded from the analysis. The details of the model are given by Czarnecki and Nowak (2005a-c). An example of an FEM mesh for a bridge with four girders spaced at 3 m is shown in Figure 1.

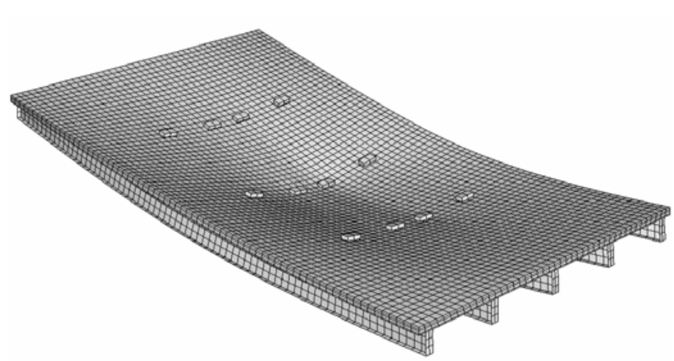

**Figure 1 Deflected Shape, Finite Element Bridge Model**

All investigated structures were designed as simply supported composite bridges in accordance with AASHTO LRFD (2004) Strength I limit state for flexure and shear. Span lengths ranging from 12 m to 42 m, with the intervals of 6 m, are considered. For each of the span lengths, three girder spacings are investigated: 1.8, 2.4, and 3 m. For all considered bridges, the longitudinal axis is assumed at right angle to the abutment. All bridges are designed as two-lane structures, and with deck slab thickness of 225 mm. Previous studies showed (Eamon and Nowak

2002 and 2004) that the effect of diaphragms on ultimate moment capacity is insignificant, therefore, the diaphragms are not considered in this study.

In the analysis, the load was applied in form of two side-by-side design trucks (AASHTO 2004). In the longitudinal direction, the trucks were positioned to generate the maximum bending moment. Different transverse positions were considered but the centerlines of the wheels of two adjacent trucks were placed no closer than 1.2 m.

**Structural Reliability Analysis and Limit State Functions**

The available reliability methods are presented in several publications (e.g. Nowak and Collins 2000) and, therefore, the details are not discussed in this paper. The methods vary with regard to accuracy, required input data, and computational effort. Most of the available procedures are suitable for a reliability analysis for individual components rather than structural systems. System reliability methods are more complex but they offer considerable advantages, and therefore they are the subject of the present paper.

In case of the component reliability, it is almost always possible to define a performance function g(X) of the basic random variables X such that $g > 0$ corresponds to a satisfactory performance and $g < 0$ corresponds to failure of a structure. On the other hand, the formulation of limit state function for the entire bridge is much more complex and requires a special approach. This is because of, among other things, the possible statistical dependence among the random variables, load redistribution after some members' failure, redundancy of the structure that is causing a load sharing.

So far, a number of approaches have been proposed that allow to define a limit state function, and consequently failure, for entire bridge. Zhou (1987) proposed that system failure occurs when two adjacent girders fail. Tabs and Nowak (1991) considered several girders must reach their ultimate capacity before the structure collapses. Ghosn and Moses (1998) defined the bridge resistance as the maximum gross vehicle load that is causing the formation of a collapse mechanism. Enright and Frangopol (1999) studied a number of system models for a five girder bridge. Estes and Frangopol (1999) assumed that failure occurs when three adjacent girders out of five fail. Liu and Moses (2001) considered a damaged steel girder bridge. It was assumed that damage can be caused by corrosion, collision, etc.

In this paper, however, it is assumed that the failure of a bridge is defined as a maximum load that the bridge can carry, or as 0.0075 of the span length deflection in any of the main members of the bridge, whichever governs. The deflection is calculated only due to live load, including static and dynamic components.

**Bridge Load Models**

The major load components for highway bridges are deal load, live load, dynamic load, environmental loads (temperature, wind, earthquake), and other loads (collision, braking). In this paper, however, only the first three are considered. Consideration of live load involves not only the weight of trucks, but also the distribution factor (fraction of the total truck load per girder), and truck position within the roadway (curb distance). The load models are based on the available statistical data, surveys, inspection reports, and analytical simulations. The load variation is described by cumulative distribution function, mean value or bias factor (ratio of mean to nominal value), and coefficient of variation.

