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Target Safety Levels for Design and Evaluation of Bridges †

Andrzej S. Nowak* and Maria Kaszynska**

Abstract

The development of limit state codes and/or load and resistance factor design (LRFD) codes for the design and evaluation of bridges requires the knowledge of the target reliability level. The optimum safety level depends on the consequences of failure and cost of safety. Selection of the target reliability can be based on consideration of these two parameters. In this paper, the analysis is performed for ultimate limit states (ULS) and serviceability limit states (SLS). Primary and secondary components are considered. Serviceability limit states have a lower level of consequences of failure. Therefore, lower values of the target reliability index are selected for SLS than ULS.

KEY WORDS: (Bridges), (Reliability Index), (Ultimate Limit States), (Serviceability Limit States), (Code Calibration)

1. Introduction

The new generation of bridge design codes is based on the reliability analysis performed using statistical models of load and resistance. The major steps in the development of the code include the selection of representative structures, formulation of limit states, development of load and resistance models, selection of the target reliability level, and finally the selection of load and resistance factors based on closeness to the target reliability. Reliability index, β, is an efficient measure of structural performance. The available methods for calculation of β are presented in the textbooks, e.g. Ref.1). The present paper deals with the selection criteria for the target β.

The reliability analysis procedures can be used for a comparison of different variants of design alternatives, materials and types of structure. Optimum safety level can also be expressed in terms of the target reliability index. The development of load and resistance factor design (LRFD) codes for the design of bridges requires the knowledge of the target reliability level. The optimum safety level depends on the consequences of failure and cost of safety. Selection of the target value can be based on consideration of these two parameters. Target reliability indices calculated for newly designed bridges and existing structures are different for many reasons. Reference time period is different for newly designed and existing bridges. New structures are designed for 50-75 year life time and existing bridges are checked for 5 or 10 year periods. Load model, used to calculate reliability index depends on the reference time period. Maximum moments and shears are smaller for 5 or 10 year periods than for 50-75 year life time. However, the coefficient of variation is larger for shorter periods. Single load path components require a different treatment than multiple load path components. In new designs, single load path components are avoided, but such components can be found in some existing bridges. Target reliability index is higher for single load path components.

Reliability indices calculated for existing bridges can be considered as the lower bounds of safety levels acceptable by the society. A drastic departure from these acceptable limits should be based on an economic analysis. The target reliability index depends on costs and has different value for a newly designed bridge and an existing one. In general, it is less expensive to provide an increased safety level in a newly designed structure. For bridges evaluated for 5 or 10 year periods (intervals between inspections), it is assumed that inspections help to reduce the uncertainty about the resistance and load parameters. Therefore, the reliability index can be lower for existing bridges evaluated for 5 or 10 year periods. Optimum safety can be determined by minimization of the total expected cost (or maximization of the utility). The optimum safety level corresponds to the minimum total expected cost. The total cost includes the cost of investment (design and construction) and the expected cost of failure. The cost of failure includes not only the cost of repair or replacement but also the cost of interruption of use, and legal costs (liability in case of injuries). Because of economical reasons, it is convenient

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to differentiate between primary and secondary components in bridges. The difference between these components depends on the consequences of failure. Target reliability index for secondary components is lower than that for primary components.

The analysis is performed for the ultimate limit states (ULS) and serviceability limit states (SLS). Serviceability limit states have a lower level of consequences of failure. Therefore, lower values of the target reliability index are selected for SLS than ULS. For the ultimate limit states, calculated reliability indices represent component reliability rather than system reliability. The reliability indices calculated for structural system are larger than for individual components by about 2. Therefore, selection of the target reliability level should be based on consideration of the system. Then, target reliability index for components can be derived using the appropriate formulas. For serviceability limit states, reliability indices vary considerably depending on the limit state. For example, the consequences of exceeding the tension stress limit in concrete girders are much l ess severe compared to the ULS.

2. Bridge Load Models

Statistical models of load and resistance for highway bridges are described in Ref. 2), 3), 4), 5). The available parameters can be used to determine the reliability indices for structures designed according to the current code provisions. Load and resistance models for highway bridges indicate a considerable degree of variation (large scatter). The basic parameters that describe each load component are bias factor (ratio of the mean-to-nominal values), λ, and coefficient of variation, V.

