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Risk Mitigation for Highway and Railway Bridges

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Abstract

Performance of the transportation network strongly depends on the performance of bridges. Bridges constitute a vital part of the transportation infrastructure system and they are vulnerable to extreme events such as natural disasters (i.e., hurricanes, earthquakes, floods, major storms), as well as hazards stemming from negligence and improper maintenance, collisions (vessels and vehicles), intentional acts of vandalism, and terrorist attacks. These structures must be protected but the current approach to risk is not rational. Therefore, the objective of this project is to develop efficient risk analysis procedures for assessment of the actual safety reserve in highway and railway bridges. The focus is on the approach at the system level using system reliability methods. Sensitivity analysis relates the reliability of bridges and of the transportation network. The results will then be used to identify the critical parameters. The target risk will be determined depending on consequences of failure and relative costs. Rational selection criteria will be developed for the target risk level for bridges (components and systems) as a part of the transportation network, based on the consequences of failure and relative costs. This will involve the development of efficient system reliability procedures that will be applied to perform sensitivity analysis relating various parameters and reliability. The resulting sensitivity functions will provide a rational basis for identification of the most important parameters that affect the network performance. Rational selection criteria for the target risk will find important applications in decision making processes regarding operation, maintenance, repair, rehabilitation and replacement. This proposal will impact education and development of human resources since it will provide undergraduate and graduate students with research opportunities. The results will be included in courses and will be disseminated to wider audiences through presentations and publications.

Chapter 1 Introduction

The transportation network strongly depends on performance of highway and railway bridges. Bridges are vulnerable to extreme events such as natural disasters (i.e., hurricanes, earthquakes, floods, major storms), in addition to hazards stemming from negligence and improper maintenance, overloading, collisions (vessels and vehicles), intentional acts of vandalism, and terrorist attacks. These structures must be protected, but the current approach to risk is not always rational. Consequently, an important question facing owners and administrators of the transportation network is how safe should bridges be under their jurisdiction. The answer requires the development of (1) risk analysis procedure for calculation of the reliability/safety of bridge components and systems, (2) selection criteria for the target (acceptable) risk, and (3) implementation mechanisms that result in bridges that maintain the acceptable risk level. The developed procedures will serve for quantification of the risk and provide important tools in the decision making process regarding the transportation network as it relates to the operation of bridges, maintenance, repair, rehabilitation and replacement.

1.1 Research Approach and Methods

The objective of this project is to develop efficient risk analysis procedures for assessment of the actual safety reserve at the network level and for individual bridges. The focus is on the system approach using the system reliability methods. The work for this project will involve the formulation of limit state functions, identification of basic parameters, and the development of advanced procedures for analysis of structural behavior. Sensitivity analysis will be performed relating the reliability and critical parameters. The target risk will be determined

depending on consequences of failure and relative costs. The results will serve as a basis for a rational decision making procedure at the network and bridge level.

The long-term goal of the proposed research is the development of risk analysis procedures for the transportation network and its components, and selection criteria for the target reliability levels for bridge components and systems. The specific objectives of this project include the following three bulleted points.

- *Development of efficient risk analysis procedures that can be used for the assessment of the actual risk in structures.* The focus will be on the approach at the system level using system reliability methods (Nowak and Collins 2000). Research will involve the formulation of limit state functions, identification of basic parameters, and the development of advanced procedures for analysis of structural behavior. A statistical database will be established for the parameters that determine the performance of the considered structures. The needs for additional statistical data will be formulated. The procedures will allow for the development of lifetime reliability profiles, and methodology for prediction of the remaining life for the considered structures.
- *Development of procedures for the selection of rational acceptability criteria for risk.* The approach will be based on the analysis of consequences of failure to perform as expected and economic analysis (costs). This will be a very important contribution of the proposed research effort as this is an area of great need in the transportation field. The acceptable risk, or target reliability level, can be different depending on exposure of human life and importance of the facility, and the acceptable risk will affect the selection of the structural systems, components, and materials.
- *Development of implementation strategy for risk control to keep risk within acceptable levels.* The developed reliability analysis procedures will be applied to assess the risk associated

with the selected structural systems. The computations will require a statistical database. The results will be compared with the selected target reliability levels to determine if the current situation requires changes. The risk control procedures will be developed to ensure that the risk is at an acceptable level. Sensitivity analysis will be carried out relating the reliability and various design and other parameters (Nowak and Czarnecki 2005). The sensitivity analysis can require the development and use of advanced non-linear structural analysis methods.