Dead load, DL, is the gravity load due to self-weight of the structural and nonstructural components permanently attached to the bridge. Therefore, it includes the weight of girders, deck slab, wearing surface, barriers, sidewalks, and diaphragms, when applicable. The statistical parameters for dead load were selected from the available literature (Nowak 1999). Four components are considered: $DL_1$ – weight of factory made elements, $DL_2$ – weigh of cast-in-place concrete, $DL_3$ – weight of wearing surface (asphalt), and $DL_4$ – weight of miscellaneous items (e.g., railing, luminaries). All components of dead load are treated as normal random variables. For $DL_1$, the bias factor, $\lambda = 1.03$, and coefficient of variation, $V = 0.08$; for $DL_2$, $\lambda = 1.05$, and $V = 0.10$; for $DL_4$, $\lambda = 1.03\sim1.05$, and $V = 0.08\sim0.10$,; and for asphalt wearing surface it is assumed that the mean thickness is 75 mm and $V = 0.25$.

The live load model was developed in conjunction with calibration of the AASHTO LRFD Code (Nowak and Hong 1991; Nowak 1993). The statistical parameters (mean values, bias factors and coefficients of variation) are derived for the maximum lane moments and shears. The multiple presence of trucks is considered by using the observed frequencies of occurrence of two vehicles in the same lane or side-by-side. For a single-lane loaded case, the ratio of the mean maximum 75-year moment to AASHTO HL-93 design moment varies from 1.3 for shorter spans (10 m) to 1.2 for longer spans (50 m), while coefficient of variation, $V = 0.11$ for all spans. For the two-lane loaded case, bias factor for each truck varies from 1.2 for shorter spans (10 m) to 1.0 for longer spans (50 m), while coefficient of variation, $V = 0.11$ for all spans.

The basic load combination includes dead load and live load (static and dynamic). Live load is represented in form of a design truck as shown in Figure 2. It is assumed that the gross vehicle weight (GVW) is a random variable, but the axle spacing and percentage of the total load per axle remain constant. The transverse position of the truck within the roadway (curb distance) is also treated as a random variable. An example of the probability density function (PDF) of the curb distance is shown in Figure 3, for two traffic lanes. Each curve represents a curb distance for a line of wheels, spaced at 1.8m for a truck.

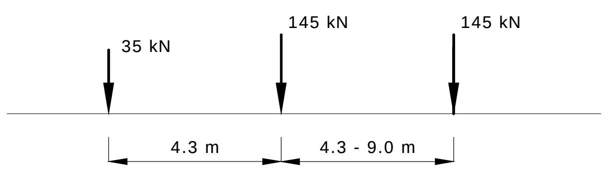


**Figure 2 Design Truck**

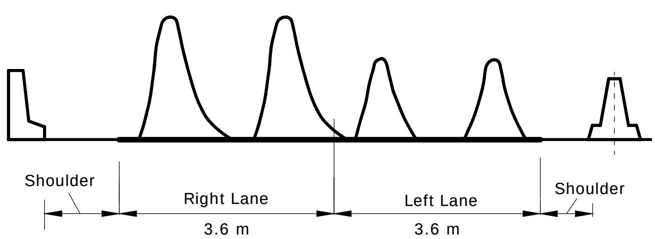


**Figure 3 Probability Density Functions (PDF) of the Curb Distance. Each PDF Represents a Line of Truck Wheels (Tantawi 1986)**

The variation in transverse traffic position is based on a survey on interstate highways in Southeastern Michigan. The PDF was approximated by a lognormal distribution with a coefficient of variation of 0.33. For a standard lane width of 3.63 m, the mean value of the distance from the lane edge to the centerline of the outermost vehicle wheel is equal to 0.91 m.