The main load combination includes dead load, live load and dynamic load. Dead load is the gravity load due to the self weight of the structural and non structural elements permanently connected to the bridge. Because of different degrees of variation, it is convenient to consider three components of dead load: weight of factory made elements (steel, precast concrete members), weight of cast-in-place concrete members, and weight of the wearing surface (asphalt). All components of dead load are treated as normal random variables. The statistical parameters were derived in conjunction with the revision of the AASHTO Standard Specifications[6] and the development of the new AASHTO LRFD Code[7]. The parameters of dead load are λ = 1.03-1.05, and V = 0.08-0.10, with the lower values applicable to structural steel and plant-cast concrete, and the upper values applicable to cast-in-place concrete[2), 3)].

Live load covers a range of forces produced by vehicles moving on the bridge. It includes the static and dynamic components. The static live load is considered first. The effect of live load depends on many parameters including the span length, truck weight, axle loads, axle configuration, position of the vehicle on the bridge (transverse and longitudinal), number of vehicles on the bridge (multiple presence), girder spacing, and stiffness of structural members (slab and girders). The effect of these parameters is considered separately. The development of live load model is essential for a rational bridge design and/or evaluation code. The model was based on the results of truck survey Ref. 2), 3), 4). The uncertainties involved in the analysis are due to limitations and biases in the survey. The available data base is small compared to the actual number of heavy vehicles in a 75 year life time. It is also reasonable to expect that some extremely heavy trucks purposefully avoided the weighing stations. A considerable degree of uncertainty is caused by unpredictability of the future trends with regard to configuration of axles and weights.

The data base includes truck configuration (number of axles and axle spacing) and weights (axle loads and gross vehicle weight). For each truck in the survey, bending moments and shear forces were calculated for a wide range of spans. Simple spans and continuous two equal spans are considered. For each span length, a cumulative distribution function (CDF) was determined for the considered internal forces (positive moment, negative moment, and shear). The maximum moments and shears for various time periods were determined by extrapolation. The bias factors in terms of the HS20 live load[6] were between 1.65 and 2.10. The HS20 live load is a three axle vehicle: 45 kN, 145 kN and 145 kN, with axle spacings of 4.3m. For spans longer than about 40m, HS20 consists of a uniformly distributed load of 9.3 kN/m and a concentrated force of 81 kN. The coefficients of variation for the maximum truck moments and shears can be calculated by transformation of CDF. Each function can be raised to a certain power, so that the calculated earlier mean maximum moment (or shear) becomes the mean value after the transformation. The slope of the transformed CDF determines the coefficient of variation. For 75 year maximum values it is 0.11 for most spans. Live load varies depending on time period. For the maximum 5 year moment (or shear), the bias factor is about 5 percent less than the maximum 75 year moment (or shear). The difference between 10 year moment and 50 year moment is about 3 percent.

The maximum one lane moment or shear is caused either by a single truck or two (or more) trucks following behind each other. For a multiple truck occurrence, the important parameters are the headway distance and degree of correlation between truck weights. The maximum one lane effect (moment or shear) is derived by Monte Carlo simulations. The analysis of two lane loading involves the distribution of truck load to girders. For moment in an interior girder, AASHTO code[6] specifies a girder distribution factor (GDF) as a function of girder spacing. The comparison of AASHTO[6] GDF's with the results of the structural analysis is carried out using the finite element method (FEM) indicates that code specified values are overly conservative in most cases, except of short spans and short girder spacing. Recent field tests indicate that the actual GDF's are even smaller than analytical results[8), 9)]. For two lane bridges, the load analysis involves the determination of the load in

each lane and load distribution to girders. The effect of multiple trucks can be calculated by superposition. The maximum effects can be calculated using Monte Carlo simulations[1].

