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STRUCTURAL RELIABILITY ANALYSIS OF CORRODED
STEEL GIRDER BRIDGE

by

Mohammed Shaalan Al Badran

A THESIS

Presented to the Faculty of

The Graduate College at the University of Nebraska

In Partial Fulfillment of Requirements

For the Degree of Master of Science

Major: Civil Engineering

Under the Supervision of Professor Andrzej S. Nowak

Lincoln, Nebraska

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STRUCTURAL RELIABILITY ANALYSIS OF CORRODED STEEL GIRDER BRIDGE

Mohammed Shaalan Al Badran, M.S.

University of Nebraska, 2013

Advisor: Andrzej S. Nowak

The natural aging, environmental impact, and appropriate design are certainly playing a serious role in safety of the bridges function. The functional safety of bridge constitutes a prime element of the transportation system serving commuters, passengers and freight. According to Federal Highway Administration, the steel bridges share represents a large amount of the United States' bridges. In last century, the collapse of both the Silver Bridge at Point Pleasant, WV over Ohio River on December 15, 1967 and over the Mianus River Bridge in Connecticut on June 28, 1983 showed that steel corrosion has been the leading reason in these disasters. The corrosion occurred when water attacked the steel surface especially in inadequately protected structure from environmental influence or shortcoming in bridge structure. Corrosion form a uniform thickness loss or concentrated pitting depend on size of the affected area, and the location of bridge as which type of the environmental action takes place, and improper design of bridge. To develop a significant deterioration that will be enough to produce a material loss of steel component, which may take several years to happen.

Bridges are designed according to AASHTO-LRFD design code, which stands for the American Association of State Highway and Transportation Officials. To evaluate the effect of corrosion action, a corroded composite steel girder in simple span bridge located in the State of Nebraska was considered for this study. The research focused on ultimate limit state (moment and shear) and serviceability limit state (deflection of beam).

In this study, the corrosion has been modeled for moment and shear stresses with different models and considering three types of corrosion, low, medium and high. Moreover, the steel girder has been modeled using ABAQUS advanced finite element

software. Materials specifications, resistance model, and load models were designed using structural reliability techniques. The reliability indices have been calculated to measure the structure performance. The conclusions of this thesis demonstrated that the moment and shear capacity of the analyzed composite steel girder bridge might decrease. According to this conclusion, the live load capacity has also decreased, while the deflection score has increased.

DEDICATION

This thesis is dedicated to

My wife Aseel whose support, love,
and nenerous scarifies made it possible
with our three cutie ones, Zainab, Ahmed, and Alhussain

My father, mother, sister, and brother

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CHAPTER 1

INTRODUCTION

1.1 INTRODUCTION

The health and prosperity of the transportation infrastructure is an important measure of national growth. The transportation network plays a pivotal role in the movement of people, goods, and services. Highway bridges are one of the main elements of the transportation infrastructure. According to Federal highway Administration, (FHWA), there are 607,380 highway bridges in the United States. Moreover, the percentage of structurally deficient bridges and bridges with significant structural problems is estimated to be 151,497 bridges in United States. Furthermore, the structurally deficient bridges are 66,749. Also, the average age of the nation's bridges is 42 years. Steel bridges have been widely constructed due to dynamic functionality of steel, effective, sustainable, and economical material in comparison to other construction materials.

Water-related corrosion is recognized as a leading cause of the deterioration of steel bridges. Corrosion reduces cross section dimensions due to material loss; as a result, this may lower the resistance of steel beams or connections to internal induced by bridge

loading (live load). Furthermore, the carrying of live load may increase because of growing freight over the years, which is normally.

The modern AASHTO bridge design code has been promoted to the Load and Resistance Factor Design LRFD rather than the allowable stress design ASD. The new design has developed based on historical data from constructed bridges, which employed in probability theory and, mainly reliability procedures. Progress in reliability theory resulted in a combination of multiple variables for loads and resistance LRFD that have yielded safer and more reasonable designs. In the current study, a corroded steel girder bridge has used to determine the effect of corrosion on bridge performance, which was evaluated using reliability index.

1.2 PROBLEM STATEMENT

Bridges have a unique role in connecting the transportation network, and are costly structures. Over the years, bridge safety and durability has been enhanced through architecture design. About 33% of all bridges in United States are made of steel. Composite steel girder bridges are a major component of the bridge infrastructure, and contains multiple steel girders that support the bridge deck and transfer loads to piers or abutments. The corrosion of steel remains a concern, necessitating protection of the girder through painting or utilizing another type of steel, such as weathering steel, which is more resistant to corrosion. Composite steel girders are exposed to environmental influences; in many cases, the bridge has a defect in some part of the deck, which allows liquid to leak to the girders. This will impact the substance of steel girders by different

amounts daily, depending on the position of the member itself (external or internal), the protective treatment to steel, the influence of de-icing and traffic volume, and the type of material used (weathering steel, carbon steel, etc.). Steel I girder corrosion will cause deterioration on composite steel girder bridges, especially the carbon steel. . Deterioration occurs due to the nature of the surrounding environment, and therefore depends upon the location of the bridge. The primary cause of corrosion is the accumulation of water and salt (from marine environments or deicing) on steel surfaces. The both corrosion penetration and fatigue problem have been considered as main reason of steel bridge deterioration (Czarnecki, 2006).

The LRFD design is relatively new. The safety approach has been considered by applying reliability of structures to estimate the load and resistance factors depending on the available background data of structures to develop AASHTO bridge design code. The evaluation of the effect of corrosion on performance using reliability analysis has been conducted according to AASHTO LRFD (2012) design code. This research considered strength or ultimate limit states (ULS) for shear and moment. In addition, serviceability limit state (SLS), which considers the deflection of steel beam was conducted in this study. The deflection limit according AASHTO LRFD is equal to $(L/800)$, where L is the span length of the bridge.

1.3 RECENT STUDIES

The investigation of recent studies can provide reasonable background knowledge also, the needed information related to the corroded composite steel girders for deflection limit state, strength limit state, corrosion penetration and

reliability procedure. The literature review researches on composite steel girder corrosion includes the work of Czarnecki (2006), Kayser (1988), and Park (1999); and reports by the NCHRP (1989).

1.4 OBJECTIVES OF THE RESEARCH

The proposed study aims to evaluate a corroded composite steel girder bridge by developing models of corrosion deterioration that indicate clearly and realistically the existing composite steel girder. Corrosion models are developed linearly to estimate the effect of steel loss on internal force capacity, and to determine bridge rating factors. Furthermore, corrosion deterioration models of a composite steel girder are promoted to represent non-linear analysis using advance finite element software (ABAQUS). A reliability model is then developed based on the results of the analysis using ABAQUS software. The reliability models will aid in the examination of the effect of ultimate limit state and the deflection limit state effect on the design and performance of a composite steel girder. The linear models will include calculation of the deformed cross section, yield strength, section module, and other needed dimension that involves in calculations (depth of the web, thickness, and width of the flange and length of bearing), and composite section analysis. On the other hand, nonlinear models include dead load, live load (HS20-44 AASHTO truck), dynamic factor, material properties, corrosion time dependent models, and boundary conditions.

The AASHTO-LRFD 2012 limit state functions have been compared with time dependent model results at zero, 20, 40, 60, 80, and 100 years. The impact on performance limit state has been investigated. Reliability figures of moment shear, and

deflection based on its indices functions with steel yielding at above times dependent have been established. The use of limit state functions for strength and deflection represents a reasonable engineering decision, which is an advantage for a rational research.

1.5 SCOPE OF THE RESEARCH

The actual work to complete this research involves the following tasks:

- 1- Completion of a literature review of the design of simple span bridge, corrosion of composite steel girder, strength limit state analysis, deflection limit state, structural reliability, simulation of composite steel girders, and material properties.
- 2- Analysis of corrosion penetration pattern.
- 3- Corrosion linear models for shear, moment, and bearing of structure member (girder) have been developed with age-related deterioration.
- 4- Modeling the composite steel girder using ABAQUS finite element analysis software to evaluate the effective stresses distribution with girder geometry.
- 5- Determine the effects of corrosion damage on composite steel girder by using reliability analysis procedure.
- 6- Evaluation of corroded composite steel girder bridge depending on reliability results in terms of reliability indices for (ULS), which are moment shear limit states. And (SLS) for deflection limit state.

The analysis was carried out for a simple span bridge of corroded composite steel girder in the state of Nebraska. The Nebraska Department of Roads (NDOR) – Bridge Division, provided information on the bridge including

blueprints. The analysis focused on the corrosion of a composite steel girder and its effect on the bridge performance. The AASHTO-LRFD limit state function for shear, moment and deflection (live load only) was employed for determines the reliability indices. The reliability approach calculation was considered in this research for decision making and results. Corrosive behavior of composite steel girder has been determined depends on monitoring and evaluated by 2 methods. The second method used to refine the results of first one.

1.6 ORGANIZATION OF THE THESIS

Chapter 1 of this thesis provides an introduction, including the problem statement, a listing of recent major studies pertaining to the current research; research objectives; and details on the scope of this research.

Chapter 2 describes the structural reliability theory. The characteristics of random variables and probability distributions with their types are presented. Limit state and structural reliability index with Monte Carlo simulations are explained.

Chapter 3 presents an introduction on the corrosion of structural steel components. Corrosion in steel and types of corrosion are explained.

Chapter 4 provides a description of load and resistance models.

Chapter 5 presents the girder corrosion model with the calculation of steel girder corrosion penetration, loss of material with the age, moment capacity, stiffness capacity, shear capacity and bearing capacity.

Chapter 6 presents the ABAQUS finite element analysis model.

Chapter 7 presents the reliability analysis for moment, shear and deflection (serviceability load).

Chapter 8 presents conclusions of this research, with suggestions for future work.