Dynamic load depends on roughness of the surface, dynamic properties of the bridge, and suspension system of the vehicle. Dynamic load factor is defined as the ratio of dynamic strain (or deflection) and static strain (deflection). Field tests conducted by Kim and Nowak (1997) and

Eom and Nowak (2001) showed that the dynamic load factor does not exceed 0.15 for a single truck and 0.10 for two heavily loaded trucks traveling side-by-side. Therefore, the mean dynamic load factor is conservatively taken as 0.10 with the coefficient of variation of 0.80.

**Bridge Resistance Models**

The load carrying capacity of a structure depends on its geometry (number of girders, girder spacing, and span length), connections, and most of all, the resistance of its components. Moreover, the resistance can be affected by uncertainties in strength of materials, dimensions and analysis. Therefore, the resistance of girders, $R_{girder}$, can be considered as a product of three factors representing strength of materials, dimensions and analysis, and consequently, as a product of random variables, it can be considered as lognormally distributed.

***Girder resistance***

The reliability analysis is performed for 54 composite girders. The statistical parameters of the load carrying capacity (resistance) for composite steel girders were derived by Nowak (1999). Ultimate moment and shear limit state equations are considered for interior girders. For moment carrying capacity, the bias factor and coefficient of variation are 1.12 and 0.10, respectively. For shear capacity, the bias factor and coefficient of variation are 1.14 and 0.105, respectively.

***Bridge (system) resistance***

The design of bridges is based on consideration of individual components, therefore, the performance of the whole structure can be underestimated because it does not account for redundancy and ductility. Failure of a component does not necessarily mean failure of the entire bridge. Therefore, bridge safety can be determined using system reliability approach that includes multiple failure path, load sharing and load redistribution after member failure. Consequently, the system reliability can be considered as a more accurate measure of safety. However, the system reliability computations are more difficult than the reliability analysis of a component, because there are many additional parameters.

The need for the system reliability analysis of bridge structures has long been recognized. There are many different modes of system failure. In this paper, the system resistance is considered in terms of the deflection of the main girders caused by live load. It is assumed that the ultimate limit state is reached when the maximum deflection of any girder exceeds 0.0075 of the span length.

The system resistance is considered in terms of the gross vehicle weight (GVW) of two side-by-side trucks with axle configuration of the design truck (AASHTO 2004), as shown in Figure 2. The calculations are performed for various transverse positions of the vehicles (within the roadway width). The incremental loading method is used. For each transverse truck position, the GVW is gradually increased until the deflection of one of the girders exceeds 0.0075 of the span length. The system resistance is defined as the GVW corresponding to this critical deflection. It can be different depending on the position of trucks within the roadway width. Each transverse position is associated with a certain probability of occurrence. Therefore, the system resistance, $R_{system}$, is equal to the expected value of the GVW, calculated for different transverse positions:

$$R_{system} = \sum_{i=1}^{n} p_i \cdot GVW_i \tag{1}$$

where $GVW_i$ = gross vehicle weight of two trucks side-by-side, causing deflection equal to 0.0075 of the span length corresponding to i-th transverse position of trucks, $p_i$ = probability of trucks occurring in the i-th position. Figure 4 shows a typical cross section of a 6-girder bridge and the truck positions considered in the system resistance analysis.

