# Reliability of Corroded Steel Bridge Girders

by

**Irene A. Cordahi**

Bachelor of Engineering in Civil and Environmental Engineering
American University of Beirut, June 2005

Submitted to the Department of Civil and Environmental Engineering
Fulfillment of the Requirements for the Degree of
Master of Engineering in Civil and Environmental Engineering

at the

**MASSACHUSETTS INSTITUTE OF TECHNOLOGY**



June 2006



Signature of Author..........................................................
Department of Civil and Environmental Engineering
May 12, 2006

Certified by..........................................................
Jerome J. Connor
Professor of Civil and Environmental Engineering
Thesis Supervisor

Accepted by..........................................................
Andrew Whittle
Chairman Departmental Committee for Graduate Studies

# Reliability of Corroded Steel Bridge Girders

by

**Irene A. Cordahi**

Submitted to the Department of Civil and Environmental Engineering
on May 12, 2006 in Partial Fulfillment of the
Requirements for the Degree of Master of Engineering in
Civil and Environmental Engineering

ABSTRACT

Corrosion is one of the main causes of deterioration of bridges. Structures exposed to harsh environmental conditions are subjected to time-variant changes of their load-carrying capacity. Thus, there is a need for an evaluation to accurately assess the actual condition and predict the remaining life of a structure. System reliability can be used as an efficient tool in evaluation of existing structures. The traditional approach is based on the consideration of individual components rather than the system as a whole. However, it has been observed that the load-carrying capacity of the whole system often is much larger than what is determined by the design of components. Quantification of this difference is the scope of this study.

Thesis supervisor: Jerome J. Connor
Title: Professor of Civil and Environmental Engineering

To my dear father

for his love and support

# Acknowledgments

Since there are many people who helped, guided, and encouraged me along this way, first, I want to thank my thesis advisor, Professor Jerome J. Connor, who was always there to help me reach my academic goals.

I offer my thanks to Professor Andrzej S. Nowak, who was my research advisor at the University of Michigan in Ann Arbor and who enhanced my interest in reliability analysis of bridge structures. Actually, professor Nowak directed a considerable number of research studies related to the topic of my thesis.

I am grateful to Massachusetts Institute of Technology's MEng 2006 students who were helping and supporting each others continuously. I also appreciate the great friendship provided by Aurelie Ble and Claire Mattelet during this academic year.

Finally and most importantly, I offer my sincere thanks and love to my father Alain Cordahi, to whom this thesis is dedicated, and who is a key person behind my academic success. Since my childhood, he believed in my intellectual capabilities and I hope that I will never disappoint him.

## Table of Contents

## List of Figures

## List of Tables

# 1. Introduction

Bridges constitute important components of the transportation network. The majority of steel bridges in the United States were constructed after World War II from 1950 to 1980. Unfortunately, bridges are aging and their probabilities of failure are increasing. There are around 190,000 steel bridges that are simply supported and continuous. More than 40,000 of them (22.5%) are structurally deficient and more than 35,000 (18.5%) are functionally obsolete. Structural elements deteriorate with time due to corrosion, wear, fatigue, and other forms of material degradation. Moreover, lately, the legal load on bridges has increased and the problem is that the majority of the old bridges fail to satisfy this requirement. In addition, the maintenance cost of a bridge is of great importance. According to ASCE (2005), it is estimated that about $ 10 billion per year for a period of 20 years will be needed to get rid of all bridge deficiencies. Deficient bridges are either repaired, or replaced. Thus, it would be better to evaluate bridges for their load-carrying capacity and to predict the change in their capacities with time due to deterioration or any other event. In addition, due to the harsh and corrosive environments, the use of roadway deicing chemicals, and the natural aging of these structures, states and governments have to deal with the high cost of repair, rehabilitation, or replacement of bridges and motorists have to handle construction detours and delay. Efficient maintenance, repair, and rehabilitation of existing bridges require the development of an efficient method to evaluate the load-carrying capacity and predict the remaining life of the structure.

Lately, the use of reliability in structural assessments improved the quality of bridges concerning safety, serviceability, and durability. Partial safety factors based on load and resistance uncertainties were proposed in the United States for building design codes not before the late 1970's (Galambos and Ravindra 1978, and Ellingwood 1980). "New bridge design codes are based on probabilistic methods" (Nowak 1999) and load and resistance parameters are considered as random variables.

The reliability index used is $\beta$. It is used to measure the reliability of structural members as well as structural systems. This index accounts for the margin of safety implied by the design procedure as well as the uncertainties in estimating the member strengths and the applied loads. It is related to the probability of failure (Nowak and Collins 2000). The assessment of the probability of failure of a system is much harder than the calculation of the probability of failure of one member even if the potential failure modes are known in advance. A bridge is considered to be a system of members like independent beams, bearings, diaphragms, foundations, slabs, girders and so on, thus, the overall reliability of the bridge may differ from the reliability of each of its components. The resistance of a bridge depends on loading, redundancies, presence of multiple loads, and the configuration and position of a truck on the bridge. Bridge redundancy, which is the capability of a bridge superstructure to continue to carry load after the failure of one of its members contributes to the system behavior. The failure of a member could be either ductile or brittle. It can be caused by the application of large live loads or the sudden loss of one element due to fatigue, brittle fracture, or an accident such as collision of trucks, ships, or debris. Bridges have different modes of failure and when a failure occurs, it could involve the failure of a component or a system. The failure of one member of a bridge will not necessary lead to the collapse of the whole bridge. For instance, when a girder reaches its ultimate capacity, it does not fail usually and the additional load will be redistributed and sustained by other girders. As a result, a bridge will not fail unless several adjacent girders have reached their ultimate capacity. Nowadays, the system effect is ignored. This study is performed to bridge the gap between a component-based design and the system effect.

Recent developments in bridge diagnostics, structural tests, and material properties are very useful. Full scale bridge tests provide important information about the structural behavior (Faber et al. 2000). For example, proof load tests proved that the load-carrying capacity of girder bridges is usually larger than analytically predicted (Safar and Nowak 1997 and 1998) due to unintended composite action, partial fixity of supports, and the influence of sidewalks, parapets, and railings.

To understand well the effect of corrosion and loss of metal sections on bridge behavior, and consequently the reliability of steel girder bridges, the mechanics of steel corrosion should be well understood. The rate of corrosion is often subjected to fluctuation and difficult to predict. Corrosion can not just cause a fracture to occur, but also yielding or buckling of members. This can result in an increase in stress, change in geometric properties, a buildup of corroded products, and the reduction in member cross section properties like the section modulus or the slenderness ratio. These parameters are critical in a member's ability to resist bending moments or axial forces. In addition, a corrosion buildup can affect structural performance as it may act as a desiccant that may promote further corrosion. In this study, three types of environment are considered. The objective of this paper is to develop a time-variant reliability-based model for evaluation of steel highway bridges during their service life.

## 2. Structural Reliability Analysis

Most of the available reliability methods are designed to check the performance of individual components like columns, beams, tension members, and connections (bolts, welds). Design codes specify the nominal values of loads to be resisted by each component by applying safety factors. The resistance or load-carrying capacity of a component is expressed as materials (grades and types) and dimensions (geometry). All structural components should satisfy the code requirement, that is, the resistance should be greater or equal to the load effect. Usually, component-based design is very conservative because of redundancy and ductility. In this study, the bridge is considered as a system of interconnected members. Thus, the redundancy and complexity of a structure are quantified.

Limit states are the boundaries between safety and failure. For a bridge, failure is the inability of the structure to carry traffic. Bridges can fail by cracking, corrosion, excessive deformations, local or overall buckling, exceeding carrying capacity for shear or bending moment, and so on. Moreover, members can fail in a ductile or brittle manner. In the traditional method, each mode of failure is analyzed separately.

There are two types of limit states; namely: the ultimate limit states related to bending capacity, shear capacity, and stability, the serviceability limit states such as cracking, deflection, or vibration, related to gradual deterioration, user comfort, and maintenance costs, and fatigue. The main concern in this latter type is the accumulation of damage caused by repeated application of loads. For this reason, the reliability model should include the load magnitude and the frequency of occurrence, not just the load magnitude as it is considered for the ultimate limit states. For the scope of this study, it is focused on the ultimate limit states of the moment-carrying capacity. To find the reliability of one component, a performance function g(X) of the random variable X is defined, knowing that when g is positive, the performance of the bridge is satisfactory and when g is negative, the structure will experience failure. Thus, $P_F = \Pr ob(R - Q < 0) = \Pr ob(g < 0)$ where $P_F$ is the probability of failure. Let R be the resistance or moment-carrying capacity of the bridge and Q the load effect or total moment applied to the considered beam. The corresponding limit state function is written as: $g = R - Q$. The evaluation of the limit state function for the whole bridge is quite complex because of the possible statistical dependence among the random variables, and the redundancy of the structure that is causing a load sharing. The limit state function can be a function of many variables like influence factors, resistance parameters, load components, material properties, dimensions, analysis factors, and so on. Thus, a direct calculation of $P_F$ may be hard. As a result, it is appropriate to measure structural safety as a reliability index $\beta$, which is a function of $P_F$.

$$\beta = -\Phi^{-1}(P_F) \qquad \textbf{(Equation 1)}$$

where $\Phi^{-1}$ is the inverse standard normal distribution function.