CHAPTER 2

STRUCTURAL RELIABILITY THEORY

2.1 INTRODUCTION

The profound changes in civil engineering over the last few decades were reflected by ideas of uncertainty recognized in civil engineering today. The recent advancement in statistical modeling have been provided by civil engineering by an increased power of making decisions under different degrees of uncertainty. Confidence should be placed in the ability of engineer emphasize any existing information when it is required because it is impossible to get sufficient statistical data for any existing problem. The estimation of structural reliability would be related to specify failure modes because it is impossible to examine all failure modes for structures, therefore representative failure scenarios should be chosen.

In designing structures, civil engineers use a probabilistic evaluation for reliability instead of using their desirable performance under applied loads during construction and service.

2.2 MAIN DESCRIPTIONS OF A RANDOM VARIABLE

A random variable is a variable whose value is uncertain or nondeterministic, such as the strength of steel or concrete or any other material or physical quantity. There are two types of random variables, a discrete random variable is defined as an integer value, which its probability is given by the probability mass function (PMF). The other type of a random variable is the continuous random variable defined as a value of an interval of real numbers, which its probability is given by the probability density function (PDF).

If the form of the distribution function and its associated parameters were specified, the statistic parameters of a random variable would be described completely. The probabilistic characteristics of a random variable may be determined in terms of the mean value, variance and standard deviation , and coefficient of variation as explained:

2.2.1 Mean Value

The mean value can be defined as the first moment about the origin.

For a continuous random variable, the mean is computed as:

$$\mu = \int_{-\infty}^{+\infty} xf_x(x)dx \quad (2.1)$$

For a discrete random variable, the mean μ , is given by:

$$\mu = \sum_{i=1}^n x_i P_x(x_i) \quad (2.2)$$

If all n observations are given equal weights of $[P_x(x_i) = 1/n]$, then the sample mean \bar{X} would be the average of the observed values for a discrete random variable, as given by:

$$\bar{X} = \frac{1}{n} \sum_{i=1}^n x_i \quad (2.3)$$

2.2.2 Variance and Standard Deviation

The variance is the second moment about the mean, denoted by σ^2 . The standard deviation is the square root of the variance, denoted by S .

For a continuous random variable, the variance is computed as:

$$\sigma^2 = \int_{-\infty}^{+\infty} (x - \mu)^2 f_x(x) dx \quad (2.4)$$

For a discrete random variable, the variance is given by:

$$\sigma^2 = \sum_{i=1}^n (x_i - \mu)^2 P_x(x_i) \quad (2.5)$$

For a sample, the standard deviation is given by:

$$S = \sqrt{\frac{\left(\sum_{i=1}^n x_i^2\right) - n(\bar{x})^2}{n-1}} \quad (2.6)$$

2.2.3 Coefficient of Variation

The coefficient of variation is denoted by V, and is defined as the value of standard deviation divided by the mean as shown below:

$$V = \frac{\sigma}{\mu} \quad (2.7)$$

2.3 PROBABILITY DISTRIBUTIONS

There are two types of probability distributions, classified as discrete and continuous distributions. In this thesis, only the most common types of continuous distributions, as normal and lognormal, are presented. Further details about other distribution types are found in (Nowak and Collins 2013).

2.3.1 Normal Distribution

The normal distribution is the most widely used probability distribution, also known as Gaussian distribution.

The probability density function (PDF) for a normal distribution is given by:

$$f_x(x) = \frac{1}{\sigma\sqrt{2\pi}} \exp\left[-\frac{1}{2}\left(\frac{x-\mu}{\sigma}\right)^2\right] \quad (2.8)$$

where μ , is the mean value and, σ , is the standard deviation, which are the parameters of the distribution.

The cumulative distribution function of the normal distribution is given by:

$$F_x(x) = \int_{-\infty}^x \frac{1}{\sigma\sqrt{2\pi}} \exp\left[-\frac{1}{2}\left(\frac{x-\mu}{\sigma}\right)^2\right] dx \quad (2.9)$$

The PDF and CDF of a normal distribution for a random variable are presented in Figure 2.1.

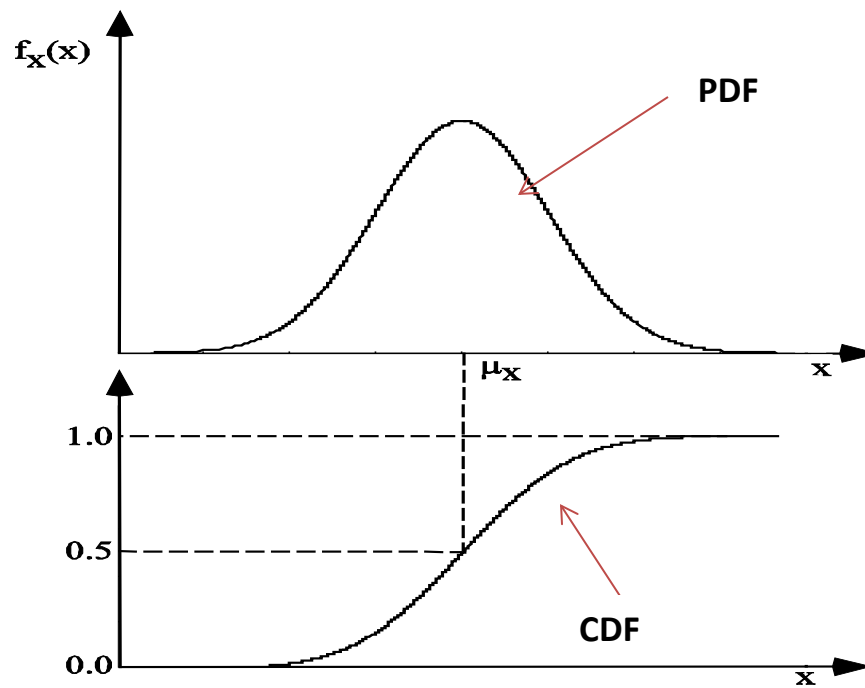


Figure 2.1: PDF and CDF of a Normal Distribution

2.3.2 Lognormal Distribution

The lognormal distribution is used for general reliability analysis, such as the random variable X is lognormal distributed if the logarithm of the random variable is normally distributed, as $Y = \ln(X)$.

The probability density function (PDF) of the lognormal distribution is given by:

$$f_x(x) = \frac{1}{x\sigma_{\ln(X)}} \phi\left(\frac{\ln(x) - \mu_{\ln(X)}}{\sigma_{\ln(X)}}\right) = \frac{1}{x\sigma_Y\sqrt{2\pi}} \exp\left[-\frac{1}{2}\left(\frac{\ln(x) - \mu_Y}{\sigma_Y}\right)^2\right] \quad (2.10)$$

where, μ_Y and σ_Y are parameters of lognormal distribution.

The cumulative distribution function (CDF) of the lognormal distribution can be determined as:

$$F_x(x) = F_Y(y) = \Phi\left(\frac{y - \mu_y}{\sigma_y}\right) \quad (2.11)$$

The PDF and CDF of the lognormal distribution are presented in Figure 2.2.

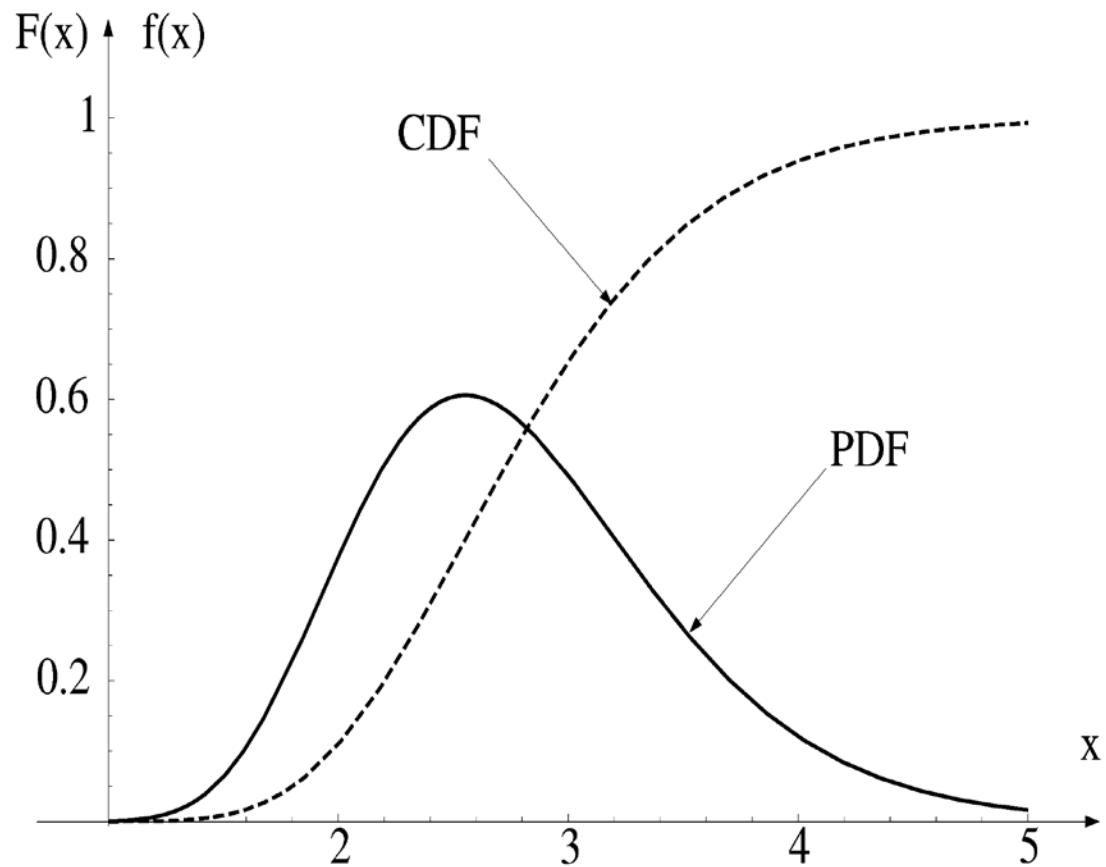


Figure 2.2: PDF and CDF of Lognormal Distribution

2.4 LIMIT STATE FUNCTION

A limit state function is a boundary between desired and undesired performance of a structure. There are three types of limit states in the reliability of structures, as presented below:

1. Ultimate limit states (ULSs) represent the collapse of the structure due to loss of structural capacity.

2. Serviceability limit states (SLSs) represent the failure states for normal operation in service condition.
3. Fatigue limit states (FLSs) represent the loss of strength for a structural component under the condition of repeated loading.