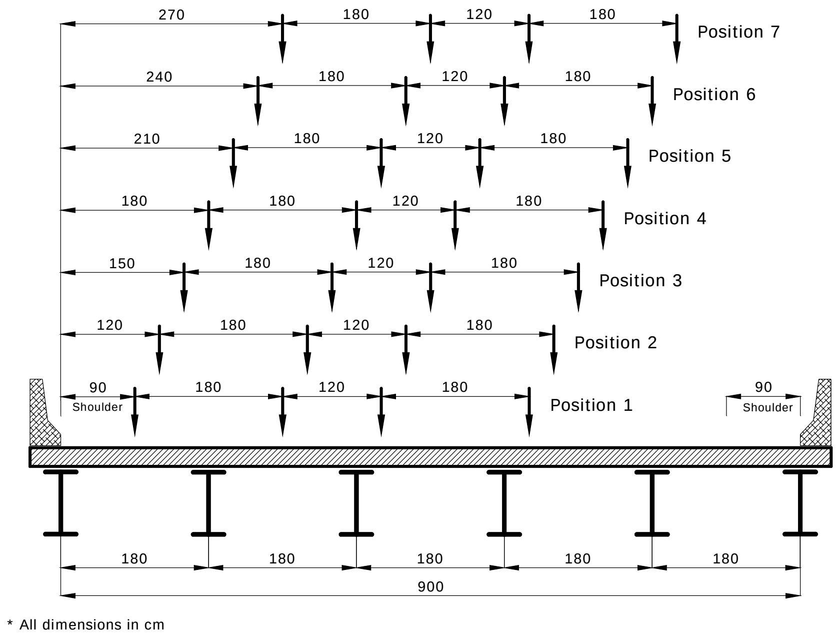


**Figure 4 Truck Positions Considered in the System Resistance Analysis**

An example of the deterministic load-deflection curve for a 30 m span composite steel girder bridge is shown in Figure 5.

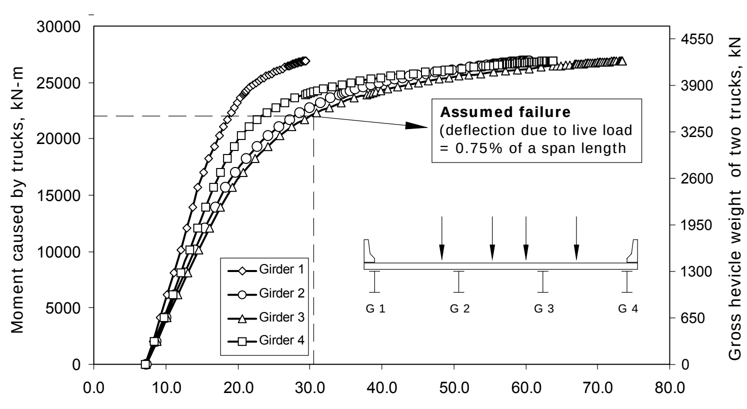


**Figure 5 Deterministic Bridge Load-Deflection Relationships for Span Length of 30 m**

**Reliability Analysis for Selected Bridges**

The reliability indices for individual girders were calculated using the procedure developed for calibration of the AASHTO LRFD Code (Nowak 1999). For each combination of span length and girder spacing, three design cases were considered: (1) with individual girders designed according to the code and thus providing reliability indices close to the target $\beta_T$ = 3.5, (2) with under-designed girders, with the girder reliability indices close to $\beta_T$ = 2.0, and (3) with over-designed girders and girder reliability indices close to $\beta_T$ = 4.5. The ratio of the actual girder resistance and the minimum required resistance for the target β, is shown in Figures 6 and 7 for $\beta_T$ = 3.5, for moments and shear forces, respectively. The ratios for shear are very large because the design is governed by the moment capacity, so the girders are over-designed with regard to shear. Therefore, the shear capacity was not considered in the system reliability analysis.

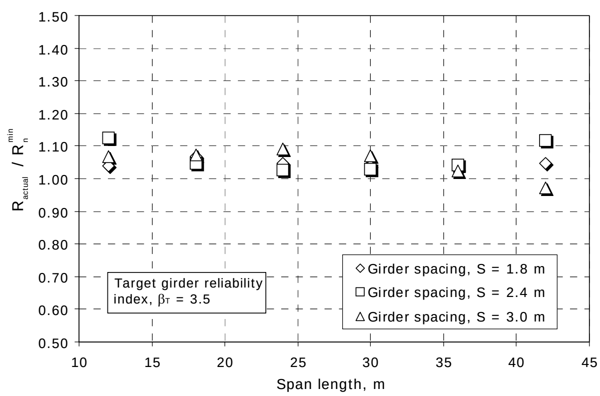


**Figure 6 Ratios of Actual Resistance to Minimum Required Resistance for Target Girder Reliability Index $\beta_T$ = 3.5, Moments**