Different methods are used for the calculation of $\beta$. These procedures vary concerning the required input data, accuracy, and computing costs. An efficient iterative procedure is based on normal approximations to non-normal distributions at the specified design point, which is the point of maximum probability of the failure boundary or limit state function. This procedure is programmed and calculations are performed by the computer. Parameters of R and Q and also the limit state function g can be obtained using Monte Carlo simulations. Thus, $\beta$ can be calculated by using the iterative procedure. The accuracy of calculations depends on the number of computer runs. The means and coefficients of variation of R and Q can be estimated using sampling techniques. In addition, point estimate methods have been developed to limit the number of function evaluations in an analysis.

So many approaches have been proposed concerning the limit state function, and failure afterwards. Zhou (1987) considered that system failure occurs when two adjacent girders fail. Tabsh and Nowak (1991) proposed that a structure collapses when several girders would have reached their ultimate capacity. According to Ghosn and Moses (1998), the bridge resistance is the maximum gross vehicle load that is causing a collapse of the structure. Estes and Frangopol (1999) assumed that failure occurs when three girders out of five fail. And Liu and Moses (2001) considered that a steel girder bridge is damaged due to collision, corrosion, and many other reasons. For the scope of this study, the failure of a bridge is assumed to be the smaller of the two: (1) the maximum load that the bridge is able to carry, or (2) 0.75% of the span length deflection in any of the main members of the bridge.

For the calculation of deflection, the static and dynamic portions of the live load are taken into account. The basic load combination used includes dead loads and live loads (static and dynamic). Live load is represented in the form of trucks. Two types of trucks were selected for the scope of this study: a three-axle vehicle (S) and a five-axle vehicle (T). The axle spacing and load distribution factors per axle are constant, but the gross vehicle weight (GVW) is a random variable. The transverse position of the truck within the roadway (curb distance) is also considered as a random variable. A Probability Distribution Function (PDF) of the curb distance is shown in Figure 1 for two traffic lanes. Each PDF represents a line of truck wheels, spaced at 1.8m. Furthermore, different possible load cases and probabilities are shown in Table 1.

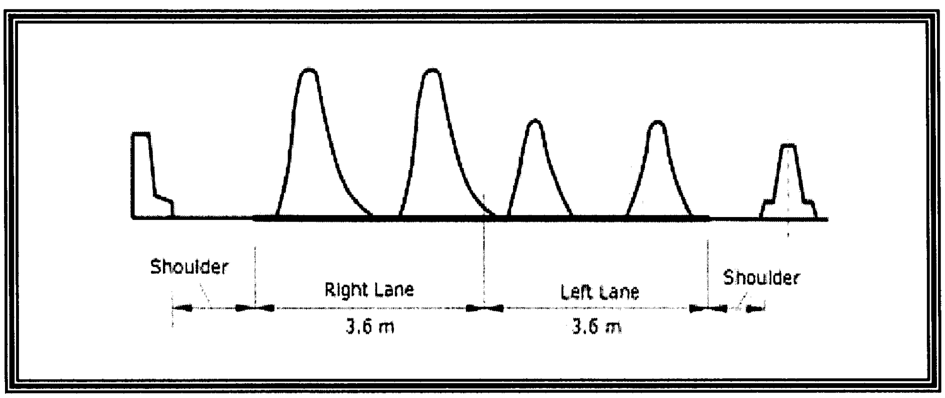



**Figure 1. Probability Density Function (PDF) of the Curb Distance (Tantawi 1986).**

The PDF was approximated by a lognormal distribution with a coefficient of variation of 0.33. For a lane width of 3.63 m, the mean value of the distance from the lane edge to the centerline of the outermost truck wheel is around 0.91 m.

| One Lane Loaded | | Two Lanes Loaded | |
|---|---|---|---|
| Truck Placement | Probability of Occurrence | Truck Placement | Probability of Occurrence |
| S | 0.133 | S<br>T | 0.2 |
| T | 0.533 | T<br>S | 0.3 |
| S | 0.067 | S<br>S | 0.1 |
| T | 0.267 | T<br>T | 0.4 |

**Table 1. Considered truck positions and probabilities of occurrence. (Tantawi 1986)**

Three probabilistic methods are used by Professor Nowak and his team to estimate the reliability indices of bridge girders and systems; namely, First Order Second Moment (FOSM), Monte Carlo method, and Rosenblueth 2n+1 Point Estimate method (Nowak and Collins, 2000).

The bridge resistance is estimated for each transverse position of the truck, in terms of the truck weight. The Monte Carlo method is used to generate the material and the component parameters of the bridge. A finite element method is used to calculate the strain, stress and deflections. The gross vehicle weight is gradually increased until the deflection exceeds the acceptable level.

For each value of curb distance, a reliability index is calculated. Afterwards, the system reliability index for the bridge is determined as the weighted average.

## 3. Corrosion Models

Corrosion is one of the most distinctive causes of deterioration in steel bridges. The degree of corrosion depends mostly on the environment where the bridge is located. The primary cause of corrosion is the reaction between water and salt used for deicing on the bridge. The oxide formed by oxidation does not firmly adhere to the surface of the steel and it flakes off easily causing pitting. As a result, extensive pitting will lead to structural weakness and disintegration of the metal. The source of water and salt is either from deck leakage or from the accumulation of road spray and condensation. The location of corrosion is found based on the source of humidity and the rate of corrosion is related to the contamination in the moisture and the ambient temperature (Park, 1997). The determination of the actual strength and the remaining life of a bridge are very important in order to avoid dangerous consequences.

During the analysis of corrosion, two important issues need to be addressed: the rate and the progress of corrosion. Albrecht and Naeemi (1984), McCrum (1985), and Kayser (1998) studied the effect of corrosion. Corrosion of steel is related to many factors like surface protection, type of steel, environmental effects, and existence of pollutants, crevices, and stress.

" Some forms of corrosion could be seen with a naked eye such as uniform corrosion, while others, like pitting and crevice corrosion, and stress corrosion, could not" (Fontana 1986). In this paper, a uniform corrosion is considered. A uniform corrosion, which is described as the loss of materials, accounts for the largest percentage of corrosion damage.

Since many factors influence the corrosion rate of steel, it is very hard to determine this rate. Furthermore, there is no appropriate statistical data. One of the methods used to solve this problem is to find a corrosion rate based on an empirical relationship and future corrosion rates could be estimated based on previous corrosion rates of the same structure.

Albrecht and Naeemi (1984) studied the behavior of various types of steel exposed to different types of environments, namely: rural, urban, industrial, and marine environments. They got a wide range of values concerning corrosion rates. This may be caused by the small samples tested which do not represent real life structural behavior. Despite that, there is a common agreement that corrosion time-penetration data best fits an exponential function (Komp, 1987):

$$C = A * t^{B} \qquad \textbf{(Equation 2)}$$

where:

C= average penetration determined from weight loss

t= time in years

A= regression coefficient numerically equal to the penetration after one year of exposure

B= regression coefficient numerically equal to the slope of Equation 2 in a log-log plot

For example, for carbon steel and rural environment, A= 34.0 with a coefficient of variation equal to 0.69, and B= 0.65 with a coefficient of variation equal to 0.10 (Albrecht and Naeemi, 1984). This previous model considers that corrosion begins right after the members of a bridge are erected, which is often not true.

Park and Nowak (1997) proposed three curves, high, medium, and low corrosion rates, as shown in Figure 2. Field observations were used to get these curves and the following assumptions were made:

1. A new bridge is painted and paint provides a protection cover for 5 to 15 years.
2. A bridge is not repainted in the future.
3. A low corrosion rate results from dry conditions of the environment, adding to this, no salt or any other chemically aggressive deicing agents is assumed to act on the bridge.
4. A high corrosion rate is typical of marine environment or heavy industrial conditions, or areas where deicing chemicals are used.