The limit state function can be defined as the performance function of a structure when all loads are assigned to the variable Q , and the capacity of the structure is assigned to the resistance, R . The formulation of the limit state function is expressed as:

$$g(R, Q) = R - Q \quad (2.12)$$

2.5 RELIABILITY INDEX

The reliability index, β , can be defined as the safety index. Then, the reliability index can be calculated from Cornell (1967, 1969) as:

$$\beta = \frac{\mu_R - \mu_Q}{\sqrt{\sigma_R^2 + \sigma_Q^2}} \quad (2.13)$$

When, $g(R, Q) < 0$, this represents failure of structure or unsafe performance, therefore, the probability of failure can be expressed as:

$$P_f = P((R-Q)<0) = P(R<Q) = P(g<0) \quad (2.14)$$

Under the assumption of considering the limit state function as normally distributed and the random variables are uncorrelated, the reliability index, related to probability of failure, can be given as:

$$P_f = \Phi(-\beta) \quad (2.15)$$

where, Φ is the standard normal distribution function.

2.6 MONTE CARLO SIMULATIONS

The Monte Carlo method is most widely used in science and engineering practice for several years. Rather than utilizing a more limited closed-form solution (one that assumes like distributions for load and resistance), Monte Carlo simulation provides a powerful method to solve the problem of determining the failure rate numerically.

The typical application of Monte Carlo simulation for bridge-structures reliability as reported in the literature (Thoft-Christensen and Baker 1982, Allen, et al. 2005, Nowak and Collins 2013) is quite simple, as follows:

- 1- Generation of uniformly distributed random variables u_1, u_2, \dots, u_n , which are between 0 and 1.

- 2- Calculation of the standard normal values using generated numbers, including the types of distributions with their statistical parameters (mean and standard deviation values) for each design variable.
- 3- Calculation of the standard random number (z_i) from the following equation as :

$$z_i = \Phi^{-1}(u_i) \quad (2.16)$$

where, Φ^{-1} is the inverse of the standard normal cumulative distribution function.

- 4- Using standard random values (mean and standard deviation values), generate the values of sample random numbers for the random normal variable (x) or the random lognormal variable [$\ln(x)$], depending on the distribution of the statistical parameters.
- 5- Since all random variables are defined, Monte Carlo simulations can be used to calculate the whole limit state. Therefore, the probability of failure (serviceability or ultimate) which is the probability of exceeding the allowable limit state can be described as:

$$P_f = \frac{n[g(x) < 0]}{N[g(x)]} \quad (2.17)$$

where, $n[g(x)]$ is the number of simulations when the limit state is not satisfied, and N is the total number of simulations for the limit state.

It is important to simulate an efficient number of sets, such that the variation of the design parameters in a single simulation will not influence the solution of the entire process of simulations. Moreover, the accuracy of the method depends on the number of simulations.

Any test data obtained can be plotted on the normal probability paper to present the cumulative distribution functions (CDF), which allows the evaluation of statistical parameters and it was assumed as normal distribution function for load. The construction and use of the normal probability paper is described in the textbook (Nowak and Collins 2013). The horizontal axis represents a basic variable for which in this study, it was representing the compressive strength and the vertical axis representing the inverse normal probability scale, which represents the distance from the mean value in terms of standard deviation as shown in the Figure 2.3. This figure considers basic properties of the normal probability paper, which states that any straight line represents a normal distribution function, the mean value can be found directly from the graph, which is the intersection of the straight line presenting the normal CDF and the horizontal axis, and the standard deviation can be found directly from the graph.

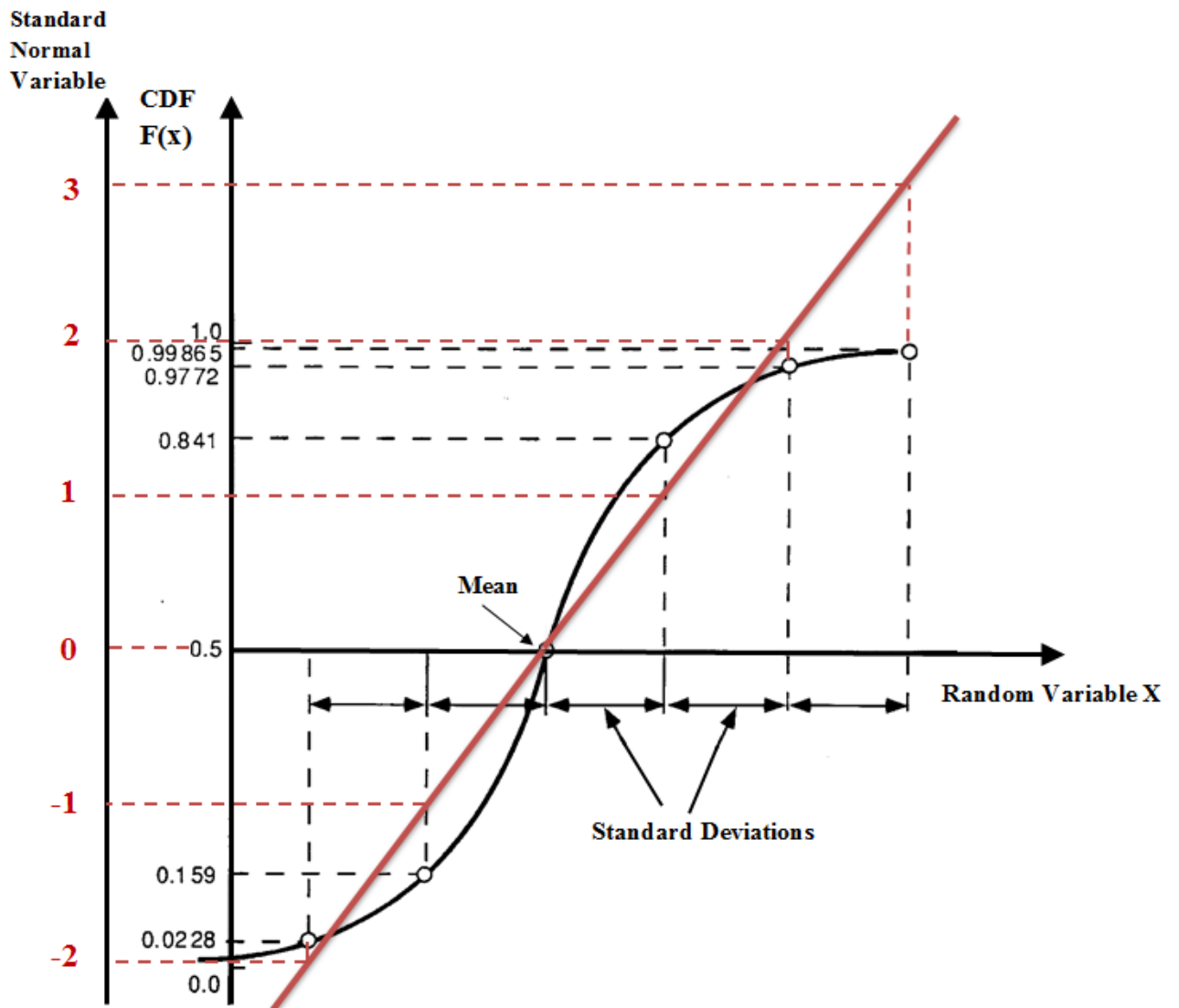


Figure 2.3: Normal Distribution Function on the Normal Probability Paper.

CHAPTER 3

CORROSION OF STRUCTURAL STEEL COMPONENTS

3.1 INTRODUCTION

In 2002 the Federal Highway Administration has been estimated the annual cost of corrosion damage in the United States is approximately \$276 billion. This demonstrated that bridge corrosion carries a high economic price tag.

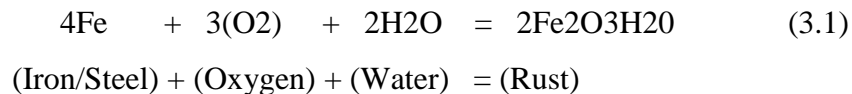
The high maintenance costs associated with corrosion-based deterioration in steel bridges in the U.S., as well as the historic collapses of steel bridges due to the corrosion of structural components (as determined through investigation), have drawn attention to the problem of bridge corrosion. As a result, the designers and bridge engineers recommended periodic inspection to the corroded bridges in order to maintain safety function of the steel bridge during assumed structures age.

In this chapter, the corrosion of composite steel girder will be investigated. The corrosion type shall be determined also, the evaluation for corrosion penetration in the steel. Corrosion causes material loss in the cross section of steel along with deferent heights through girder span. The easy way to calculate the material loss is to clean the corroded area on the steel part then, weigh it and compare between the clean part weight and corroded one. However, this is not the real situation for estimation the corrosion of steel girder. In fact, the corroded steel section (girder) is attached to other parts of the

bridge that prevent any removal. In addition, the location of corrosive damage is important since the max shear and bearing section will be at girder supports but the max moment and deflection section will be on the mid span of the steel girder.

3.2 CORROSION REACTION OF STEEL

Steel corrodes when it is exposed to water (humidity) and the reaction of Iron with Oxygen will be developed and a rust area then forms on the metal surface. The color of corroded area sometimes may differ from another one. This phenomenon happens because of different distance between the surface and moisture. The following chemical equation (3-1) shows one simple reaction to clearly corrosion:



The steel of composite steel girder has different ingredients depends on the types of attached metals. According to this the penetration of steel due corrosion will be varied from one to other. The most available steel is the carbon steel product. Weathering steel have been introduced as more resistance steel to corrosion. However, the cost of it is also more than carbon steel. To maintain lower cost the designers canceled the initial painting of steel girder.