The reliability models will be developed for various limit states, including ultimate limit states, serviceability, and extreme events (AASHTO 2007). The transportation networks considered in this study include highway and railway bridges.

The reliability analysis requires the development of a statistical database. The proposed study will use the available information, and if needed, additional parameters will be obtained by simulations and/or engineering judgment. Statistical models for bridges and some extreme events are described by Ellingwood et al (1980), Nowak (1993, 1995 and 1999) and Nowak and Knott (1996). The system capacity and demand (loads and extreme events) are random variables. The causes of uncertainty can be put into three categories: (1) material factor, (2) fabrication factor, and (3) analysis factor. The resulting variation has been modeled by tests, observations of existing structures, and engineering judgment. The information is available for the basic structural materials and components. However, bridges may require special methods of analysis. Verification of the analytical model can 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.

Computer methods of structural analysis have improved the accuracy of representing the actual behavior of bridge components. Advanced programs (e.g., ABAQUS, NASTRAN,

ANSYS) are available for linear and nonlinear analysis of complex structural systems. A dense element mesh allows for an accurate determination of strain/stress at almost any point in the structure. One major problem that remains is how to represent boundary conditions and material properties. For example, the actual support in situ is often different than an idealized type, and strength of material and modulus of elasticity can be different than what is assumed in design. The deterministic analysis is a useful tool, but there is need to include the randomness of such parameters.

The major selection criteria for the target risk level are consequences of failure and cost of increasing reliability, or the benefit of decreasing β (Lind and Davenport 1972). The consequences of failure can include costs of repair or replacement, limited use/operation during repair or replacement, public inconvenience due to delays and detours, a higher risk of accidents, increased insurance premium, and loss of human life and limb. The cost is another factor that can have a significant effect on selection of the target reliability. The more expensive it is to increase the structural safety level, the more acceptable a lower reliability value. Therefore, the target reliability is lower for existing structures than for newly designed ones.

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 considering the reliability indices corresponding to the structures designed using an existing code (Kaszynska and Nowak 2002, 2003, 2005). If there are no reported problems for the considered class of structures, then it can be concluded that the current and/or 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.

Chapter 2 Highway Bridge Load Models

Statistical models of load and resistance for highway bridges are described by Nowak (1993; 1995 and 1999). The basic parameters that describe each load component are bias factor, the ratio of the mean-to-nominal values signified by λ; and coefficient of variation, V. The main load combination includes dead load, live load and dynamic load. Dead load is the gravity load resulting from 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: the weight of factory made elements, steel and precast concrete members; of cast-in-place concrete members; and of the wearing surface, asphalt. All components of dead load are treated as normal random variables. The statistical parameters of dead load are λ = 1.03-1.05 and V = 0.08-0.10. The lower values are applicable to structural steel and plant-cast concrete, and the upper values are applicable to cast-in-place concrete.

Live load includes the static and dynamic components. The static 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 bias factors in terms of the HS20 live load (AASHTO 2002) are between 1.65 and 2.10 (Nowak 1993). 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 coefficient of variation is 0.11 for most spans.

The dynamic load model is a function of three major parameters: road surface roughness, bridge dynamics (frequency of vibration) and vehicle dynamics (suspension system). 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 and it does not exceed 0.15 of live load for a single truck and 0.10 of live load for two trucks side-by-side (Eom and Nowak 2001).

2.1 Resistance Model

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) a material factor including strength of material, modulus of elasticity, cracking stress, and chemical composition; (2) a fabrication factor including geometry, dimensions, and section modulus; and (3) an 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, or 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.

$$R = R_n \, M \, F \, P \tag{1}$$

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 (Nowak 1999).

For steel girders, the parameters of R are $\lambda_R = 1.12$ and $V_R = 0.10$ for moment and $\lambda_R = 1.14$ and $V_R = 0.105$ for shear. For reinforced concrete T-beams, the parameters of R are $\lambda_R = 1.12$ and $V_R = 0.135$ for moment and $\lambda_R = 1.20$ and $V_R = 0.155$ for shear. For prestressed concrete, $\lambda_R = 1.05$ and $V_R = 0.075$ for moment and $\lambda_R = 1.15$ and $V_R = 0.14$ for shear.

2.2 Railway Bridge Load and Resistance Models

Railway bridge load and resistance models will be based on the ongoing research and cooperation with the Union Pacific. The data that follows was obtained from Union Pacific.