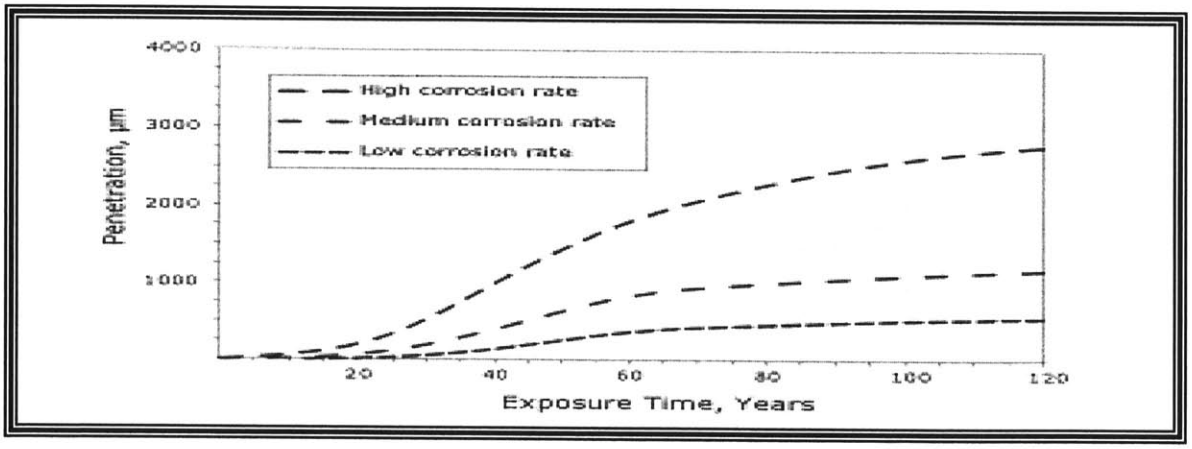



**Figure 2. Corrosion Penetrations (Park and Nowak 1997).**

The corrosion process has two phases. The first one is related to the life of coating and the other to the progress of corrosion. Corrosion begins when the coating is lost and then it starts progressing. The mean value of coating is 5 years for high corrosion rates and 15 years for low corrosion rates.

As shown in Figure 3, corrosion occurs on the entire web surface due to deck leakage and on the top of the bottom flange due to traffic spray accumulation. "At midspan, this model, derived from field surveys, assumed that the corrosion of the web is reaching ¼ of the web height" (Kayser 1988). Despite that some data show that the corrosion of exterior girders is different from the one of interior girders, here, all the girders corrode at the same rate.

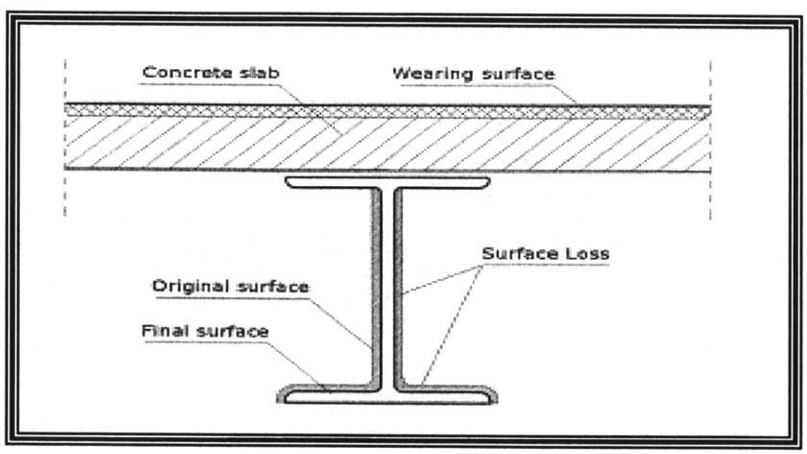



**Figure 3. Corrosion of a Steel Girder Bridge (Kayser 1988).**

Engineers have to deal with the problem of corrosion in order to avoid substantial damage of the structure. The corrosion effects on steel bridges may range from non-structural maintenance problems to undesirable effects like failure or collapse. Kulicki et al. (1990) found four major categories of corrosion effects which are loss of section, creation of stress concentration, introduction of unintentional fixity, and introduction of unintended movement. The first category is the most common one. The loss of metal could be uniform or localized in a form of pits, holes or edge scallops. The loss of section of some components may not affect the overall capacity of the bridge, but the deterioration of some members may cause a significant damage to the behavior of the structure as a whole. The loss of material results in smaller net section that might increase the stress level at a certain load or the stress range due to cyclic loading. Furthermore, it may induce the reduction of fracture and buckling resistances of a member. Usually, the loss of a section decreases the geometrical properties like the moment of inertia and the radius of gyration.

In addition, freezing of bearings may be caused by corrosion. Thus, high unpredicted stresses could be caused in the frozen elements and in the adjacent bridge members. When unexpected movements of the members occur, then, the most probable cause of that would be the built-up of corrosion products and this deformation could impact the behavior of the entire bridge.

## 4. Structural Model

A nonlinear model is used for structural analysis. The advantage of this method is that it accounts for stress and load redistribution that happens when the structure passes from an initial localized yielding in a member, to the ultimate capacity of that member, and finally to a complete bridge collapse. This analysis also quantifies the ability of a structure to resist additional loads even after initial yielding of a bridge member. Three types of nonlinearities are taken into consideration; namely: material nonlinearities, geometric nonlinearities, and boundary nonlinearities.

This study is involved in the analysis of 56 simply supported composite bridges, which are designed according to AASHTO LRFD Code (2004) Strength I limit state for flexure and shear. Here come some of the assumptions considered. Girder spacings are between 1.8m and 3m, and span lengths are between 18m and 42m. For each of the span lengths, three girder spacings are considered: 1.8, 2.4, and 3m. Furthermore, for every bridge, the longitudinal axis is assumed at right angle to the abutment. All bridges are two-lane bridges and have no skew. In all cases, the slab thickness is 225mm. In case of 4- and 6-girder bridges, the roadway width is about 9.9m and in case of a 5-girder bridge, it is about 10.5m. The deck overhang is around 0.45m. All bridges have I-beam girder sections and are designed according to HL-93 loading. The girders are made of structural steel A50. A typical cross section of a bridge is shown in Figure 4.

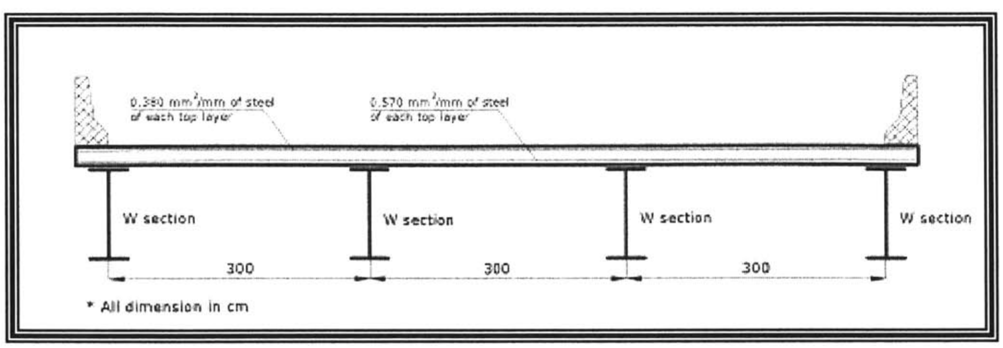



**Figure 4. Typical Cross Section of a Bridge (AASHTO LRFD 2004).**

All the analysis done hereunder was performed by Professor Nowak and his team at the University of Michigan in Ann Arbor. Finite elements models are capable of analyzing complex geometry modeling with a large number of elements. As a result, ABAQUS, which is a Finite Element Analysis program, was used to perform the structural analysis in this study. Before using the developed model in order to calculate the bridge resistance, it was calibrated first. Several experiments were done using ABAQUS models to find the degree of accuracy of the program. It was found out that the analytical results are within 5% of the experimental values. Thus, it was concluded that the FEM models used for the scope of the study are reliable within an acceptable margin of error.

The geometry of the bridge superstructure could be idealized in many different ways for the purpose of finite element analysis, but the choice of the structural model should be based on a logical compromise between the required accuracy and simplicity. In this case, isotropic solid elements are used to represent the deck and shell elements to model bridge girder flanges and slabs. This is found to be the most convenient model. Actually, 8-node hexahedral elements represent the concrete deck with three degrees of freedom at each node, 4-node quadrilateral structural shell elements with six degrees of freedom per node represent girder flanges and webs, and uniformly distributed layers of steel represent the reinforcement. It is assumed that there is a complete connection between girders and concrete slabs without the use of shear connectors. Despite that the available data require that edge-stiffening elements could perform favorably, these data are limited and cannot be always applied to all the structures. Secondary elements were not included in the model. Previous studies (Eamon and Nowak 2002 and 2004) proved that the effect of diaphragms on ultimate moment capacity is negligible, thus, the diaphragms are not included. For nonlinear models, the number of degrees of freedom ranges from 45,000 to 200,000 and the time needed for calculation ranged from 1.5 to 20 hours.

The materials used in the analysis are concrete, reinforcing steel, and structural steel. Some of the assumptions made are found hereunder. The elastic-plastic stress-strain relationship without hardening was used for structural and reinforcing steel, but the yield stress values are different. Concrete is modeled using an elastic-plastic material model with isotropic hardening for compression and crack detection surface for tension. In addition, the concrete damaged plasticity model with strain-softening behavior of cracked concrete, and large deformations and geometric nonlinearities are taken into account.

In the study, the load was applied in form of two side-by-side trucks. In the longitudinal direction, the trucks are placed to generate the maximum bending moment. Different transverse positions are also considered and the centerlines of the wheels of two adjacent trucks are placed no closer than 1.2m.