The most famous classification of corrosion types has been done by Fontana in 1986 which depends on his experiments of monitoring different corrosion types. He classified the corrosion as uniform, crevice, pitting, galvanic, intergranular, selective leaching, erosion, and stress corrosion.

3.3 CORROSION OF COMPOSITE STEEL GIRDER

Corrosive damage to steel bridge girders is a time-dependent process that also depends on the environmental impact related to the location of the bridge. The source of corrosion is an electrochemical reaction that requires the availability of humidity (moisture) and oxygen simultaneously. Subsequently, corrosion normally accumulates and develops with the aging of steel components over several decades. On the other hand, research conducted by scientists has shown that the shape of corrosion has a specific likeness; this implies that there are definite types of corrosion that can be categorized depending on the visible shape of attack (Fontana and Green, 1967). Section loss due corrosion for a study conducted by the American Society for Testing and Materials (ASTM) in 1960 is shown in Table 3.1.

Table 3.1: Corrosion Rates for Carbon Steel at Various Locations (ASTM, 1960)

Location	Environment	Section Loss (μm) 1 Year	Section Loss (μm) 1 Year
Phoenix, AZ	rural	6.6	9.2
Vancouver,	rural-marine	17.3	26.7
Detroit, MI	industrial	23	28.9
Potter County,	rural	21.8	41.1
State College,	rural	25.1	45.9
Durham, N.H.	rural	35.4	54.7
Middletown,	semi-	36.2	57.6
Pittsburgh, PA	industrial	42.8	61.3
Bethlehem, PA	industrial	55.1	75.3
Newark, NJ	industrial	72.4	102
Bayonne, NJ	industrial	127	155
East Chicago,	industrial	111	169
Cape Kennedy, FL	marine	41.1	173
Brazos River,	industrial-	107	187
Cape Kennedy, FL 54m from	marine	61.3	263
Kure Beach, NC 240 m	marine	85.1	292
Cape Kennedy, FL 54m from	marine	70.8	330
Daytona Beach,	marine	209	592
Cape Kennedy, FL 54 m from	marine	191	884
Point Reyes,	marine	315	1004
Kure Beach, NC 24 m	marine	712	1070
Cape Kennedy, FL beach	marine	1057	

The loss of steel in girder cross section showed that the dimensions are decrease relatively symmetric. That is mean the corrosion type is uniform. For sure, the air content (Oxygen) plays an important role in developing this corrosive damage. The scientists refer to this type as atmospheric corrosion. The atmospheric environment has been classified to three types, which are dry corrosion, damp corrosion, and wet corrosion. The main deferment between these types is the water content (moisture). The dry one has been considered as insignificant comparing with the others two types. Figure 3.1 presents a typical location of uniform corrosion on a composite steel girder bridge.



Figure 3.1: Presents a Typical Location of Uniform Corrosion on a Composite Steel Girder Bridge. The Picture is Courtesy of The Nebraska Department Of Roads.

3.4 CORROSION PENETRATION

The estimation of corrosion penetration in composite steel girder is difficult process. The corrosion type is uniform but it also depends on environment location that related to the category of dry, damp, and wet. Another assumption we need to consider that the available data for corrosion depends on previous investigations which related to steel material not to the existence steel structure. Therefore, it is empirical procedure. From the literature review we can notice two methods have been to estimate the corrosion penetration of steel girders.

3.4.1 Albrecht and Naeemi (1984)

This study is the base of the other one. Their experiment includes different types of steels exposed to urban, rural, industrial, and marine environment. The corrosion rate varies highly comparing with the next method. The samples of steel that have been used were small. As a result, from the data of corrosion behavior they discover the corrosion loss as in equation (3.2). In addition, Figures 3.2 through 3.4 show the corrosion penetration with exposures time depending on this method data A and B parameters of carbon steel (which is the type of steel for investigated composite steel girder) in rural, urban and marine environment respectively by (Albrech and Naeemi, 1984).

$$C = A.t^B \quad (3.2)$$

Where:

C	Average corrosion penetration rate in micrometer
t	Number of years
A & B	Parameters determined from analysis of experimental data

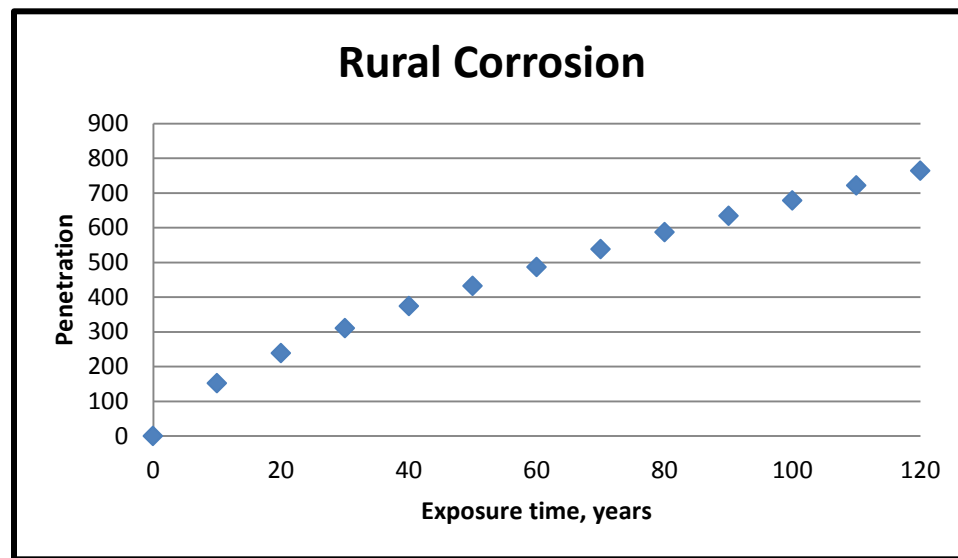


Figure 3.2: Rural Corrosion of Carbon Steel (Albrech and Naeemi, 1984)

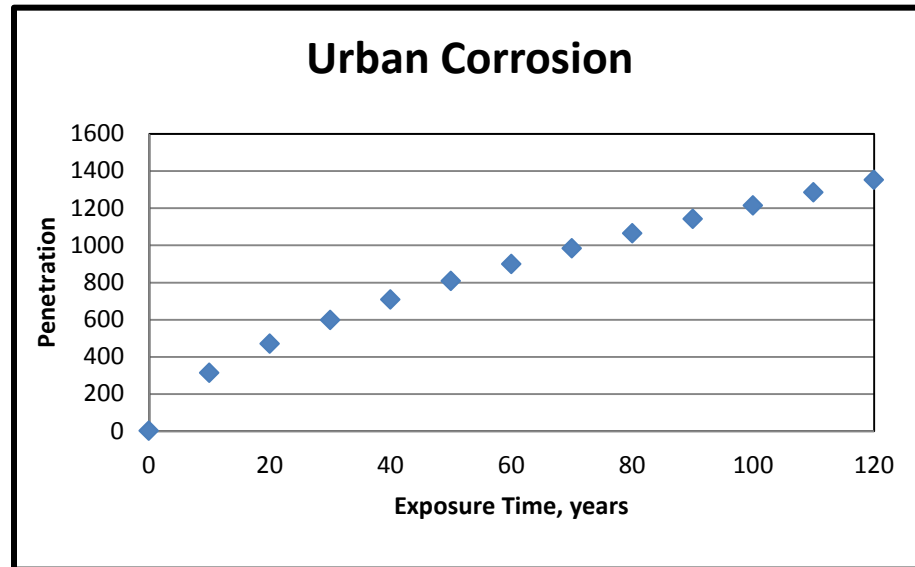


Figure 3.3: Urban Corrosion of Carbon Steel (Albrech and Naeemi, 1984)

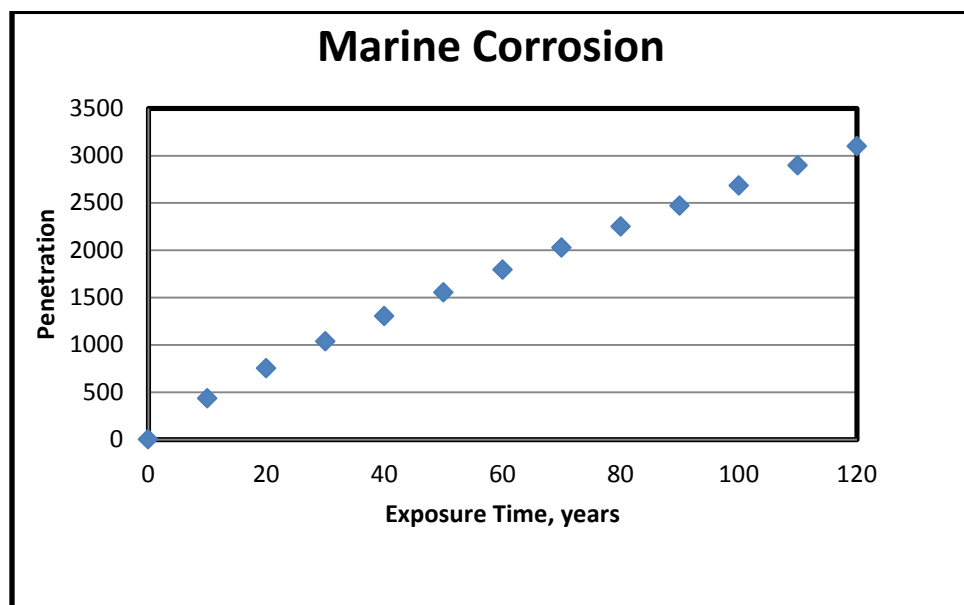


Figure 3.4: Marine Corrosion of Carbon Steel (Albrech and Naeemi ,1984)

3.4.2 Park (1999)

Depends on probability theory where there are uncertainties, which affect the data of corrosion penetration in the previous research of Albrech and Naeemi (1984). Park (1999) suggested three curves for low, medium, and high corrosion, which the penetration rate can be estimated using Figure (3.5). In addition, Table 2.2 shows the values of corrosion penetration rate.