- Boone, 106 bridges, 202.48 – 325.72 (123.24 miles)
- Clinton, 189 bridges, 2.14 – 194.33 (192.19 miles)
- Columbus, 38 bridges, 44.71 – 131.64 (86.93 miles)
- Geneva, about 150 bridges, 0.0 – 137.26 (137.26 miles)
- Kearney, 43 bridges, 146.73- 266.8 (120.07 miles)
- North Plate, 4 bridges, 282.0 – 289.59 (7.59 miles)
- Omaha, 56 bridges, 1.46 - 32.15 and 330.70– 351.15 (51.14 miles)

Typical construction types:

- steel beam
- through plate girder

- deck plate girder
- reinforcement concrete slab
- concrete beam
- through truss riveted
- prestressed concrete box
- prestressed concrete slab

74% of bridges were constructed before 1950

20% of bridges were constructed after 1980

45% of bridges is shorter than 50 ft

23% is longer than 100 ft

Chapter 3 Reliability Analysis Procedure

The available reliability methods are presented in several publications, such as Nowak and Collins (2000). The reliability index is defined as a function of P_F,

$$\beta = -\Phi^{-1}(P_F) \tag{2}$$

where Φ^{-1} = inverse standard normal distribution function. There are various procedures available for calculation of β. In this study, β, is calculated using an iterative procedure and Monte Carlo simulations.

Two types of limit states are considered. 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.

3.1 Acceptability Criteria for Bridges

The approach is based on the analysis of consequences of failure to perform as expected and economic analysis of costs. Selection of the target reliability, which is an efficient measure of structural performance, can be based on consideration of these two parameters. The acceptable

risk, or target reliability level, can be different depending on exposure of human life and importance of the facility. The acceptable risk will affect the selection of the structural systems, components, and materials. While the probability of failure is important for users of bridges, it is more pertinent to investors and owners, who demand to know if the level of safety of the structures under their jurisdiction is sufficient. Moreover, the consequences of not paying heed to the deteriorating infrastructure are becoming readily apparent. Issues stemming from a lack of quality can cause death, delays, detours, increased risk of accidents, repair costs and costs of replacement. Recent extreme events - natural disasters (i.e., hurricanes, earthquakes, floods, and major storms), improper maintenance, collisions (vessels and vehicles), intentional acts of vandalism, and terrorist attacks - are having a tremendous effect on public and political attitudes toward risk. Hurricanes Ike, Katrina, and Rita brought a sudden national awareness of the vulnerability of the vast civil infrastructure that supports our lives. The sudden and tragic collapse of the I-35W Bridge in Minneapolis a year ago raised a question about the safety of other US bridges. Decisions regarding investment in new infrastructure facilities and systems, preventive maintenance and/or repair, or corrective actions can entail billions of dollars. The consequences of a poor decision can expose thousands of people to the possibility of injury, death, or financial ruin.

From a safety perspective, the probability of bridge failure should be infinitely small. The problem therein is that targeting a significantly high level of reliability induces higher costs of construction and maintenance. The goal is to find what probability of failure can be accepted by society. Beyond this, the levels the target reliability vary based on materials, factors involved in fabrication, the difference between estimated strength and actual strength, structural design and the manner in which failure will occur if it does occur. In inevitable situations, such a serious

natural disasters, failure of the structure should not occur suddenly. Structure failing gradually and in the predicted way can allow for evacuation and avoidance of human losses. The most dangerous are unexpected rapid failures, such as those caused by exceeding shear capacity.

The target reliability has to be defined for ultimate limit states (ULS) and serviceability limit states (SLS). Exceeding ultimate limit strength can cause serious consequences and often requires replacing the element or the entire structure. Serviceability limit states have a lower level of failure consequences. The cost of repair is also much lower. Therefore, lower values of the target reliability index can be selected for SLS than ULS. The similar distinction has to be made between primary and secondary components.

3.2 Target Risk Levels for Bridges

Development of target reliability was assessed for girder bridges, but other types are currently being studied. Suspension, cable-stayed, and arch bridges were constructed in areas where application of girder bridges was impossible—that is, extensive obstacles like wide valleys and rivers. The cost of their construction and maintenance is much higher, and the consequences of failure can be more serious. Therefore, the target reliabilities for ultimate and serviceability limit states will be higher than for girder bridges. As far as railway bridges are concerned, development of the target reliability was not performed and requires profound studies.

3.3 Target Risk Levels for Girder Bridges

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 the 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 the final loss of prestress. The corresponding target reliability index is 1.0.

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 a 50 year life time and existing bridges are checked for 5 or 10 year periods. Load model, used to calculate the reliability index, depends on the reference time period. Maximum moments and shears are smaller for 5 or 10 year periods than for a 50 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.