## 5. Load Models

The main load components for highway bridges are dead load, live load, dynamic load, environmental loads like temperature, wind, and earthquake, and other loads such as collision and braking. Dead loads and live loads, which consist mostly of truck traffic, are only considered in this study. Two statistical parameters are taken into account; namely: the bias factor $\lambda$, which is the ratio of mean to nominal value, and the coefficient of variation V. Dead Load, DL, is defined as the gravity load due to self-weight of the structural and nonstructural components permanently attached to a bridge. It consists of the weight of the girders, deck slab, barriers, wearing surface, sidewalks, and diaphragms, when applicable. Statistical parameters for dead load were found in the available literature (Nowak 1999). Four components are considered, namely: DL1, which is the weight of factory-made elements, DL2, which is the weight of cast-in-place concrete, DL3, which is the weight of wearing surface or asphalt, and finally, DL4, which is the weight of miscellaneous items like railing and luminaries. "All these components of the dead load are treated as random variables. $\lambda$=1.03 and V=0.08 for factory-made components like girders and diaphragms. $\lambda$=1.05 and V=0.10 for cast-in-place components like deck, barriers, and sidewalk. $\lambda$=1.03~1.05 and V=0.08~0.10 for miscellaneous components. Concerning asphalt wearing surface, it has a mean thickness of 75 mm and V= 0.25" (Nowak 1999).

Live load accounts for the vehicular traffic on the bridge. It does not just involve the weight of trucks, but also the span length, the axle loads, the traffic volume, the number of vehicles on the bridge, girder spacing, and mechanical properties of structural members, the distribution factor or the fraction of the total truck load per girder, and the truck position within the roadway, in other words, the curb distance. It is the sum of the static and dynamic portions. The dynamic load is related to the roughness of the surface, the dynamic properties of the bridge, and the suspension system of the vehicle. The dynamic load factor (DLF) is an equivalent static load that represents the dynamic component. It is defined as the ratio of dynamic strain to static strain or deflection. It is assumed that the GVW (Gross Vehicle Weight) is a random variable, but the percentage of the total load per axle and the axial spacing remain constant. Furthermore, the curb distance or the transverse position of the truck within the roadway is also considered as a random variable. The load model developed for the AASHTO LRFD Bridge Design specifications is used. The model is useful for the prediction of maximum moments and shear forces for bridge spans of different lengths and for a wide range of time periods, from 1 day to 75 years. The actual load data from a survey of heavily-loaded trucks were used to evaluate the live load parameters. Figure 5 shows the mean maximum moment per lane variation with respect to time for a span length of 24 m.

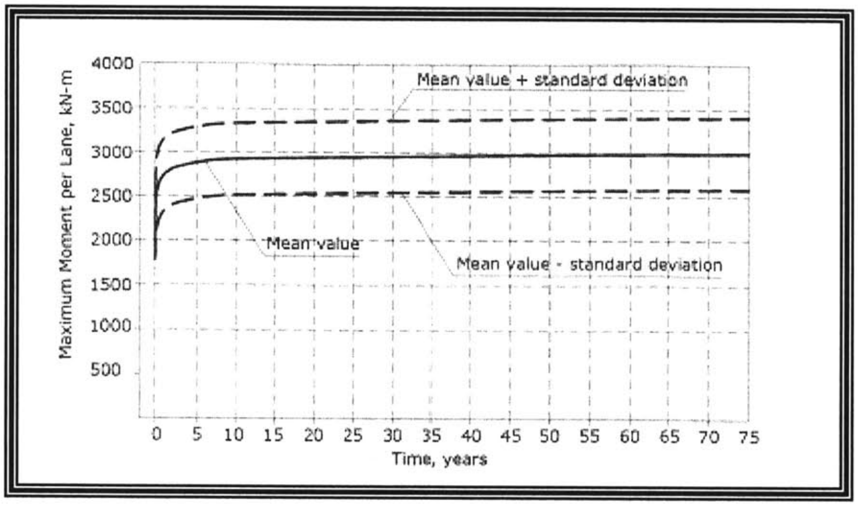



**Figure 5. Mean Maximum Moment over Time (Nowak, 2000)**

For a single-lane loaded case, the ratio of the mean maximum moment in 75 years to AASHTO HL-93 design moment ranges from 1.3 for shorter spans, which are around 10 m to 1.2 for longer spans, which are around 50 m, whereas the coefficient of variation, V=0.11 for all spans. For the two-lane loaded case, $\lambda$ for each truck varies from 1.2 for shorter spans to 1.0 for longer spans and the coefficient of variation, V=0.11 is also the same. Thus, $\lambda$ for the total moment on the bridge would be equivalent to 1.2*2 trucks = 2.4 and 1.0*2 = 2.0. Furthermore, V for each truck varies from 0.14 at 10 m to 0.18 at 50 m. It can be seen that at the beginning of service life, there is a drastic change in the mean value of the moment. But since during this period, corrosion is less likely to happen, this sudden change will not have an impact on the life-time reliability of a bridge.

According to field measurements (Kim and Nowak 1997 and Eom 2001), the DLF does not exceed 0.15 for a single truck and 0.10 for two heavily loaded trucks traveling side-by-side. Thus, the mean DLF is conservatively considered equal to 0.10 with a coefficient of variation equal to 0.80 (Kim and Nowak 1997 and Eoam and Nowak 2001).

The design live load is specified as the effect of a design truck, as it is shown in Figure 6, and a uniformly distributed load of 9.3 KN/m (AASHTO LRFD Code 2004).

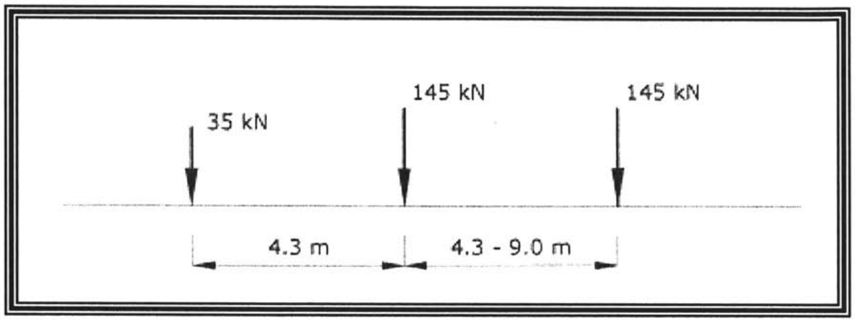



**Figure 6. Truck Configuration and Load Distribution (AASHTO LRFD 2004).**

The limit state is reached when any of the main members, such as girders, experiences an unacceptable deflection. For this reason, it is important to take into account the transverse position of a truck within the roadway width. All the statistical data used in this study were deduced from a survey of the lateral position of vehicles on interstate highways in Southeastern Michigan (Tantawi 1986). This model considers that every $15^{th}$ truck on the bridge is accompanied by another truck side by side. It also assumes that with each $10^{th}$ simultaneous occurrence of the scenario of side-by-side trucks, the truck weights are partially correlated, that means that $\rho = 0.5$, and with every $30^{th}$ occurrence, the truck weights are fully correlated, which means that $\rho = 1.0$. In addition, if multiple trucks are on a single lane, every $50^{th}$ truck is followed by another truck, knowing that the distance between these 2 trucks ranges from 4.5 m to 30 m, every $150^{th}$ truck is followed by a partially correlated truck by weight, and every $500^{th}$ truck is followed by a fully correlated truck.

# 6. Resistance Models

The load carrying capacity of a bridge depends on its geometry, in other words, its number of girders, girder spacing, and span length, its connections, and mostly the resistance of its elements. The element resistance is based on the material strength and dimensions. In addition, due to corrosion, the resistance R could be affected by a section loss. As a result, the component resistance is a random variable as a function of other parameters, which are also random variables. These parameters relate to uncertainties associated with material, dimension, fabrication, and methods of analysis.

For the scope of this study, the resistance parameters are depending on the available material and component tests. By simulations of moment-curvature relationship, flexural capacity of composite girders is established. Reliability is performed for 56 steel girders composite with a reinforced concrete deck slab. Conservatively, the effects of secondary elements like barrier and parapet were neglected. Statistical parameters of the load carrying capacity corresponding to composite steel girders were derived using Monte Carlo simulations. The mechanical properties of the girders and concrete deck slab were generated for every Monte Carlo run. For interior girders, ultimate moment and shear limit state equations are used. For moment-carrying capacity, the following parameters are taken: bias factor $\lambda$=1.12 and coefficient of variation $V_R$=0.10. On the other hand, for shear capacity, $\lambda$=1.14 and $V_R$=0.105. The basic parameters used in this study are summarized in Table 2.

| Type of Structure | λ | V |
|---|---|---|
| Noncomposite steel girders | | |
| Moment | 1.12 | 0.10 |
| Shear | 1.14 | 0.105 |
| Composite steel girders | | |
| Moment | 1.12 | 0.10 |
| Shear | 1.14 | 0.105 |
| Reinforced concrete T-beams | | |
| Moment | 1.14 | 0.13 |
| Shear | 1.20 | 0.155 |
| Prestressed concrete girders | | |
| Moment | 1.05 | 0.075 |
| Shear | 1.15 | 0.14 |

**Table 2. Statistical Parameters of Component Resistance (Nowak 1999)**

While designing a bridge, individual components constitute the basis of the process, thus, the performance of the entire structure is underestimated because redundancy and ductility are not considered. The failure of a component does not directly lead to the failure of the whole bridge. It is like a parallel electrical circuit, if one way is deteriorated, the current can pass by other ways. For this reason, bridge safety can be estimated using system reliability, which includes load sharing, load redistribution after member failure, and multiple failure paths. Thus, system reliability is a more precise measure of safety, but, in this situation, computations are much more complex than the ones done for the analysis of a single component, since there are additional parameters.