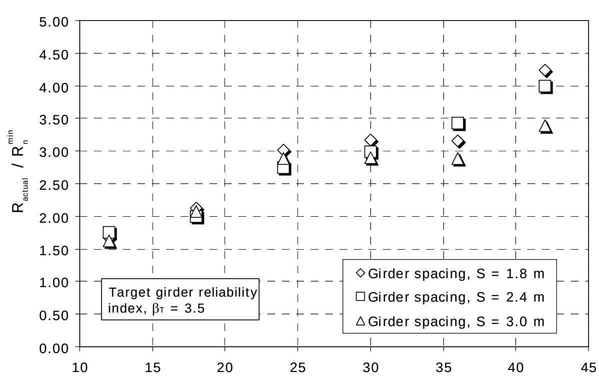


**Figure 7 Ratios of Actual Resistance to Minimum Required Resistance for Target Girder Reliability Index $\beta_T$ = 3.5, Shear Forces**

In the system reliability analysis, the two main random variables are bridge system resistance and bridge live load. Both are represented in form of the gross vehicle weight (GVW) of two identical trucks placed side-by-side, each truck with axle configuration of the design truck

(AASHTO 2004). The system reliability index was calculated for various truck positions within the roadway width, and the final system reliability index was determined as an expected value using the weighting factors based on the curb distance distributions (Figure 3).

Figures 8a to 8c show the relationship between the girder reliability index and the corresponding system reliability index for three different girder spacings, and span length of 18, 30, and 42 m, respectively.

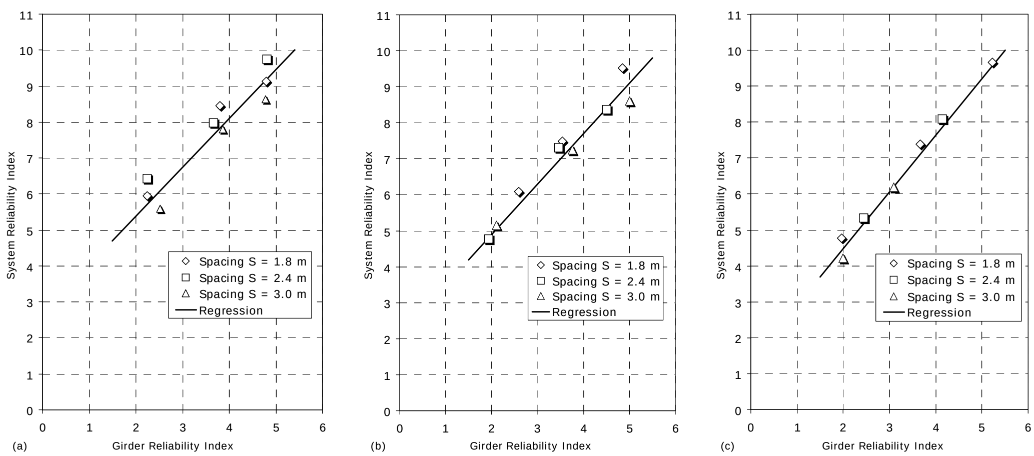


**Figure 8 Girder Reliability Index versus System Reliability Index for Different Girder Spacings and Span Length of (a) 18 m, (b) 30 m, and (c) 42 m**

The results indicate that the ratio of system reliability to girder reliability decreases with increasing girder reliability. This is because for over-designed bridges ($\beta_T$ = 4.5), the ratio of system capacity to girder capacity is smaller than that for under-designed bridges ($\beta_T$ = 2.0). Moreover, the system reliability indices increase with number of girders. This is due to the increased redundancy of the system. It is observed that not only does the ratio of system reliability to girder reliability decreases with increasing girder reliability, but it also decreases with the increase of span length or girder spacing.

Figures 9a to 9c show the girder and system reliability indices as a function of span length, for girder spacing of 3.0 m, for the three considered cases of the target girder reliability index, $\beta_T$, of 2.0, 3.5, and 4.5. Also shown are wide flange sizes for the selected main girders.