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 main load combination includes dead load, live load and dynamic load. Dead load model is not time-dependent. Live load varies depending on time, and the maximum 5 year moment (or shear) is about 5 percent less than the maximum 50 year moment (or shear). The difference between a 10 year moment and a 50 year moment is about 3 percent. Dynamic load allowance, as a fraction of the live load, is changing with time too. Resistance also depends on the reference time period because of deterioration of the structure, particularly strength loss due to corrosion or fatigue.

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 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, such as liability in case of injuries. Because of economical reasons, it is convenient 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 a 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 less severe as compared to the ULS.

Chapter 4 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 β_e be an element reliability index and β_s 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 P_{Fe},

$$P_{Fe} = \Phi(-\beta_e) \tag{3}$$

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

$$P_{Fs} = \Phi(-\beta_s). \tag{4}$$

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

$$P_{Fs} = (P_{Fe})^n \tag{5}$$

where n is a number of elements.

For n = 2 and $\beta_e = 3.5$, the system reliability is $\beta_s = 5.3$, when elements are fully uncorrelated. However, the elements are usually partially correlated and therefore, $\beta_s = 3.5$ 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 $\beta_T = 3.0$ and for single load path components $\beta_T = 3.5$. The corresponding 50 year values are $\beta_T = 3.5$ for multiple path components and $\beta_T = 4.0$ for single path components.

4.1 Primary and Secondary Components

A primary component is a main structural element, the 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 $\beta_T = 2.25$ for 5 year evaluation, $\beta_T = 2.50$ for 10 year evaluation and $\beta_T = 2.75$ for 50 year period.

4.2 Current Design Codes

Reliability indices were calculated for representative bridges in conjunction with development of the LRFD AASHTO bridge design code (Nowak 1995). The analysis was performed for selected existing structures and for idealized structures. The idealized bridges were considered without any overdesign, and it was assumed that the provided resistance is exactly equal to factored design loads.

Structural failure can be associated with various limit states. Ultimate limit states are related to loss of load carrying capacity—such as flexural strength, shear capacity, loss of stability, rupture and so forth. Serviceability limit states are related to cracking, deflection and vibration. Fatigue limit states are reached as a result of repeated load applications. 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 Tab. 4.1 and 4.23. The reliability indices are shown for idealized bridges, designed exactly according to the code provisions. However, most of existing structures are overdesigned. The ratio of the existing resistance and resistance required by the code varies. The actual values are shown in tables 4.1 and 4.2 (Nowak 1995). For the ultimate limit states, the required resistance is determined for components, $\beta_T = 3\text{-}5$, and structural systems, $\beta_T = 5.5$.

Table 4.1 Reliability Indices for AASHTO (1989), 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 4.2 Reliability Indices for AASHTO (1989), 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

For serviceability limit states, reliability indices vary depending on the considered limit state. For prestressed 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 by Ef-Hor and Nowak (1995) and Nowak and Saraf (1996).

The analysis of a wide range of design cases indicates that the ultimate limit state never governs for prestressed concrete girders. The number of prestressing strands is always determined by the allowable tension stress at the final stage, after full loss of prestressing force.

Table 4.3 Resistance Ratios for LRFD Code, 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	1.29	1.17	1.10	1.33	1.30	1.23	1.26	1.16	1.09
18	1.12	1.03	0.97	1.22	1.13	1.08	1.09	1.02	0.97
27	1.12	1.04	0.99	1.18	1.13	1.09	1.08	1.03	0.99
36	1.13	1.06	1.01	1.17	1.13	1.10	1.08	1.04	1.00
60	1.05	1.00	0.97				1.03	1.00	0.97

Table 4.4 Resistance Ratios for LRFD Code, 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	1.09	1.01	0.94	1.21	1.14	1.06	1.16	1.08	1.01
18	1.22	1.12	1.05	1.33	1.24	1.18	1.24	1.16	1.10

The target reliability indices from table 4.5 can be used to derive the evaluation criteria for 5 and 10 year periods. 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. It is recommended to use live load factor λ_L = 1.25 for 5 year evaluation and λ_L = 1.30 for 10 year evaluation.

Table 4.5 Recommended Target Reliability Indices for Evaluation

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

4.2.1 Single and Multiple Load Path Components

The difference is mostly due to the system behavior. Therefore, it is recommended to use reduced resistance factors for single load path components. However, if instead of resistance factor, only dead load and live load factors can be adjusted for single path components, then it is recommended to use λ_D increased by 5 percent (compared to multiple path components), λ_L = 1.55 for normal traffic and λ_L = 1.25 for a controlled vehicle.