There are many modes of system failure. In this case, a bridge fails if a load exceeding its capacity is applied to it or if an excessive deflection of any of its main members is caused by live load. The deflection limit is 0.0075 of the span length and it causes a critically high level of deformation. To standardize these two issues, it was decided that the bridge resistance is expressed by a bending moment caused by live load. Thus, whichever of these two previous events occurs first, will be the one that controls. This method is independent from the truck configuration and weight. If a truck weight is used, it will cause some discrepancies especially in case of short spans. For example, a truck of a certain weight would cause a different bending moment, and consequently a different deflection for various axle configurations. Thus, since the bending moment is not the only factor that governs the failure of the bridge, the bridge resistance is also expressed in function of the gross vehicle weight (GVW). To solve the problem with axle configuration, all calculations are performed according to the AASHTO LRFD (2004) design truck. The load distribution and axle spacing are considered as deterministic values.

The configuration in which the structure is loaded with two side-by-side trucks was assumed. The longitudinal position of the trucks that corresponds to the generation of the maximum bending moment was found. The incremental loading method is used. The gross vehicle weight of trucks is gradually increased until the failure of the structure occurs. The performed analysis shows that the deflection limit happens prior to the ultimate failure of a bridge. Thus, the bridge resistance is taken as the live load moment causing the critical deflection. Since the vehicle could have different positions on the bridge, the bridge resistance is found for a member of transverse positions. Each of these positions has a probability of occurrence, which is used in the reliability analysis. As a result, the system resistance is equal to the expected value of the GVW, as it is shown in Equation 3.

$$R_{system} = \sum_{i=1}^{n} p_i * GVW_i \qquad \textbf{(Equation 3)}$$

where:

$GVW_i$ = gross vehicle weight of two side-by-side trucks causing deflection equal to 0.0075 of the span length corresponding to the ith transverse position of trucks

$p_i$ = probability of trucks in the ith position.

Figure 7 shows deterministic bridge load-deflection results for a span length of 30 m. Calculations were performed for intact bridges, where no corrosion has yet occurred and for three different corrosion rates. The time span considered is from 0 to 120 years, with 20 years interval.

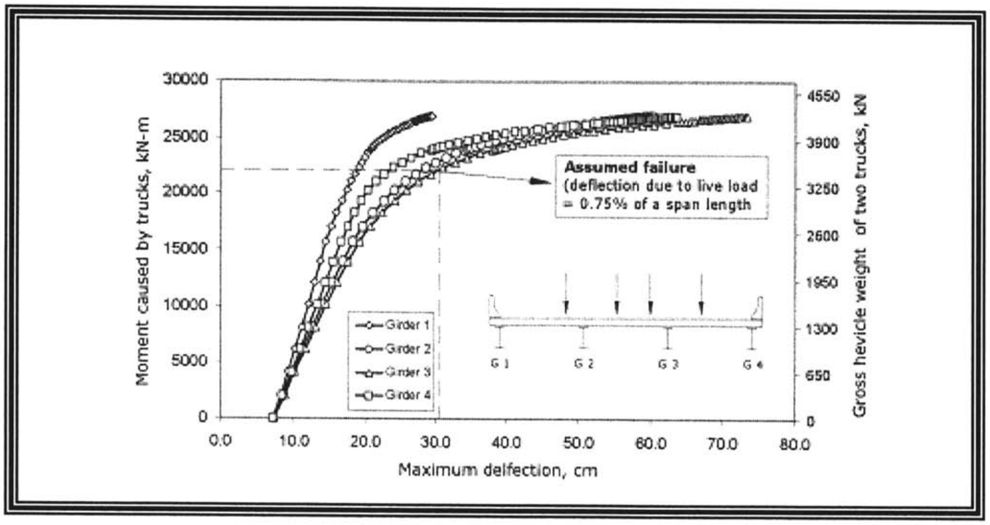



**Figure 7. Deterministic Bridge Load-Deflection Relationships for Span Length of 30 m ( Nowak 1999).**

Field testing and laboratory results of actual bridges show that the traditional design analysis models do not precisely predict structural behavior. One of the most considerable discrepancies in behavior can be seen in the prediction of ultimate capacity. Despite the difficulty of finding data and consequently the limitation of the existence of data, the measurements of girder bridge ultimate capacities are 1.4 to 3 times the predicted values by AASHTO. In the code, the capacity of a single girder is found and multiplied by the number of girders of the bridge in question.

These differences in results are obtained because current models do not take into consideration some important features of actual bridges that impact the structural behavior of the bridge. There are plenty of those features but the most important is the presence of secondary elements like barriers, diaphragms, and sidewalks. Despite the fact that recent studies and experiments show that these secondary elements could be beneficial for the behavior of the structure, even until ultimate capacity, they can not be relied on to act as structural components because they are not designed for this purpose.

## 7. Results and Analysis

56 bridges were considered in this study. They are all designed according to AASHTO LRFD Code (2004). All of them carry two lanes of traffic and are supported by 4, 5, or 6 longitudinal girders. According to the Code, the design is component-based and the girders were designed using the lightest possible sections. Moreover, just hot rolled sections were assumed. Consequently, all girder resistances were within 5% of the values specified by the Code, except for two bridges with span length 12 and 48 m, respectively. In this latter case, the girders were overdesigned by 12%.

12 random variables symbolize the variation in dimensions, loads, and material properties. They are: depth of reinforcing steel in concrete slab, yield strength of structural and reinforcing steel, weight of concrete, weight of steel, weight of asphalt, dynamic load in girders, weight of truck on the bridge, model uncertainty for flexure and shear in steel, modulus of elasticity of concrete, compressive strength of concrete, modulus of elasticity of steel, and transverse position of trucks within the roadway.

In order to calculate the girder reliabilities, the calibration procedure was used. For each span length and girder spacing, three design cases were considered, namely: with individual girders designed according to the code, and consequently, having reliability indices close to the target $\beta_T = 3.5$, with under-designed girders with reliability indices around $\beta_T = 2.0$, and over-designed girders with reliability indices close to $\beta_T = 4.5$. Ratios of the actual resistance to the minimum resistance for target girder reliability index $\beta_T = 3.5$ for moments and shear are shown in Figures 8 and 9.

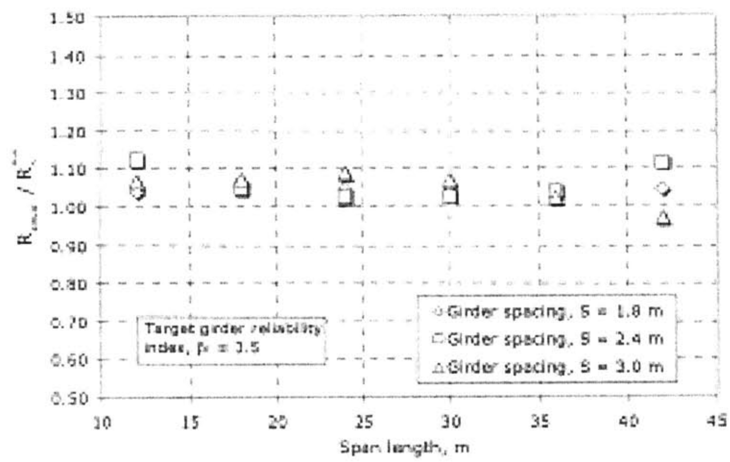



**Figure 8. Ratios of Actual Resistance to Minimum Required Resistance for Target Girder Reliability Index , Moments (Nowak 1997).**

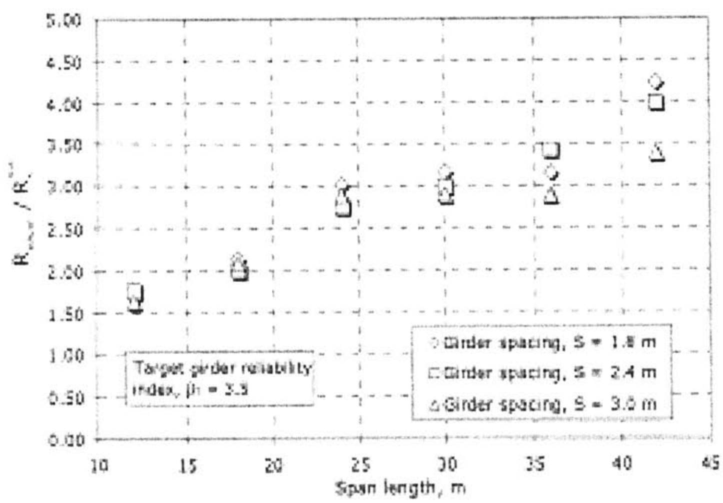



**Figure 9. Ratios of actual resistance to Minimum Required Resistance for Target Girder Reliability Index , Shear Force (Nowak 1997).**

Live load distribution factors specified in the code were applied. Preliminary calculations proved that shear does not govern in the design and reliability analysis. For moment capacity, the reliability indices are about 3.5, whereas for shear, they are ranging from 6.5 for short spans to 10 for long spans. Thus, the shear capacity was neglected in the system analysis.