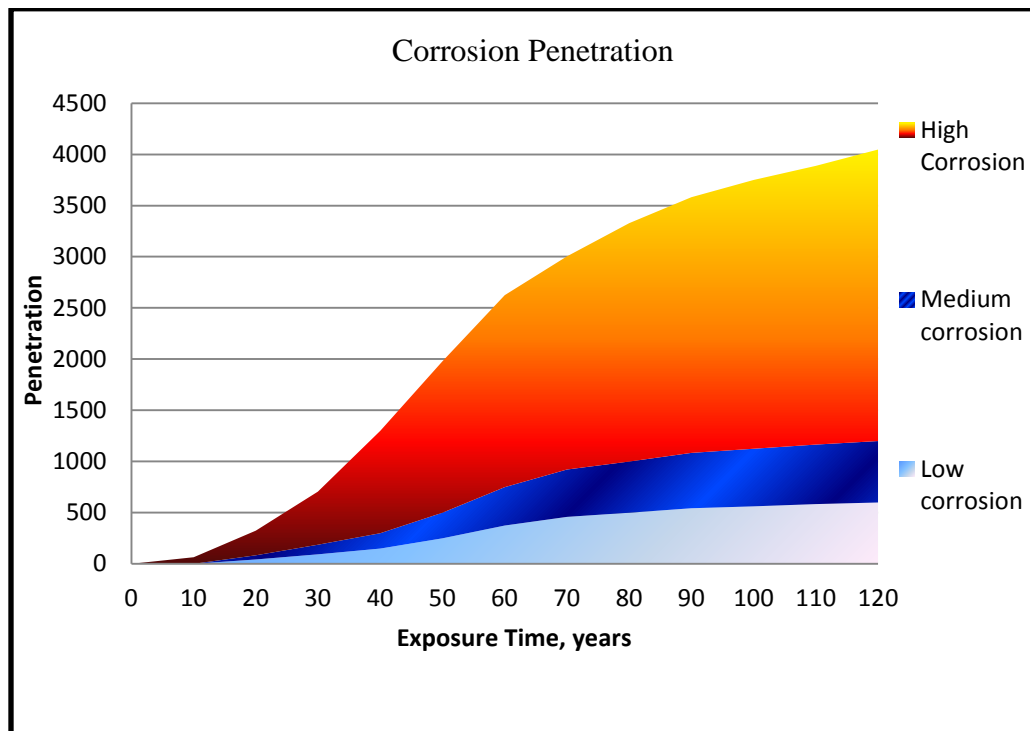


Figure 3.5: Corrosion Curves as High, Medium, and Low (Park , 1999)

Table 3.2: Values of Corrosion Penetration Rates, (Park , 1999)

# of Year	Low μm = .000039 in	Medium μm = .000039 in	High μm = .000039 in
0	0	0	0
10	0	30	63
20	42	85	240
30	93	167	520
40	150	370	1000
50	250	630	1480
60	375	784	1875
70	460	958	2083
80	500	1042	2330
90	542	1083	2500
100	563	1125	2625
110	583	1167	2725
120	600	1209	2850

CHAPTER 4

LOAD AND RESISTANCE MODELS

4.1 INTRODUCTION

The bridge design depends on the prime load combination of dead load, live load, environmental load and other specific loads. The dead load components contain the deck (slab) weight, wearing service weight and barriers weight. Live load is divided into two components, static and dynamic. The moving truck represents the live load value such as HS20-44 from AASHTO-LRFD design code. In addition, the dynamic impact (IM) is added to live load as design requirements. The environmental loads included temperature, wind and earthquake. The last ones are the specific loads, which include collision and emergency braking. The development of load models using the available statistical data were demonstrated by Nowak (1995-2013). Nowak used the reliability theory to develop the design of bridges. The load components have been treated as random variables. Different components of load and resistance have a relation that has been modeled as probabilistic data.

In this study, the major loads of the considered bridge were modeled, and the load combination represents the highway bridge loads simultaneously. Practically these loads are dead load, live load and dynamic load. All other load components will not be considered in this study, as it requires a special area of research.

4.2 DESIGN FORMULA

The design formula in AASHTO LRFD code 2012 is shown in the following equation:

$$1.25 DL + 1.5 DW + 1.75 LL (1 + IM) < \phi R \quad (4.1)$$

Where, DL, DW, and LL are nominal values of the load components, $IM = 0.33$,

ϕ is the resistance factor, and R is the nominal value of resistance, which is either the moment strength or shear strength.

The deterministic values (1.25, 1.5 and 1.75) are load factors of LRFD design code due to uncertainties. The design values of load and resistance have to be conservative to provide an adequate safety level. Loads are usually overestimated, as shown in Figures 4.1 and 4.2 (Adapted from Nowak and Collins 2013).

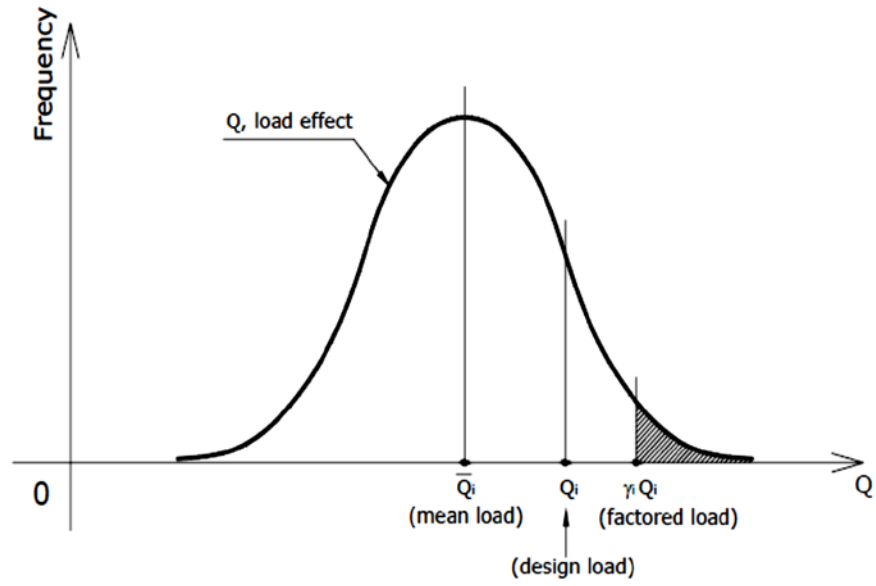


Figure 4.1: Mean Load, Design (Nominal) Load and Factored Load.

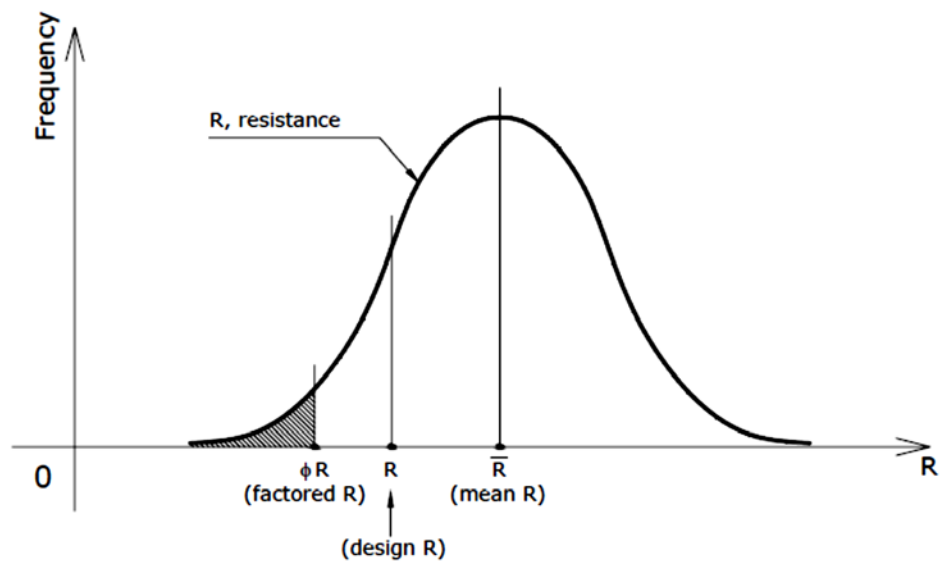


Figure 4.2: Mean Resistance, Design (Nominal) Resistance and Factored Resistance.

4.3 LOAD MODELS

The statistical parameters of the total load, Q , are determined as a function in terms of statistical parameters for load components. The mean of Q , is denoted by μ_Q , which is determined by the sum of the mean values of load components as shown in the following equation:

$$\mu_Q = \mu_{DL} + \mu_{DW} + \mu_{LL} + \mu_{IM} \quad (4.2)$$

where, μ_{DL} is the mean dead load; μ_{DW} is the mean dead load for wearing surface; μ_{LL} is the mean live load, and μ_{IM} is the mean dynamic load. The mean values of load components are calculated based on bias factors, λ , and the nominal (design) value of the considered load component.

The variance of Q , is denoted as σ^2_Q , which is the summation of variances of load components as shown in the following equation:

$$\sigma^2_Q = \sigma^2_{DL} + \sigma^2_{DW} + \sigma^2_{LL} + \sigma^2_{IM} \quad (4.3)$$

Then, the standard deviation of Q , is denoted by σ_Q , which is equal to the square root of variance. However, the coefficient of variation of Q , is denoted as V_Q , which is evaluated using the following formula as:

$$V_Q = \sigma_Q / \mu_Q \quad (4.4)$$

The total load is determined by the sum of the components such as dead load, live load and dynamic load. For these load components only summary statistics were available. However, it was assumed that the load effect is normally distributed (Q), which is treated as a random variable.