The dynamic load model is a function of three major parameters: road surface roughness, bridge dynamics (frequency of vibration) and vehicle dynamics (suspension system). An analytical procedure was developed by Ref. 5) for simulation of the dynamic load on girder bridges. It was observed that dynamic deflection is almost constant and it does not depend on truck weight. Therefore, the dynamic load, as a fraction of live load, decreases for heavier trucks. For the maximum 75 year values, the corresponding dynamic load does not exceed 0.15 of live load for a single truck and 0.10 of live load for two trucks side-by-side. The coefficient of variation of dynamic load is about 0.80. The results of the simulations indicate that DLF values are almost equally dependent on road surface roughness, bridge dynamics and vehicle dynamics. The actual contribution of these three parameters varies from site to site and it is very difficult to predict. Field tests were performed to verify the analytical results[8)-10)]. The results clearly indicate that the dynamic load factor (DLF) decreases for heavier trucks.

3. Bridge Resistance Models

The capacity of a bridge depends on the resistance of its components and connections. The component resistance, R, is determined mostly by material strength and dimensions. R is a random variable. The causes of uncertainty can be put into three categories: (1) material factor including strength of material, modulus of elasticity, cracking stress, and chemical composition, (2) fabrication factor including geometry, dimensions, and section modulus, and (3) analysis factor including approximate method of analysis, idealized stress and strain distribution model. The resulting variation of resistance has been modeled by tests, observations of existing structures and by engineering judgment. The information is available for the basic structural materials and components. However, bridge members are often made of several materials (composite members) which require special methods of analysis. Verification of the analytical model may be very expensive because of the large size of bridge members. Therefore, the resistance models are developed using the available material test data and by numerical simulations.

Therefore, R is considered as a product of the nominal resistance, R_n and three parameters: strength of material, M, fabrication (dimensions) factor, F, and analysis (professional) factor, P,

$$R = R_n M F P \qquad (1)$$

the mean value of R, $m_R = R_n m_M m_F m_P$ and coefficient of variation, $V_R = (V_M^2 + V_F^2 + V_P^2)^{1/2}$, where, m_M, m_F, and m_P are the means of M, F, and P, and V_M, V_F, and V_P are the coefficients of variation of M, F, and P, respectively. The statistical parameters are developed for steel girders, composite and non-composite, reinforced concrete T-beams, and prestressed concrete AASHTO-type girders[3), 11), 12)].

The behavior of non-composite steel girders depends on the strength of steel (F_y), and on compactness of the section. From simulations, the mean-to-nominal ratio (bias factor) and coefficient of variation of non-compact sections are $\lambda = 1.075$ and $V = 0.10$. For compact sections they are 1.085 and 0.10, respectively. However, the steel industry (American Iron and Steel Institute) provided test data which were used to revise the statistical parameters. On the basis of this data, the observed bias factor is assumed $\lambda = 1.095$ and the coefficient of variation is $V = 0.075$. The parameters of the professional factor, P, are: $\lambda = 1.02$ and $V = 0.06$. Therefore, for the resistance, R, the parameters are $\lambda_R = 1.12$ and $V_R = 0.10$. For composite steel concrete girders, the moment-curvature relationships were simulated for typical sections with a concrete slab width considered of 1.8 m, and the slab thickness of 175 mm. The analysis showed that for MF, the bias factor, $\lambda = 1.06$ and $V = 0.105$. Based on the data from the American Iron and Steel Institute, the statistical parameters are $\lambda = 1.07$, and $V = 0.08$. For the analysis factor, P, $\lambda = 1.05$ and $V = 0.06$. Hence for the ultimate moment, $\lambda = 1.12$ and $V = 0.10$. For shear, the statistical parameters of MF were obtained by simulations; mean-to-nominal, $\lambda = 1.11$, and $V = 0.10$. However, using the recent test data provided by the American Iron and Steel Institute, the statistical parameters are $\lambda = 1.12$, and $V = 0.08$. The parameters for the analysis factor are taken as $\lambda = 1.02$ and $V = 0.07$. Therefore the resulting parameters of R are $\lambda_R = 1.14$ and $V_R = 0.105$.