The reliability of structures depends on a certain number of load and resistance parameters, which are represented by statistical parameters like the mean, bias factor, and coefficient of variation. However, some statistics could be based on insufficient data or subjective judgment. Thus, it is crucial to select the most sensitive parameters in the analysis, which will be the prime targets that will control the probability of failure.

As the number of random variables increases, the computational effort needed to determine the system reliability becomes more complex and more expensive. Thus, some adjustments should be made to accelerate the calculations without penalizing the accuracy of the results. In the system, each composite girder is considered as a single random variable. This will reduce by a considerable amount the number of random variables in the simulation. In addition, in the preliminary calculations, it was found that some random variables like the yield strength of reinforcing steel and depth of reinforcing steel in concrete have a negligible effect on the behavior of the structure. Thus, they were neglected. Furthermore, since a failure or collapse of the bridge occurs when an excessive deflection due to live load is applied to any of the main girders, the randomness of weight of concrete, steel, and asphalt do not really influence the system reliability of the bridge.

After minimizing the number of random variables to a reasonable level, the system reliability analysis was performed using the Rosenblueth 2n+1 method. This method was used to find the statistical parameters for bridge resistance. The effects of span length, girder spacing, and resistance correlations were investigated. In order to calculate the system reliability, two trucks were positioned longitudinally to maximize the midspan moment. Multiple curb distances were considered starting from 0.9 m from the curb, and then at 0.3 m intervals. For each transverse position, computations were repeated to find the ultimate load carrying capacity. Finally, the reliability index for the bridge was equal to the expected value of the reliability indices related to several transverse truck positions.

The procedure was repeated for several exposure times, taking into consideration the section loss due to corrosion. Live load was applied in the form of trucks. The load-deflection curves were determined by using a nonlinear FEM program.

Figure 10 shows the difference between the girder and the system reliability indices for 6 span lengths and girder spacing of 2.4 m. It also shows the size of the hot-rolled steel section used for main girders. In this case also, intact bridges are considered. The relation between these 2 indices is more elaborated in Figure 11.

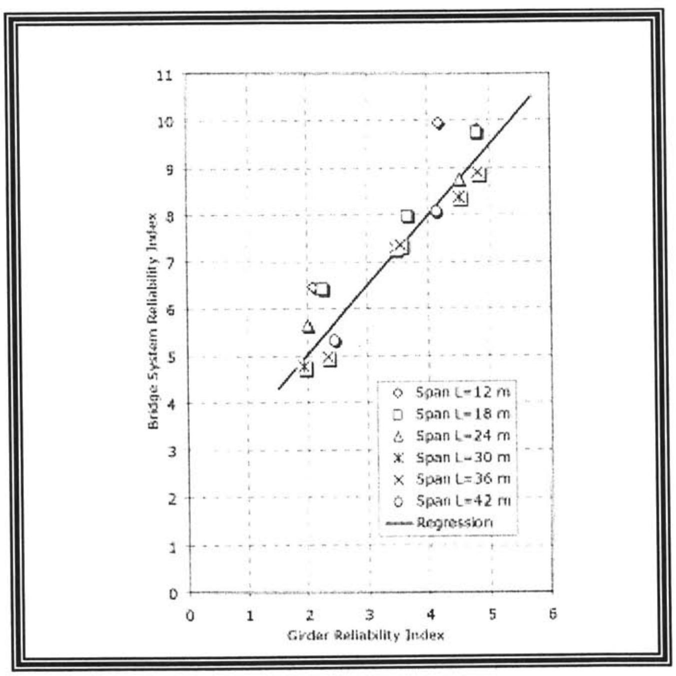



**Figure 10. Girder Reliability Index versus System reliability Index for Different Spans and Girder Spacing of 2.4 m (Nowak 1999).**

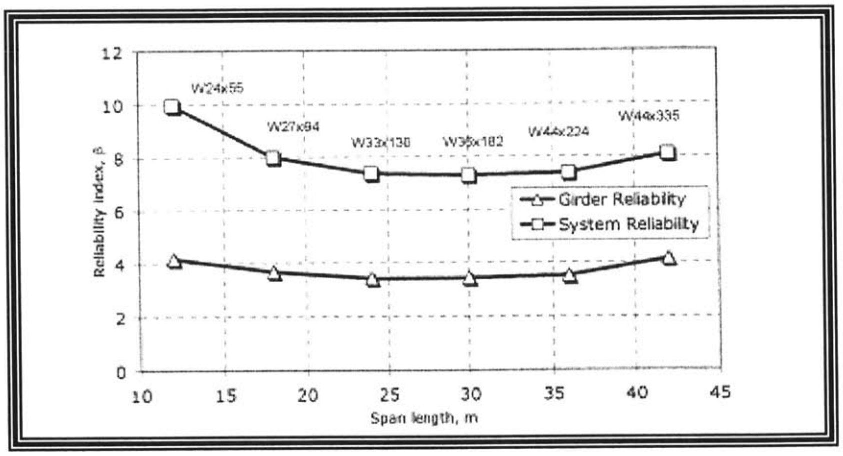
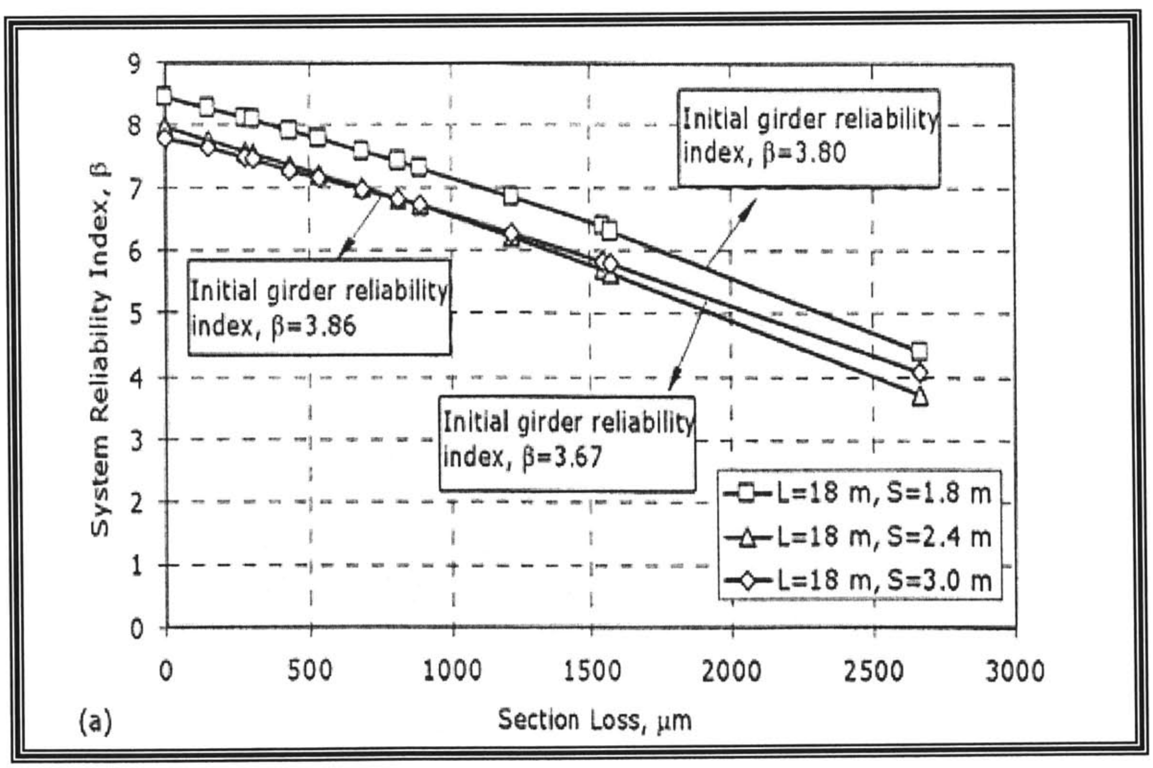



**Figure 11. Girder reliability Indices and Corresponding System reliability Indices for Different Spans and Girder Spacing of 2.4 (Nowak 1999).**

Figures 12(a) and 12(b) show the relation between the system reliability index $\beta$ and the section loss for space lengths of 18 and 42 m.