4.3.1 DEAD LOAD

The dead load for the structural and nonstructural elements that are permanently connected to the bridges can be defined as the gravity load due to the self-weight. The statistical parameters of the dead load are summarized into Table 4.1. Based on the bridge class for the abbreviation of representing the type of the dead load this table uses DC1 as the factory made element, the element, which is known as cast-in-place concrete is shown by DC2. Finally, the DW represents the dead load of the wearing surface. The dead load model is shown in Figure 4.3.

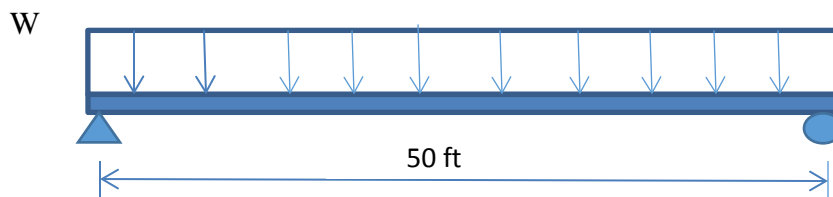


Figure 4.3: Dead Load Model used in this Study

Table 4.1: Bias Factor and Coefficient of Variation of Dead Loads

Component Type	Bias Factor (λ)	Coefficient of Variation (V)
DC1	1.03	0.08
DC2	1.05	0.10
DW	1.00	0.25

4.3.2 LIVE LOAD

The design of live load governs a range of forces presented by vehicles moving on the bridge. Many parameters effect the live load on the bridge, such as, the span length, truck weight, axle loads, axle configuration, position of the vehicle on the bridge, number of vehicles on the bridge, girder spacing, and stiffness of structural members, which include the slab and girders. The load itself, in addition to the distribution of this load, characterizes live load on bridges. Therefore, the most important item to be considered is the load spectrum per girder (Nowak and Collins, 2013).

However, it would be possible to figure out the statistical data for any particular lifetime based on the load data available. In fact, this study focused on moment effects on one simply supported span bridge, so that the statistical data for positive moment would be sufficient. The statistical parameters are adapted from the lectures of Nowak.

As it can be shown, the live load effect mostly depends on the length of the bridge, which is not surprising, regarding to the structural analysis, the moment and shear depend on the length of the bridge. The load model is shown in Figure 4.4.

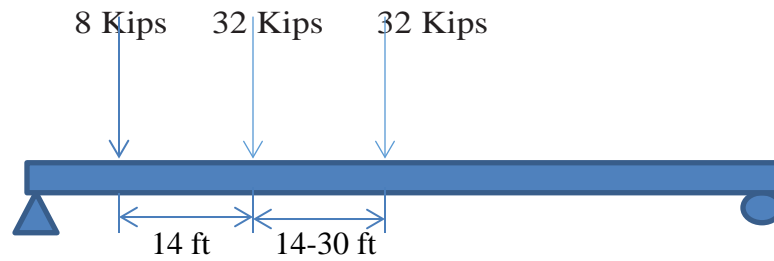


Figure 4.4: Live Load Model (Moving Truck HS20-44) used in this Study (Span=50 ft)

4.3.3 DYNAMIC LOAD (IM)

The dynamic load represents the impact of moving vehicle (truck) on the road service. This load will be considered according to AASHTO LRFD code (2012), when designing a bridge. Dynamic load can be defined as the ratio of dynamic deflection and static deflection. Actual dynamic load depends on three cases:

- 1- Road roughness.
- 2- Bridge dynamics.
- 3- Vehicle dynamics.

Moreover, these three cases can be considered as additional static loads added to live load. The specified value of dynamic load in AASHTO-LRFD is equal to 0.33 of the design truck effect, and is considered zero for the uniformly distributed load. Table 4.2 presents the dynamic load allowance and Table 4.3 presents the statistical parameters for live load.

Table 4.2: Dynamic Load Allowance (IM)

Component	<i>IM</i>
Deck Joints—All Limit States	75%
All Other Components	
♦ Fatigue and Fracture Limit State	15%
♦ All Other Limit States	33%

Table 4.3: Bias Factor and Coefficient of Variation for Live loads

Component Type	Bias Factor (λ)	Coefficient of Variation (V)
Single Lane loaded	1.3-1.2	0.11
Two lane loaded	1.2-1.0	0.11
MI	Mean= 0.1	0.8

4.4 RESISTANCE MODELS

The symbol R denotes the resistance of a structural component, which is a function of strength behavior formed from structure components and connections. The resistance limit as defined as the load-carrying capacity depends on the structure properties such as the material strength, the section geometry, and the dimensions. The resistance, R is treated as a random variable because of various categories of uncertainties. Simply, R is considered as a product of three factors as shown in the following equations :

$$R = R_n \cdot M \cdot F \cdot P \quad (4.6)$$

$$\mu R = R_n \cdot \mu M \cdot \mu F \cdot \mu P \quad (4.7)$$

$$V_R = \text{SQRT} (V_M^2 + V_F^2 + V_P^2) \quad (4.8)$$

$$V = \sigma / \mu \quad (4.9)$$

$$\mu = \lambda \cdot R_n \quad (4.10)$$

Where: R_n is the nominal (design) value of resistance, μR is mean value of R , and V_R is the coefficient of variation of R .

M is the materials factor representing material properties, in particular strength and modulus of elasticity, μM is the mean value of M, and V_M is the coefficient of variation of M.

F is the fabrication factor representing dimensions and geometry of the component, including the cross-section area, moment of inertia, and the section of modulus, μF is the mean value of R, and V_F is the coefficient of variation of F.

P is the professional factor representing the approximations involved in the structural analysis and idealized stress/strain distribution models, μR = mean value of P, and V_P is the coefficient of variation of P.

Finally, λ is bias factor, and σ is the standard deviation.

In this study, the design resistance, R_n (nominal resistance), is the value of resistance specified by the code . For a compact steel beam in plastic analysis, the limit of bending resistance is represented by the following formula:

$$R_n = F_y \cdot Z \quad (4.11)$$

Where: F_y is the yield stress of steel, (for W 27x94 girder =36 ksi) and Z is the plastic section modulus, (for the above girder = 276 in³).

Table 4.4 shows the statistical parameters of the material (M), fabrication (F), and professional (P) factors in addition to the resistance (R).

Table 4.4: Bias Factor and Coefficient of Variation of Factors M, F, and P, and

Resistance (R)

Type of Structure	Material and Fabrication factors, F & M (λ)	Material and Fabrication factors, F & M (V)	Professional, P (λ)	Professional, P (V)	Resistance, R (λ)	Resistance, R (V)
Composite Steel Girder						
Moment	1.07	0.08	1.05	0.06	1.12	0.10
Shear	1.12	0.08	1.02	0.07	1.14	0.105

CHAPTER 5

GIRDER CORROSION'S MODEL

5.1 INTRODUCTION

The steel girder bridges inadequacies require more investigation to improve the design and the material selection. One of the important disadvantages in steel bridges is time dependent corrosion. Thus, the structure component encounters increasing traffic and capacity loss. As corrosion damage will degrade the cross section of composite steel girder. The impact of corrosion on the internal forces (shear and moment) depends on the location of cross section, which means corrosion will be measured separately according to difference in damage between the two locations, the support and mid span. However, when corrosion has the same volume, it will consider one cross section for analyzing.

In this study, the corrosion deterioration mostly appeared at girder supports. However, the mid span cross section will be less affected. It seems that the bridge has a deck leak at the cross section where the abutment is contacting the bridge. In this chapter, linear analysis has been applied to the girder cross section to evaluate the reduction capacity in the moment (bending), the shear, and the bearing.

5.2 GIRDER DETERIORATION MODEL

The corrosion model should not be the same in every place, the situation of one bridge site is different from another so, the corrosion pattern will be different also. The typical corrosion model of steel girder has been introduced by Kayser in 1988. Figures 5.1 and 5.2 represent the corrosion model of them at the support (shear section) and mid span (moment section) respectively.

The corrosion penetration of steel has been estimated then, its effect on the cross section of steel girder at support and mid span has been calculated. Moreover, the load carrying capacity for bending moment and shear in girder is calculated by assuming that the two models have been represented a real application relatively.

In this study, the models have been used to calculating the effected properties of corroded steel girder adopted from (Kayser, 1988). On the other hand, (Park 1999) developed the models mathematically by adding deferent dimensions criteria to calculate the affected area. Then, the models have been divided into three types of model depending on the corrosion pattern.

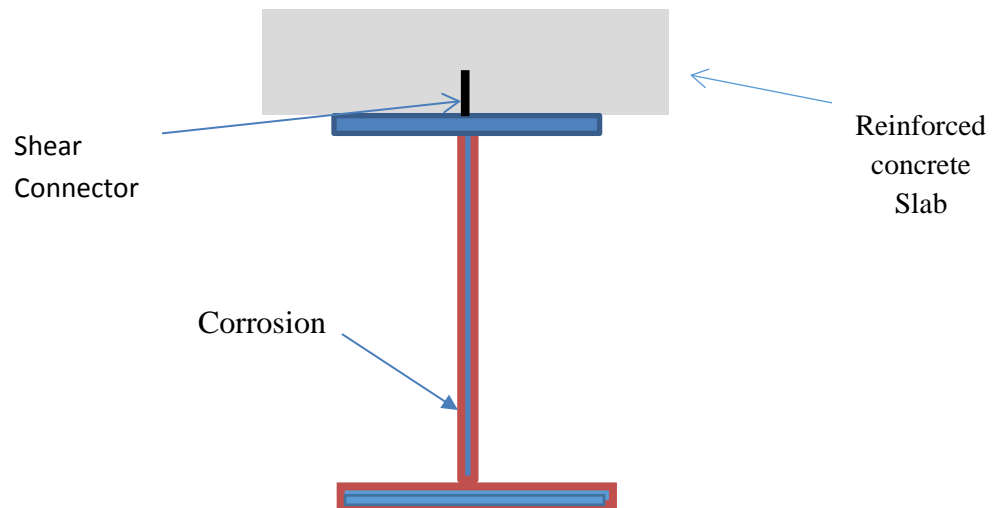


Figure 5.1: Composite Steel Girder Corrosion Model of Cross Section at Supports.