For reinforced concrete T-beams, the moment-curvature relationships are developed for typical bridge T-beams. The major parameters which determine the structural performance include the amount of reinforcement, steel yield stress and concrete strength. The parameters of MF for lightly reinforced concrete T-beams are $\lambda = 1.12$ and $V = 0.12$ (the mean-to-nominal and coefficient of variation). The parameters for analysis factors are $\lambda = 1.00$ and $V = 0.06$. Therefore, for R the parameters are $\lambda_R = 1.12$ and $V_R = 0.135$. The relationship between shear force and shear strain is established for representative T-beams. The nominal (design) value of shear capacity is calculated according to current AASHTO[6)]. The parameters of the shear capacity, V_n, depend on the amount of shear reinforcement. If shear reinforcement is used, $\lambda = 1.13$ and $V = 0.12$. For the analysis factor, P, $\lambda = 1.075$ and the coefficient of variation is $V = 0.10$. Therefore, for the shear resistance, $\lambda_R = 1.20$ and $V_R = 0.155$. If no shear reinforcement is used, then $\lambda_R = 1.40$ and $V_R = 0.17$.

For prestressed concrete, the moment-curvature relationships are developed for typical AASHTO girders. The results show that the bias factor for the ultimate moment is 1.04 and the coefficient of variation is about 0.045. The coefficient of variation is very small because all sections are under-reinforced and the ultimate moment is controlled by the prestressing tendons. For the analysis factor bias, λ = 1.01 and V = 0.06. Therefore, the bias factor for R, λ_R = 1.05 and V_R = 0.075. The shear capacity of prestressed concrete girders is calculated for typical AASHTO type girders, the parameters of FM are λ = 1.07 and V = 0.10. For P, λ = 1.075 and V = 0.10. Therefore, for the shear resistance, λ_R = 1.15 and V_R = 0.14.

4. Reliability Analysis Procedure

The available reliability methods are presented in several publications, e.g. Ref. 1). In this study the reliability analysis is performed using Rackwitz and Fiessler procedure[1), 3)], Monte Carlo simulations and special sampling techniques[1)]. Limit states are the boundaries between safety and failure. In bridge structures failure is defined as inability to carry traffic. Bridges can fail in many ways, or modes of failure, by cracking, corrosion, excessive deformations, exceeding carrying capacity for shear or bending moment, local or overall buckling, and so on. Some members fail in a brittle manner, some are more ductile. In the traditional approach, each mode of failure is considered separately. Ultimate limit states (ULS) are mostly related to the bending capacity, shear capacity and stability. Serviceability limit states (SLS) are related to gradual deterioration, user's comfort or maintenance costs. The serviceability limit states such as fatigue, cracking, deflection or vibration, often govern the bridge design. The main concern is accumulation of damage caused by repeated applications of load (trucks). Therefore, the model must include the load magnitude and frequency of occurrence, rather than just load magnitude as is the case in the ultimate limit states. For example, in prestressed concrete girders, a crack opening under heavy live load is not a problem in itself. However, a repeated crack opening may allow penetration of moisture and corrosion of the prestressing steel. The critical factors are both magnitude and frequency of load. Other serviceability limit states, vibrations or deflections, are related to bridge user's comfort rather than structural integrity.

A traditional notion of the safety limit is associated with the ultimate limit states. For example, a beam fails if the moment due to loads exceeds the moment carrying capacity. Let R represent the resistance (moment carrying capacity) and Q represent the load effect (total moment applied to the considered beam). Then the corresponding limit state function, g, can be written,

$$g = R - Q \tag{2}$$

if $g > 0$, the structure is safe, otherwise it fails. The probability of failure, P_F, is equal to,

$$P_F = \text{Prob}\,(R - Q < 0) = \text{Prob}\,(g < 0) \tag{3}$$

A direct calculation of P_F may be very difficult, if not impossible. Therefore, it is convenient to measure structural safety in terms of a reliability index. The reliability index, β, is defined as a function of P_F,

$$\beta = -\Phi^{-1}(P_F) \tag{4}$$

where Φ^{-1} = inverse standard normal distribution function. There are various procedures available for calculation of β. These procedures vary with regard to accuracy, required input data and computing costs. In calibration of the AASHTO LRFD Code[7)], the reliability index was calculated from the following formula, with k as a constant, equal to about 1.9[3), 13)],

$$\beta = \frac{R_n \lambda_R (1 - kV_R)[1 - \ln(1 - kV_R)] - m_Q}{\sqrt{[R_n V_R \lambda_R (1 - kV_R)]^2 + \sigma_Q^2}} \tag{5}$$

5. Criteria in the Selection of the Target Reliability Index

The major selection criteria are consequences of failure and cost of increasing reliability (or benefit of decreasing β). However, in practice, it is difficult to obtain the data needed for the derivation of the optimum target reliability index. Therefore, a good reference can be established by consideration of the reliability indices corresponding to the structures designed using an existing code. If there are no reported problems for the considered class of structures, then it can be concluded that the current (existing) code is adequate, and possibly conservative. The minimum calculated value of the reliability index can be taken as the target value.