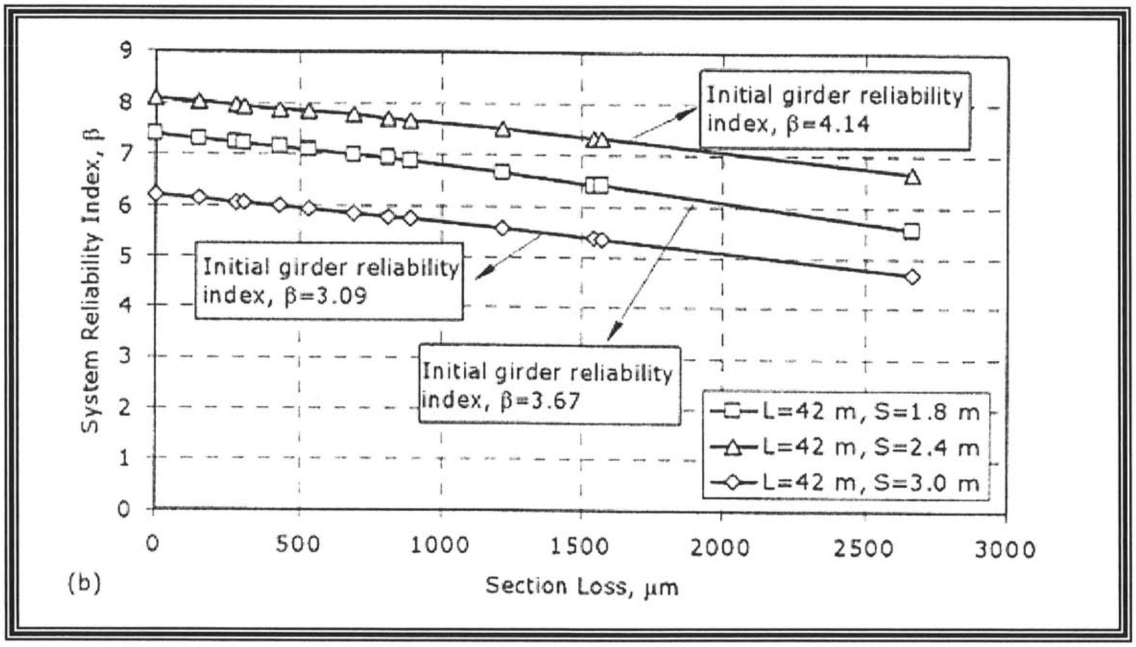



**Figure 12 (a)&(b). System Reliability Index versus Section Loss (Nowak 1999)**

Figure 13 shows the relationship between the system reliability index and the exposure time for three different corrosion rates. All the bridges considered are composite steel structures with 42, 30, and 18 m span lengths, respectively.

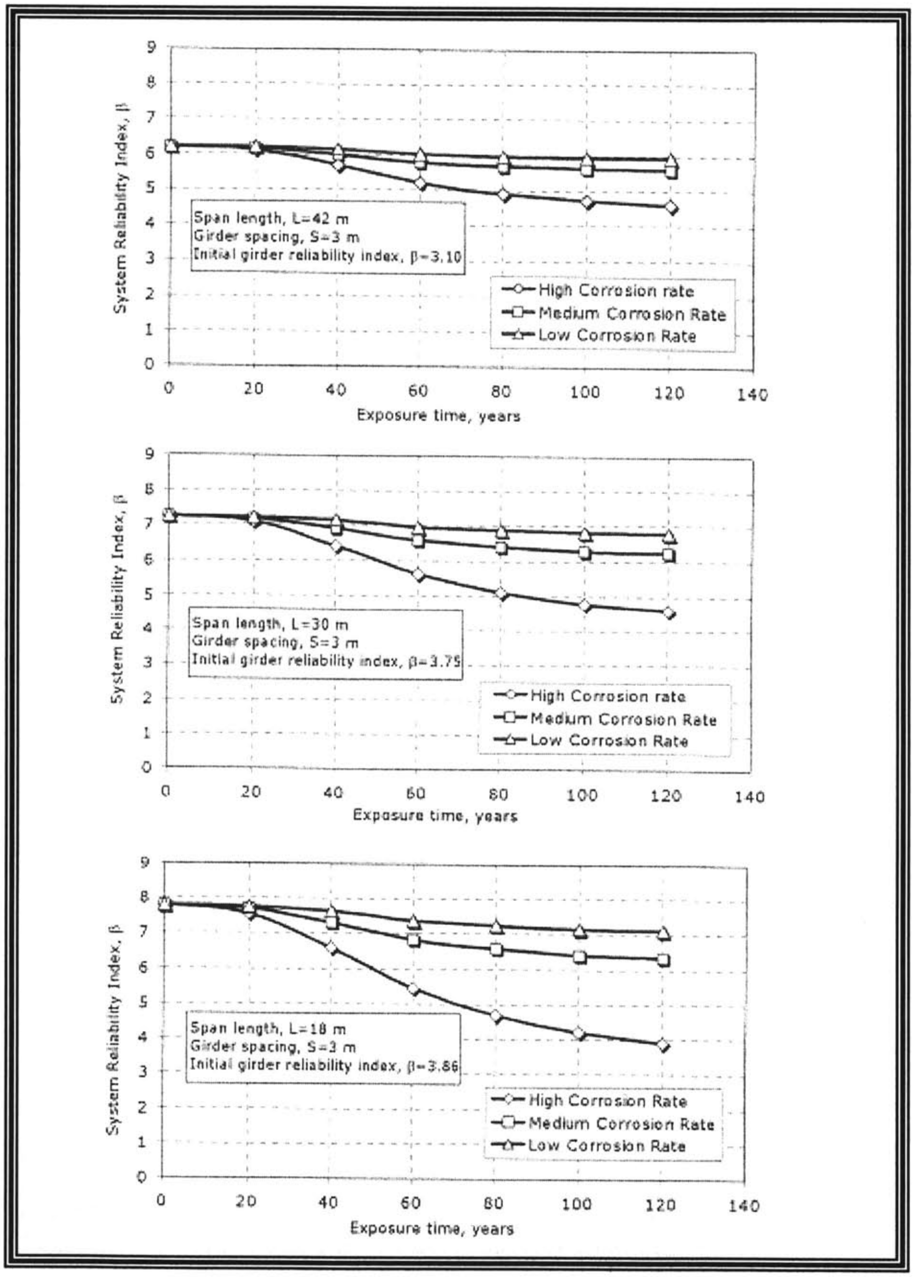



**Figure 13. System Reliability versus Exposure Time for spans 42, 30, and 18 m and Girder Spacing of 3 m (Nowak 2000).**

Furthermore, Figure 14 shows the system reliability index versus the girder reliability index for three bridges. W44*335 girders were used for 42 m span length bridges, W44*198 for 30 m span length bridges, and W30*108 for 18 m span length bridges. All these bridges considered have 4 girders spaced at 3 m. The initial reliability indices found for the interior girders are 3.10, 3.75, and 3.86, respectively.

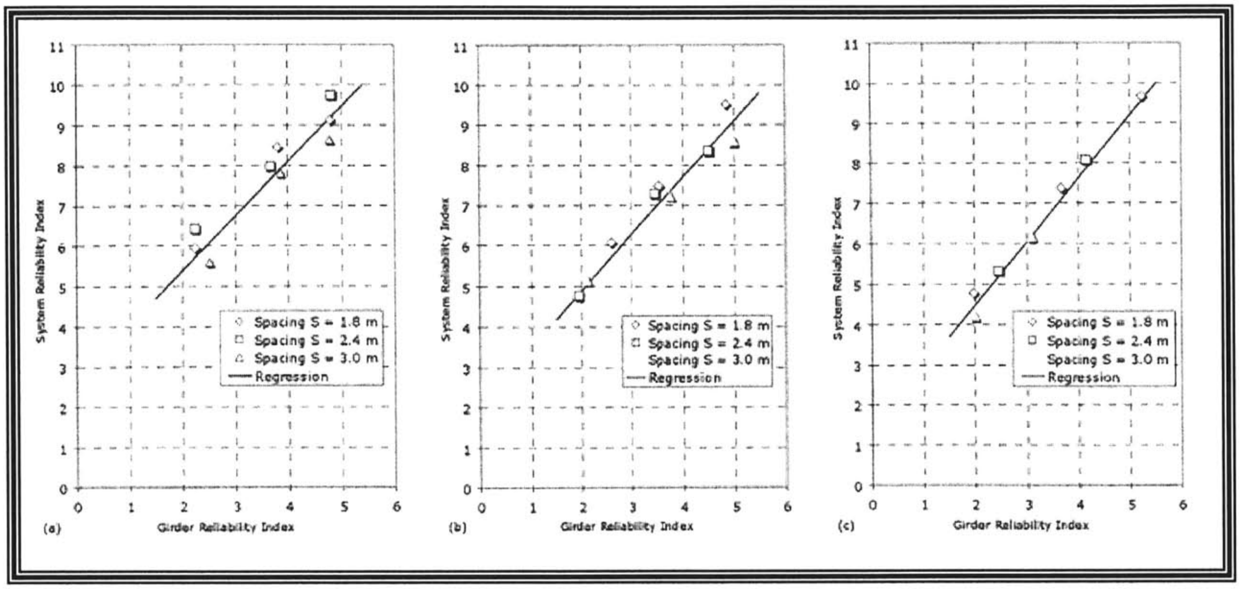



**Figure 14. System Reliability Index versus Girder Reliability Index for Different Girder Span lengths (Nowak 2000)**

From the results obtained, it was concluded that in case of short bridges, the decrease in system reliability due to corrosion is more significant. Moreover, for a small girder spacing, the decrease in system reliability due to corrosion is greater than the one for larger girder spacing. Considering that the same thickness loss occurs per year, it is realized that smaller sections lose more capacity than larger ones.