4.2.2 Primary and Secondary Components

The difference is mostly due to consequences of failure. Therefore, it is recommended to increase the resistance factor for secondary components. If instead of resistance factor, only dead load and live load factors can be adjusted for secondary components, then it is recommended to use λ_D and λ_L reduced by 5 percent (compared to primary components).

4.2.3 Ultimate limit state

For ultimate limit states, the calculated reliability indices represent component reliability rather than system. The reliability indices for structural system were calculated by Tabsh and Nowak (1991) and they are larger than β for individual girders by about 2; that is, instead of β = 3-4 for components, for the system β = 5-6. The selection of the target reliability level should be

based on consideration of the system. Then, target reliability index for components (girders) can be derived using the appropriate formulas from the system reliability.

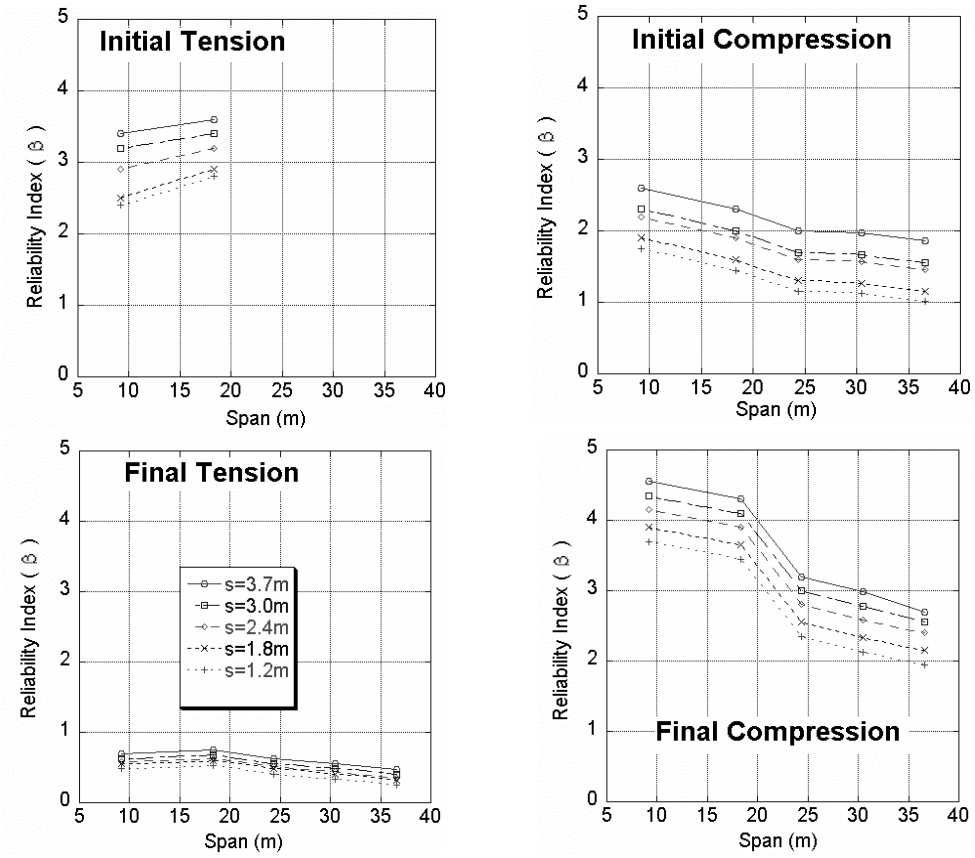
In the development of LRFD Code (AASHTO 1994), the target reliability index for a component, a girder, was selected equal to $\beta_T = 3.5$. The load and resistance factors were determined so that the corresponding reliability indices for ULS exceed the target value.

4.2.4 Serviceability limit states

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, or corrosion, of steel or concrete. A 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 $\beta_T = 1$. For compression stress the target reliability is $\beta_T = 3.0$. The calculations were carried for serviceability limit states in prestressed concrete girder bridges (Nowak and El-Hor 1995). The resulting reliability indices are presented in fig. 4.1.



s = girder spacing.

Fig. 4.1 Reliability Indices for SLS in Prestressed Concrete Girder Bridges Designed by AASHTO (2002)

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.

4.3 Recommended Values of the Target Reliability Index

Recommended values of the target reliability indices for design and evaluation of bridges are listed in table 4.5. The numbers are rounded off to the nearest 0.25.

Chapter 5 Conclusions

This project evaluated the target reliability index for highway 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 were 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.

Target reliability for railway bridges is currently under investigation. Research based on the cooperation with the Union Pacific will be presented in a future project (proposal to be submitted).

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