Special consideration must be given to the cases of single and multiple load path components, primary and secondary components, and duration of the time period.

5.1 Single and Multiple Load Path Components

Reliability indices calculated for elements can serve as a basis for the selection of the target reliability index. Let βe be an element reliability index and βs be the system reliability index. For a single path component, $\beta e = \beta s$. From the system reliability point of view, a multiple path system can be considered as a parallel system of at least two elements. The probability of failure for the element is PFe,

$$P_{Fe} = \Phi(-\beta_e) \tag{6}$$

and the probability of failure for the system, P_{Fs} is,

$$P_{Fs} = \Phi(-\beta_s) \quad (7)$$

For n uncorrelated elements, the relationship between P_{Fe} and P_{Fs} is

$$P_{Fs} = (P_{Fe})^n \quad (8)$$

where n is a number of elements, and

$$\beta_s = -\Phi^{-1}(P_{Fs}) \quad (9)$$

Hence, for n = 2 and β_e = 3.5, the system reliability, β_s = 5.3, when elements are fully uncorrelated. However, the elements are usually partially correlated and therefore, β_s = 3.5 to 5.5, depending on the coefficient of correlation (3.5 for full correlation). The recommended target reliability indices for 5 year evaluations of multiple load path components are β_T = 3.0 and for single load path components β_T = 3.5. The corresponding 50 year values are β_T = 3.5 for multiple path components and β_T = 4.0 for single path components.

5.2 Primary and Secondary Components

A primary component is a main structural element, failure of which causes the collapse of the whole structure. In case of bridges, girders are the primary components. It is assumed that the consequences of failure of primary components are about 10 times larger than those of secondary components. Therefore, the probability of failure of secondary components can be 10 times larger than for primary components. The resulting target reliability indices for secondary components are β_T = 2.25 for 5 year evaluation, β_T = 2.50 for 10 year evaluation and β_T = 2.75 for 50 year period.

5.3 Time Effect

The target reliability indices depend on the considered time period. Theoretically, the reduced reliability indices can be obtained by reducing the load factors and/or increasing the resistance factors. However, the major difference between various time periods is in the live load model. Therefore, the live load factor is considered as the only variable. The considered adjustments are related to the live load factor rather than live load. The live load factor can be reduced by 5-10% for 5 year and 10 year evaluations (inspection intervals).

5.4 Current Design Code as a Reference

Reliability indices were calculated for representative bridges in conjunction with development of the LRFD AASHTO bridge design code [3, 13, 14]. The analysis was performed for selected existing structures and for idealized structures. The idealized bridges were considered without any over-design, it was assumed that the provided resistance is exactly equal to factored design loads. Structural failure can be associated with various limit states. Each structural component is designed to satisfy various safety requirements corresponding to different limit states. But in most cases, only one of these limit states governs. Therefore, it is practically impossible to avoid over-design. Optimum design requires the optimization of the governing limit states.

For ultimate limit states of flexural capacity (bending moment) and shear capacity, the results are shown in **Table 1** and **Table 2**. The reliability indices are shown for idealized bridges, designed exactly according to the code provisions. However, most of existing structures are over-designed. The ratio of the existing resistance and resistance required by the code varies. The actual values are shown in **Table 3** and **Table 4**[3),13)]. For the ultimate limit states, the required resistance is determined for components, β_T = 3.5 and structural systems, β_T = 5.5.