The effect of correlation in terms of the coefficient of correlation $\rho$ between girder strengths on the system reliability was also considered. In this case, just the yield stress of structural steel, $F_y$, was taken into account. Since practically, no data was available for the calculation of $\rho$, four cases were analyzed; namely: a full correlation $\rho$=1.0, a medium correlation $\rho$=0.6 and $\rho$=0.4, and no correlation $\rho$=0.0. Figure 15 shows the relationship between the system reliability index versus the span length for different degrees of correlation.

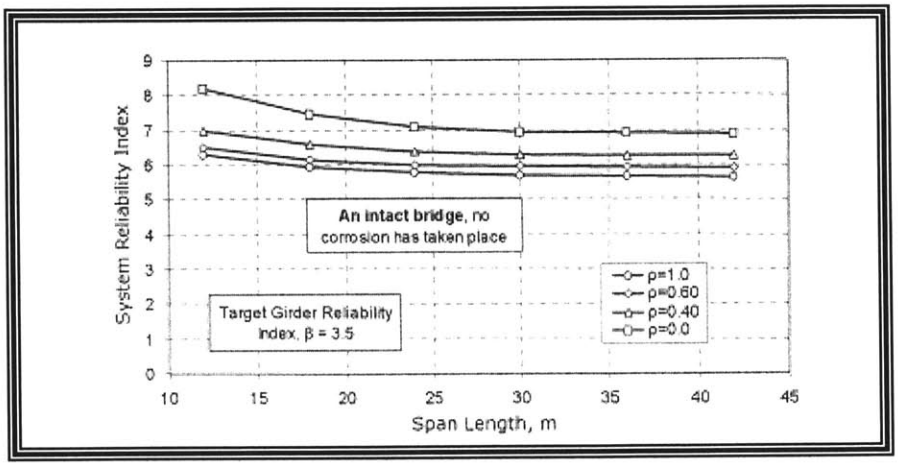



**Figure 15. System Reliability Index versus the span length for Different Degrees of Correlation (Nowak 2000).**

The difference between system reliability and girder reliability is due to the effect of redundancy. As the correlation between the girder strengths increases, the system reliability index decreases. In correlated cases, girder bridges can be considered as parallel systems.

# 8. Conclusion

Reliability is a rational measure of structural performance in the design of bridges and assessment of existing ones. In this study, the time-variant system reliability profiles were found for different rates of corrosion. In an environment where the rate of corrosion is low, the bridge life-performance is not affected by this reaction. But in high corrosion rate environments, the safety of the structure is considerably affected, it could be reduced over 120-year life. It was also found that system reliability is much higher than girder reliability, especially when there is no correlation between girder resistances. The difference between system and girder reliabilities is related to the bridge redundancy.

Sensitivity analysis highlights the importance of the strength and dimensions of steel in the analysis, whereas parameters related to concrete slab are of smaller importance. Reliability indices for the whole bridge were compared to reliability indices for individual girders. It was observed that the ratio of $\beta_{system} / \beta_{girder}$ decreases when $\beta_{girder}$ increases; it is equal to 2 for $\beta_{girder}$=1 and to 1.3 for $\beta_{girder}$=6.

Correlation between girders plays a crucial role in the analysis. For intact bridges with fully correlated girder resistance, it was found that the system reliability is smaller by 10 to 30% depending on the girder reliability, span length, and girder spacing. It was also found that the higher the girder reliability, the smaller the influence of girder resistance correlation on the capacity of the entire bridge. Thus, for highly corroded environments, the correlation between girder resistances will have a bigger influence. Furthermore, over time, the bridge reliability decreases much more for shorter spans compared to longer spans. The goal of the paper was to show that it is more beneficial to consider a bridge structure as a system.

This study was limited to simple span bridges, thus, continuous structures were not analyzed. For this latter type of structures, corrosion will have smaller influence on life-time performance because the girders of continuous bridges experience lower maximum load effects. However, more research should be conducted concerning this issue. Even in this thesis, the results were based on analysis, thus, more research should be done to validate the accuracy of this model.

# 9. References


AASHTO *LRFD Bridge Design Specifications*, American Association of State Highway Transportation Officials, Washington D.C., 2004.

Albrecht, P., and Naeemi, A.H., *"Performance of weathering steel in bridges,"* National Cooperative Highway Research Program, Report 272, TRB, Washington DC, July 1984.

ASCE, *"2005 report card for America's infrastructure."* Reston, VA. <www.asce.org/reportcard>, 2005.

Eamon, C., and Nowak A.S., *"Effects of edge-stiffening elements and diaphragms on bridge resistance and load distribution,"* ASCE Journal of Bridge Engineering, 2002.

Eamon, C., and Nowak, A.S., *"Effects of secondary elements on bridge structural system reliability considering moment capacity,"* Structural Safety, 2004.

Ellingwood, B., Galambos, J.V., MacGregor, J.G., and Cornell, C.A., *"Development of a Probability Based Load Criterion for American National Standard A58."* NBS Special Publication 577, Washington D.C., National Bureau of Standards, 1980.

Eom, J. and Nowak, A.S., *"Live Load Distribution for steel Girder Bridges,"* Journal of Bridge engineering, ASCE, Vol.6, No. 6, pp. 489-497. 2001.

Estes, A. and Frangopol, D.M., *"Repair optimization of highway bridges using system reliability approach,"* Journal of Structural Engineering, ASCE, 125(7), 766-775, 1999.

Faber, M.H., Val, D.V., and Stewart, M.G., *"Proof load testing for bridge assessment and upgrading,"* Engineering Structures, 2000.

Fontana, M.G., *"Corrosion Engineering,"* McGraw-Hill Book, New York, Third Edition, 1986.

Galambos, T.V., and Ravindra, M.K., *"Load and Resistance Factor Design,"* Journal of Structural Division, ASCE, ST9, Proc. Paper 14008, Sep. 1978.

Ghosn, M., and Moses, F., *"Redundancy in Highway Bridge superstructures,"* NCHR Report 406. Transportation research Board, Washington, D.C., 1998.

Kim, S-J., and Nowak, A.S., *"Load Distribution and impact Factors for I-Girder Bridges,"* ASCE Journal of Bridge Engineering, Vol.2, No. 3, August 1997.

Kayser, J.R., *"The effect of corrosion on the reliability of steel girder bridges,"* Phd Thesis, Department of Civil and Environmental Engineering. University of Michigan, Ann Arbor, MI, 1988.

Komp, M.E., *"Atmospheric Corrosion Rating of Weathering Steels. Calculations and Significance."* Material Performance, 1987, Vol.26(7).

Kulicki, J., Prucs. Z., Sorgenfrei, D., Mertz, D., Young, W., *"Guidelines for Evaluating Corrosion Effects in Existing Steel Bridges."* NCHRP Report Number 333, Transportation Research Board, Washington, DC, 1990.

Liu, Y. and Moses, F., "Bridge design with reserve and residual reliability constraints." Structural Safety, 11(1), 29-42, 2001.

McCrum, R., Arnold, C.J., and Dexter, R.P., *"Current Status Report Effect of Corrosion on Unpainted Weathering Steel Bridges,"* Testing and Research Division, Report No. R-1255, Michigan Department of Transportation, 1985.

Nowak, A.S., *"Calibration of LRFD Bridge Design Code,"* NCHRP Report 368, Transportation Research Council, Washington, D.C., 1999.

Nowak, A.S., and Collins, K.R., *"Reliability of Structures,"* McGraw-Hill, New York, 2000.

Park, C-H., *"Lifetime Reliability Model for steel Girder Bridges,"* in Safety of Bridges, ed. P.C. Das, Thomas Telford Publishing, London, 1997.

Park, C-H., and Nowak, A.S., *"Lifetime Reliability Model for steel Girder Bridges,"* in Safety of Bridges, ed. P.C. Das, Thomas Telford Publishing, London, 1997.

Safar, V., and Nowak, A.S., *"Field evaluation of steel Girder Bridge,"* Transportation Research Record, 1997.

Safar, V., and Nowak, A.S., *"Proof Load testing of Deteriorated Steel Girder Bridges,"* ASCE Journal of bridge Engineering, May 1998.

Tabsh, S.W., and Nowak, A.S., "Reliability of Highway Girder Bridges," Journal of Structural Engineering, ASCE, Vol. 117, No.8, August 1991.

Tantawi, H.M., *"Ultimate Strength of Highway Girder Bridges,"* PhD dissertation, Department of civil Engineering, University of Michigan, Ann Arbor, Michigan, 1986.

Zhou, J., *"System Reliability Models for Highway Bridge Analysis,"* PhD Dissertation, Department of Civil and environmental Engineering, University of Michigan, Ann Arbor, MI.,1987