The next step was to investigate the effect of correlation between the resistances of individual girders represented by the yield stress, $F_y$, of structural steel. Four cases are considered: (1) no correlation, with coefficient of correlation $\rho = 0$, (2) full correlation with $\rho = 1$, (3) and (4) partial correlation, with different values of $\rho$ depending on the number of girders. The correlation between girder strengths resulted in a reduction of the number of different random variables considered in system reliability analysis.

For a 6-girder bridge with girder spacing of 1.8 m, it was assumed that a partial correlation of $\rho = 0.33$ can be represented by the case of two adjacent girders having identical strength, and $\rho = 0.66$ is represented by four adjacent girders being identical. In addition, also considered were cases with randomly distributed correlated girders (two or four out of six). A similar approach was used in case of 5-girder bridges, with girder spacing of 2.4 m. However, as this time bridges had only 5 girders, so the global correlation between the strength of girders was $\rho = 0.40$ and $\rho = 0.60$ for cases when two and four girders were correlated, respectively. For 4-girder bridges, with girder spacing of 3.0 m, it was assumed that two or three girders were correlated resulting in the global correlation between the strength of girders of $\rho = 0.50$ and $\rho = 0.75$ for cases when two and three girders were correlated, respectively.

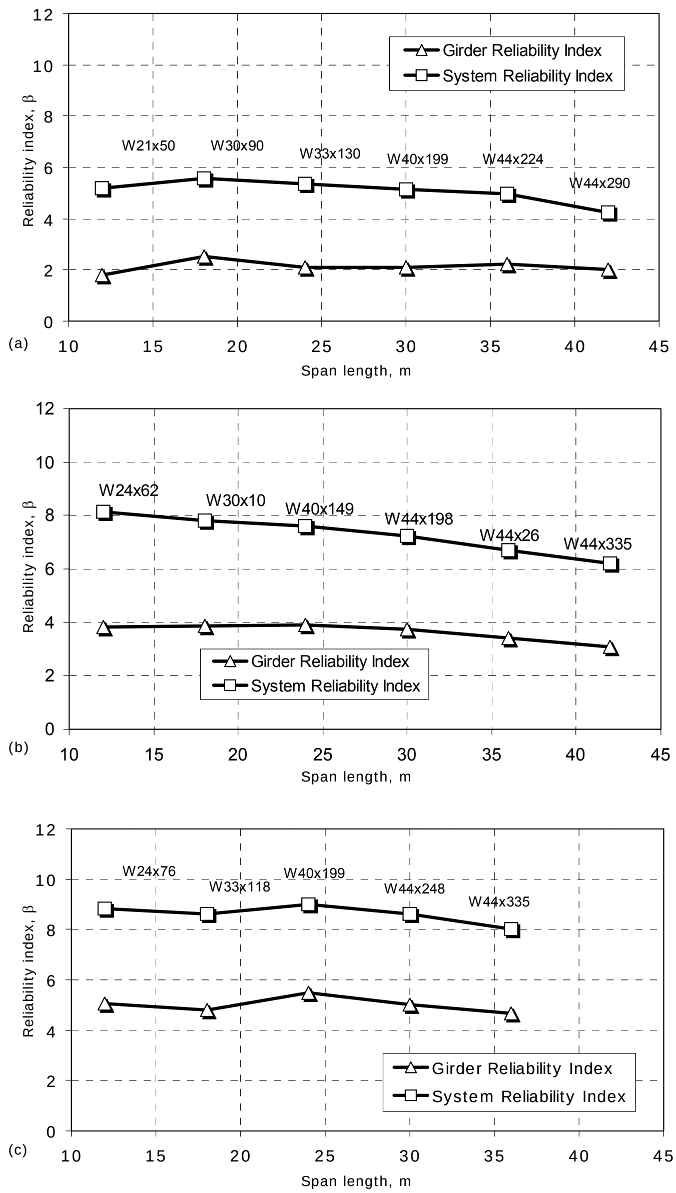


**Figure 9 Girder Reliability Indices and Corresponding System Reliability Indices for Different Spans and Girder Spacing of 2.4 m, Target Girder Reliability Index of (a) $\beta_T$=2.0, (b) $\beta_T$=3.5, and (c) $\beta_T$=4.5**

Examples of the effect of correlation on the system reliability indices for composite steel girder bridges are shown in Figure 10. The target reliability index for the girders is $\beta_T = 3.5$.