(Shear Section Model)

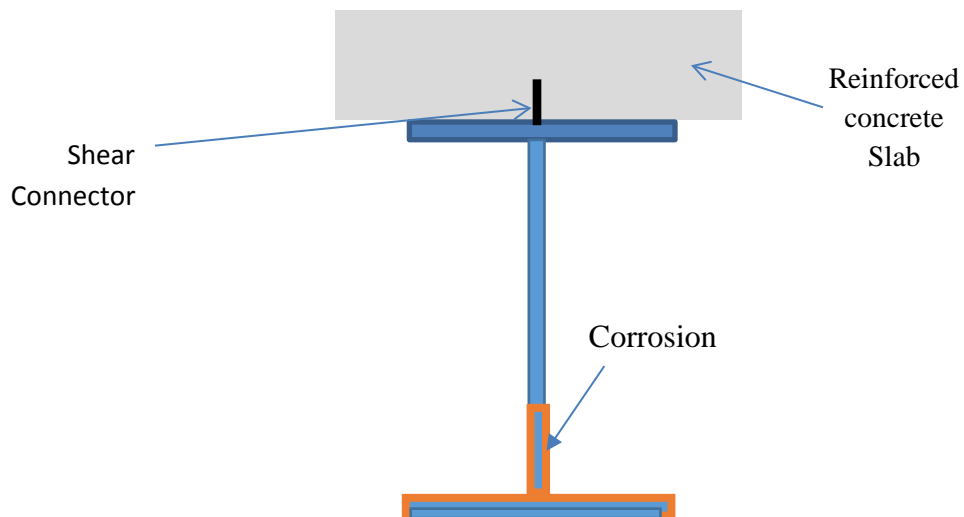


Figure 5.2: Composite Steel Girder Corrosion Model of Cross Section at

Mid Span. (Moment Section Model)

5.3 BRIDGE SPECIFICATIONS

The data of bridge has been received from Nebraska Department of Roads (NDOR) as in the following:

- Simple Span Bridge with 50 ft builds in 1953.
- 5 girders of W 27x94 A36 steel.
- Composite deck, 6.5 in thickness and $f_c=4000$ psi.
- 1.5 in Hunch above girder.
- 2 in wearing surface.

5.4 REDUCED CROSS SECTION

Due to corrosion penetration in the steel girder bridge the dimensions of cross section will decrease following the loss of material at specific location. This will reduce the area of cross section as a function of dimensions. In addition, moment of inertia will be reduced as it a function of area. Girder load carrying capacity will be decreased depends on the type of corrosion (low, Medium, and High). The corrosion level as low, medium, and high has a distinguished differently accords to the Figures 5.3 and 5.4, which are shown the reduction in cross section with deferent corrosion level. Also, a big difference in area reduction between supports cross section (shear section) and mid span cross section (moment section). This is according to the difference of the considered corrosion models as clarified in Figures 5.1 and 5.2.

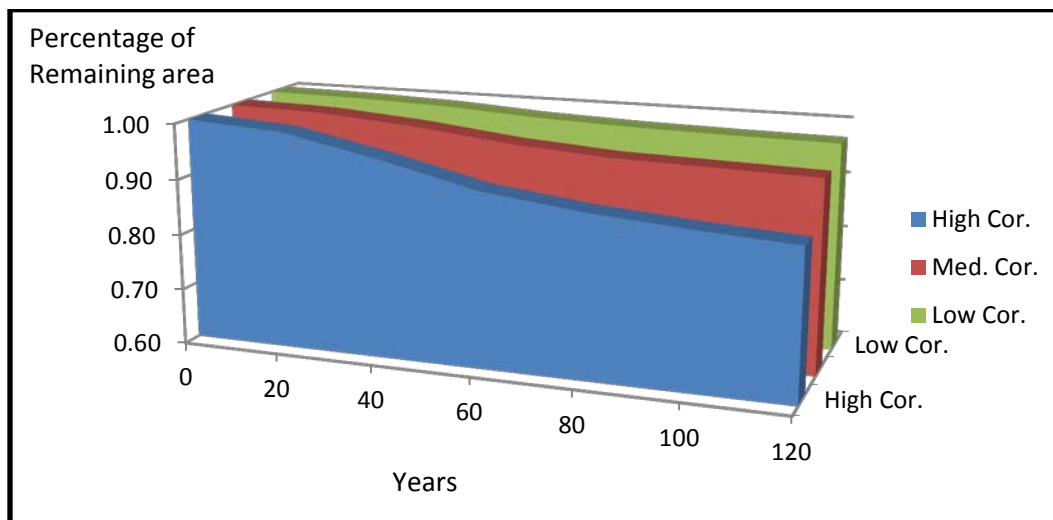


Figure 5.3: Cross Section Reduction at Mid Span (Moment Section).

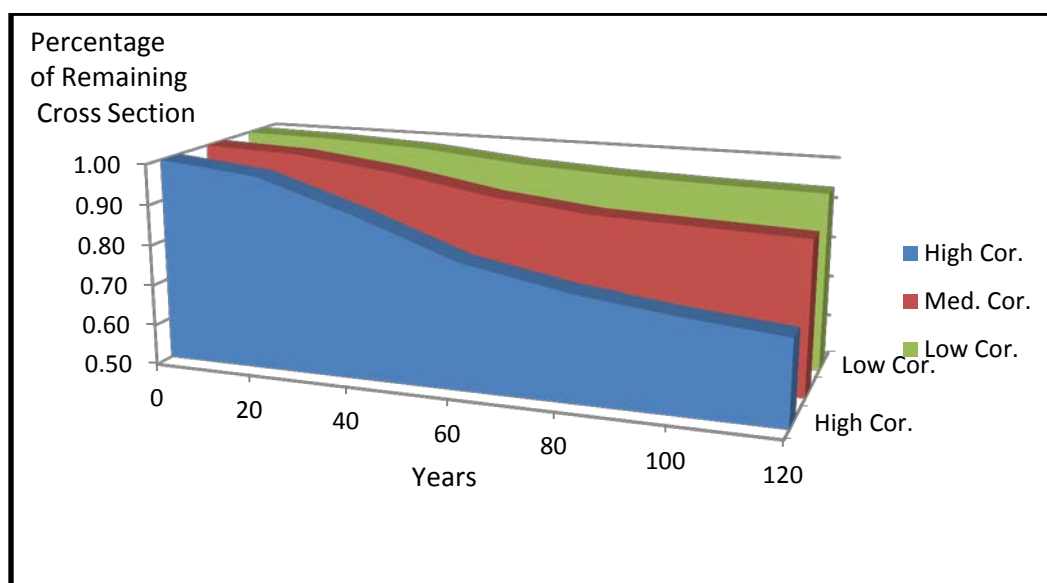


Figure 5.4: Cross Section Reduction at Girder Supports (Shear Section).

5.5 SHEAR CAPACITY

The resistant to shear load capacity may be effected by corrosion, as the resistance to shear in girder primary performed by the web part. The design of web is according to the theory of elastic non-buckling stress. Therefore, the thinner steel girder web is not preferable to avoid the slenderness. The application to examine the slender web is highly recommended. To check the web panel at supports (shear critical section), the plate buckling theory should be investigated. The section of the web should be modeled according to that theory. Figures 5.5 through 5.7 have shown the percentage of the remaining shear capacity with deferent levels of corrosion.