For serviceability limit states, reliability indices vary depending on the considered limit state. For prestressed

Table 1 Reliability Indices for AASHTO (1996), Simple Span Moment

Span (m)	Steel Girders Spacing			R/C T-Beams Spacing			P/C AASHTO Girders Spacing		
	1.2 m	1.8 m	2.4 m	1.2 m	1.8 m	2.4 m	1.2 m	1.8 m	2.4 m
9	2.00	2.66	3.10	2.24	2.73	3.07	1.90	2.58	3.05
18	2.90	3.54	3.96	2.97	3.42	3.71	2.98	3.62	4.07
27	2.85	3.39	3.76	2.94	3.28	3.53	2.95	3.49	3.88
36	2.75	3.24	3.57	2.88	3.16	3.37	2.90	3.34	3.68
60	3.19	3.56	3.82				3.23	3.65	3.96

Table 2 Reliability Indices for AASHTO (1996), Simple Span Shear

Span (m)	Steel Girders Spacing			R/C T-Beams Spacing			P/C AASHTO Girders Spacing		
	1.2 m	1.8 m	2.4 m	1.2 m	1.8 m	2.4 m	1.2 m	1.8 m	2.4 m
9	3.36	3.90	4.36	2.89	3.25	3.60	2.93	3.35	3.72
18	2.66	3.23	3.66	2.34	2.72	3.03	2.39	2.80	3.13
27	2.04	2.53	2.92	1.91	2.22	2.47	1.94	2.26	2.53
36	1.92	2.37	2.71	1.85	2.09	2.30	1.91	2.16	2.38
60	2.32	2.74	3.02				2.06	2.32	2.51

Table 3 Reliability Indices for AASHTO LRFD Code (1998), SimpleSpan Moment

Span (m)	Steel Girders Spacing			R/C T-Beams Spacing			P/C AASHTO Girders Spacing		
	1.2 m	1.8 m	2.4 m	1.2 m	1.8 m	2.4 m	1.2 m	1.8 m	2.4 m
9	4.19	4.20	4.20	3.98	3.99	4.00	3.88	3.88	3.88
18	4.28	4.28	4.29	3.95	3.94	3.96	3.99	4.00	4.01
27	4.30	4.30	4.31	3.78	3.82	3.86	3.97	4.00	4.02
36	4.32	4.32	4.32	3.68	3.73	3.76	3.90	3.97	4.01
60	4.26	4.27	4.28				3.82	3.93	3.99

Table 4 Reliability Indices for AASHTO LRFD Code (1998), SimpleSpan Shear

Span (m)	Steel Girders Spacing			R/C T-Beams Spacing			P/C AASHTO Girders Spacing		
	1.2 m	1.8 m	2.4 m	1.2 m	1.8 m	2.4 m	1.2 m	1.8 m	2.4 m
9	4.43	4.44	4.44	3.98	3.99	4.00	4.14	4.15	4.15
18	4.49	4.49	4.49	3.94	3.93	3.95	4.04	4.05	4.06
27	4.29	4.30	4.30	3.68	3.71	3.73	3.78	3.80	3.82
36	4.46	4.45	4.46	3.69	3.72	3.74	3.77	3.81	3.83
60	4.47	4.47	4.48				3.68	3.73	3.77