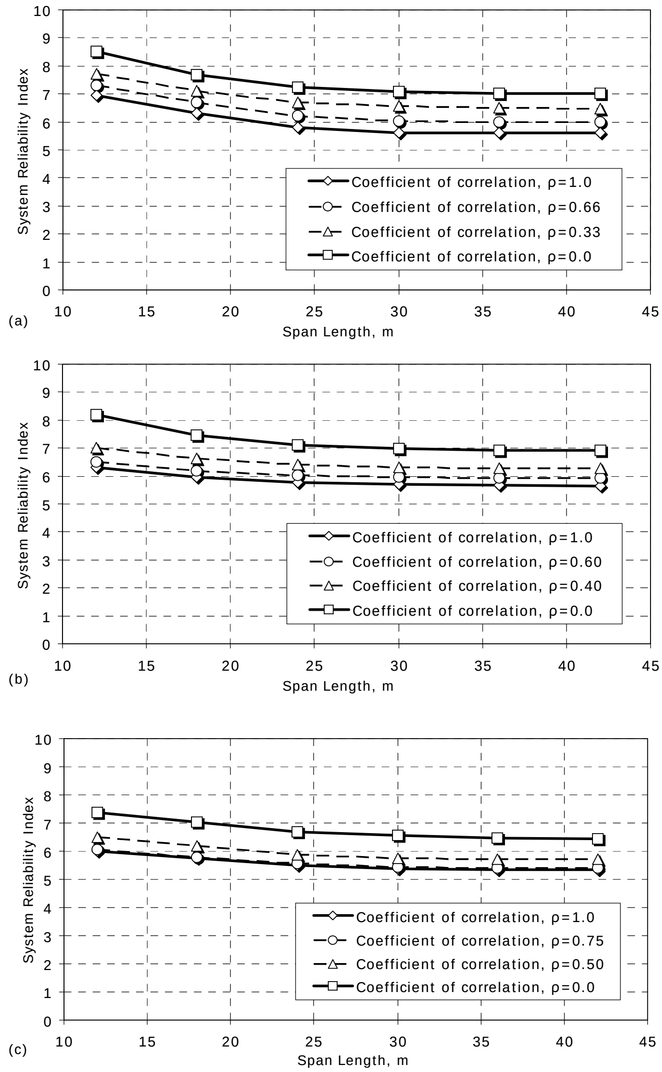


**Figure 10 System Reliability Index for Different Degree of Correlation between Girder Resistance and Different Span Length, Girder Spacing of (a) 1.8 m, (b) 2.4 m, and (c) 3.0 m**

## Conclusions

This paper shows that reliability can be considered as a rational measure of structural performance in the design of bridges and assessment of exiting bridges. The reliability analysis procedure is demonstrated on a representative sample of composite steel girder bridges. Load and resistance parameters are identified, limit state function for the whole bridge was formulated, and reliability analysis is performed for girders and structural systems. It was found that the system reliability is considerably higher than the girder reliability, in particular in case of uncorrelated girder resistances. The difference between girder reliability and system reliability can be considered as a measure of bridge redundancy. The calculated reliability indices for the whole bridge, $\beta_{system}$, are compared with the reliability indices determined for individual girders, $\beta_{girder}$. It was observed that the ratio of $\beta_{system}$ / $\beta_{girder}$ decreases with increasing $\beta_{girder}$. It ranges from 1.6 for $\beta_{girder}$ = 4.5 to 3.1 for $\beta_{girder}$ = 2.0. It was also found that the correlation between girder resistances plays an important role and it can decrease $\beta_{system}$ by 10 to 30% depending on the girder reliability, span length, and girder spacing. The paper shows that much can be gained by considering a bridge structure as a system.

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