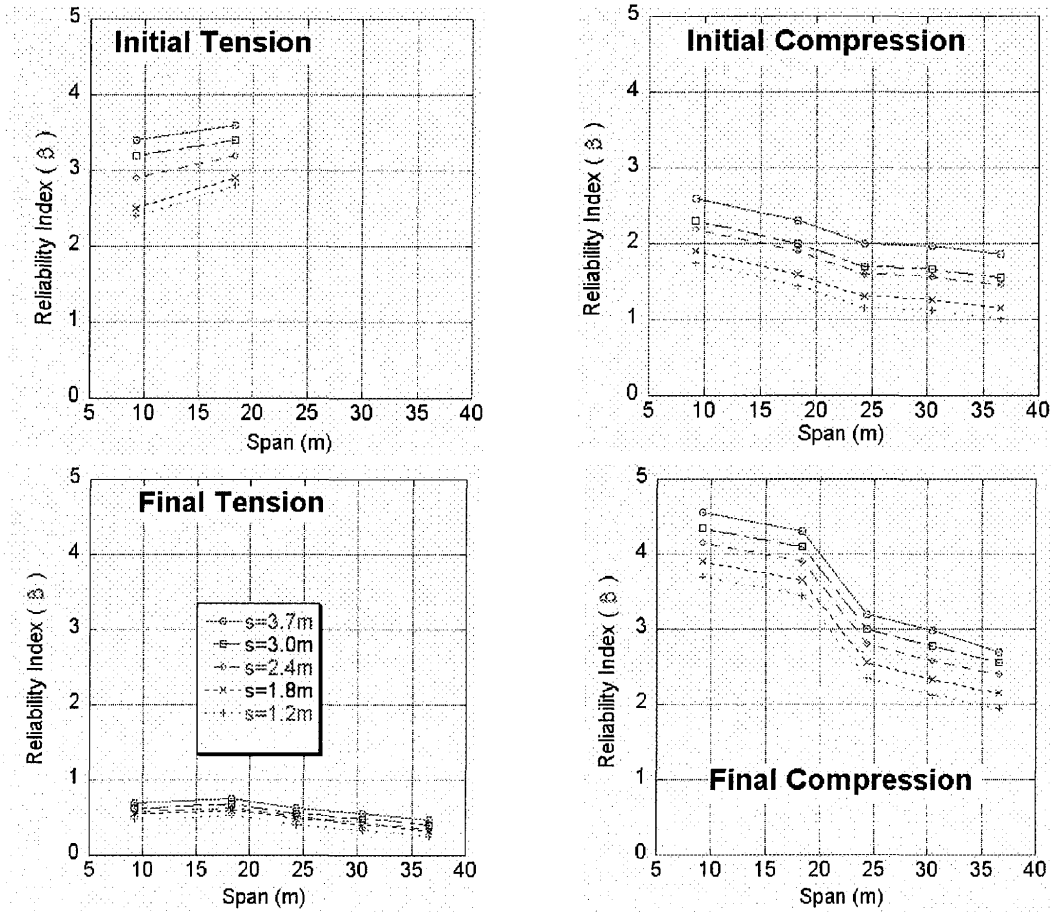

Fig. 1 Reliability Indices for SLS in Prestressed Concrete Girder BridgesDesigned by AASHTO[6], s = girder spacing.

Table 5 Recommended Target Reliability Indices for Design and Evaluation, ULS

Time Period	Primary Components		Secondary Components
	Single Path	Multiple Path	
5 years	3.50	3.00	2.25
10 years	3.75	3.25	2.50
50 years	4.00	3.50	2.75

concrete AASHTO type girders, the lowest values are obtained for tension stress after the final loss of prestress. Tension stress limit is imposed mainly as a protection of prestressing steel. An open crack may cause an accelerated deterioration of steel or concrete. A compression stress limit is considered to prevent formation of an excessive permanent deformation (kink) in the girder. The consequences of exceeding the tension stress limit are much less severe compared to the ULS. The calculations were carried for serviceability limit states in prestressed concrete girder bridges[15),16)]. The resulting reliability indices are presented in **Fig. 1**.

The current practice can be considered as representing at least a minimum acceptable limit. For serviceability limit states, reliability indices vary depending on limit state. For prestressed concrete AASHTO type girders, the lowest values are obtained for tension stress after final loss of prestress. Tension stress limit is imposed as a protection against cracking. An open crack may cause an accelerated deterioration (corrosion) of steel or concrete.

6. Recommended Values of the Target Reliability Index

Recommended values of the target reliability indices for design and evaluation of bridges are listed in **Table 5**. The numbers are rounded off to the nearest 0.25.

For SLS in prestressed concrete girders, the compression stress limit is considered to prevent the formation of an excessive permanent deformation (kink) in the girder. The consequences of exceeding the tension stress limit are much less severe compared to the ULS. Therefore, the proposed target reliability index for tension is $\beta_T = 1.0$. For compression stress, the target reliability is $\beta_T = 3.0$.

7. Conclusions

Target reliability index is considered for girder bridges. Reliability index for primary and secondary components depends on the consequences of failure. Ultimate and serviceability limit states are investigated. For ultimate limit state, reliability index for moment and shear varies depending on the spacing between the girders. It is also higher for single load path components compared to multi-load path components. For the serviceability limit states, stress limits in prestressed concrete girders are considered. The design is governed by tension stress limit at service loads (after final loss of prestress). The corresponding target reliability index is 1.0.

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