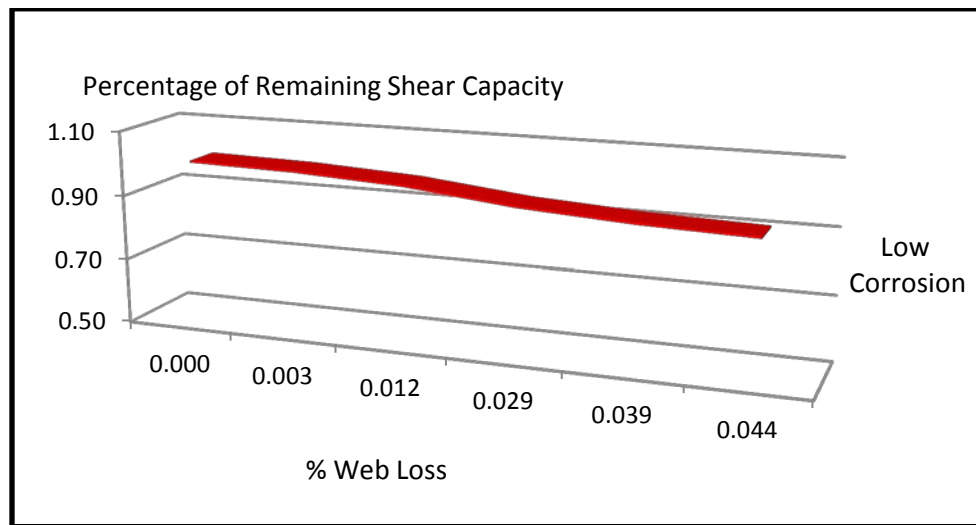


Figure 5.5: Reduction in Shear Capacity with Web Loss for Low Corrosion

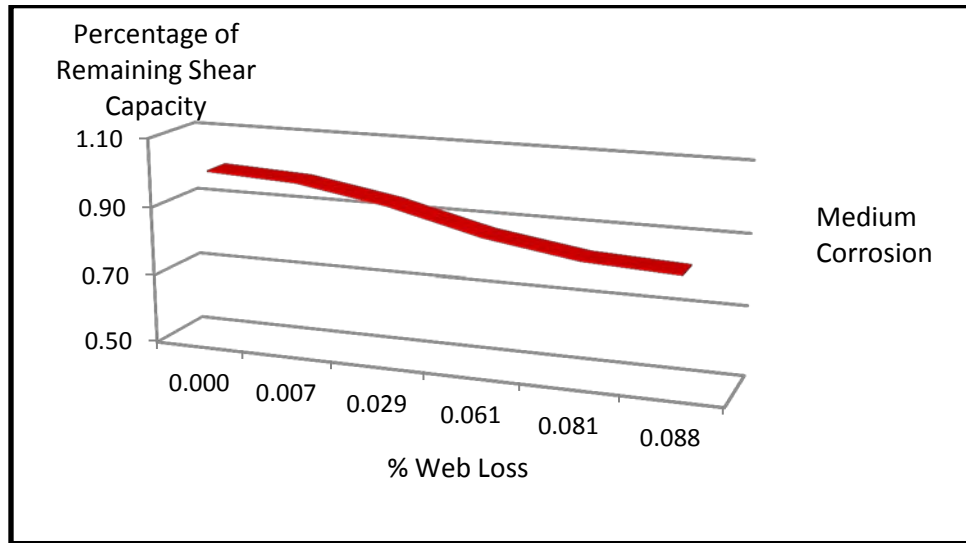


Figure 5.6: Reduction in Shear Capacity with Web Loss for Medium Corrosion

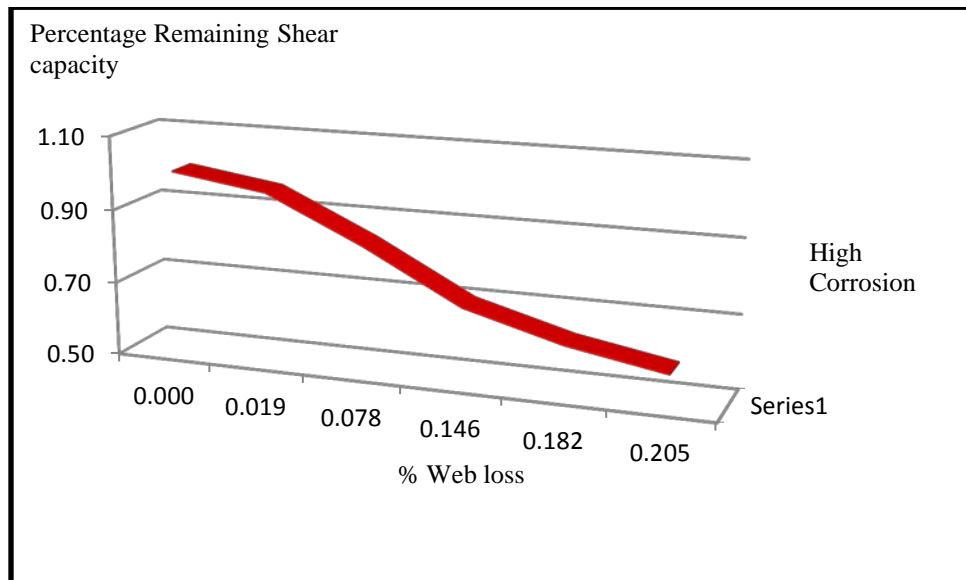


Figure 5.7: Reduction in Shear Capacity with Web Loss for High Corrosion

5.6 BEARING CAPACITY

The steel girder resistance to loading forces may decrease due to corroded web section, which directly above the supports. The corrosion has an impact of bearing since the web section will be reduced and there is no presence of stiffeners to support the web to resist the bearing reaction. The installation of stiffeners is needed if the nominal shear load is more than 0.75 of the shear capacity design (AASHTO-LRFD). Therefore, for most cases when installing a new girder, it has enough carrying capacity to resist shear forces. However, due to deterioration caused by corrosion, the stiffener option may be mandatory to be installed. The evaluation of bearing capacity in this study has been done by plate theory calculation as the web section at supports resist the stresses of bearing. This section has been modeled according to the plate theory assumptions. The length of web bearing assumed to be as the width of flange that is approximately dominated for most design cases. Some researches add the flange thickness and the web fillet also. In this study, the bridge under investigation has been designed without bearing stiffeners. The following Figures 5.8 through 5.10 show the bearing capacity reduction versus the web section loss.

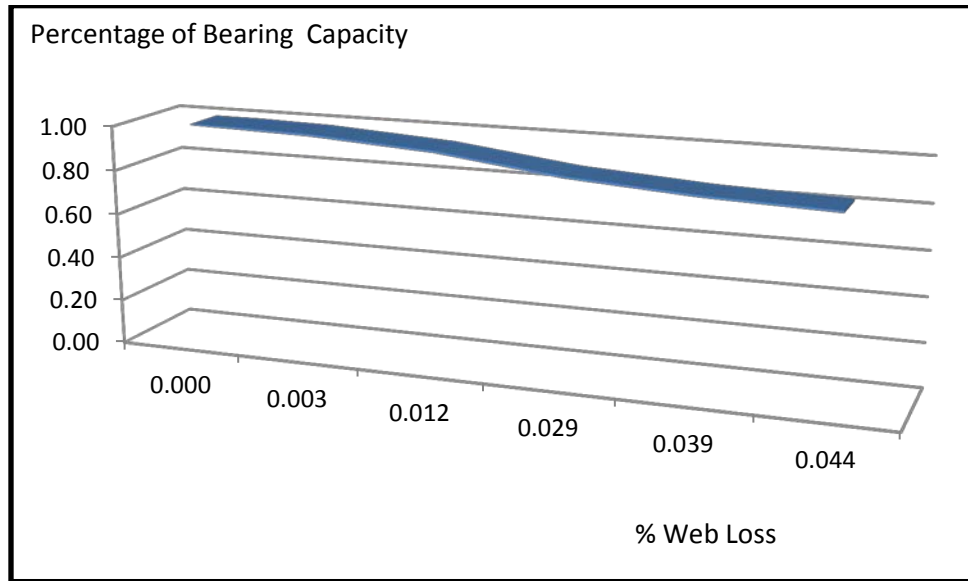
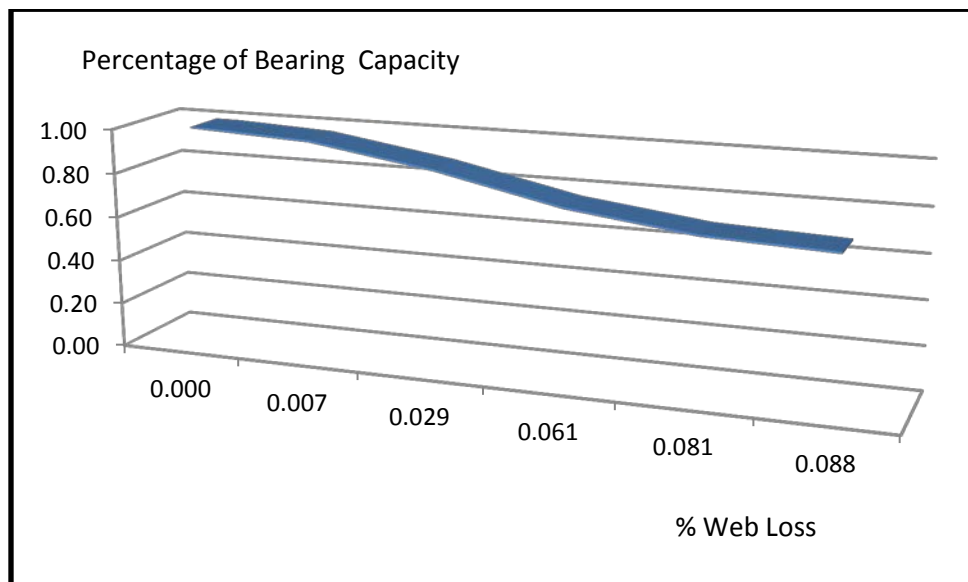
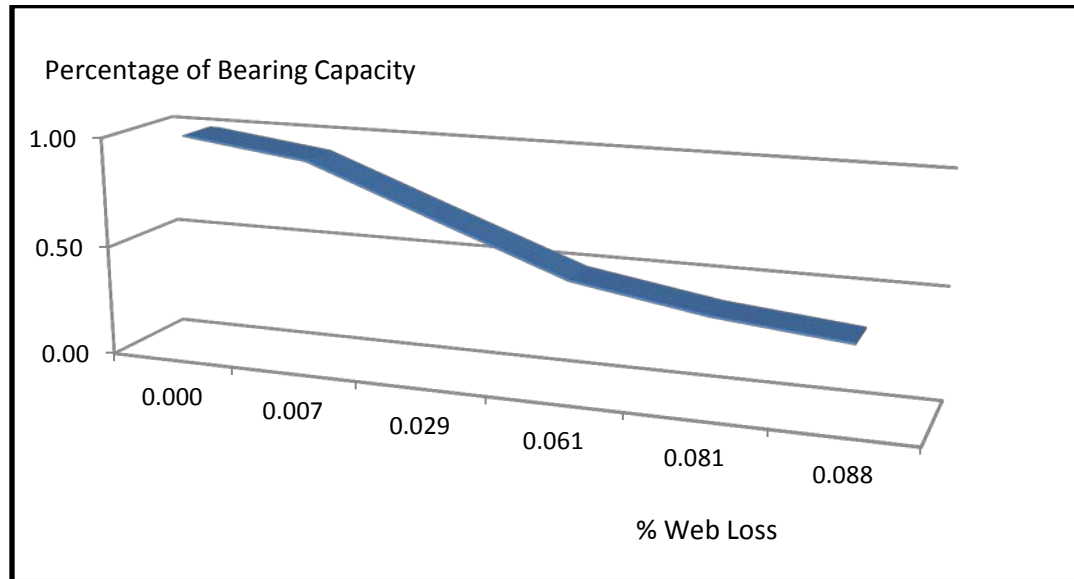


Figure 5.8: Reduction of Bearing Capacity with Web Loss for Low Corrosion



Figures 5.9: Reduction of Bearing Capacity with Web Loss for Medium Corrosion



Figures 5.10 Reduction of Bearing Capacity with Web Loss for High Corrosion

5.7 MOMENT CAPACITY AND BENDING STIFFNESS

The design limit state of simply supported composite steel bridge girder in bending assumes three criteria of failure as in the following:

- 1- The stress exceeds yielding of steel failure.
- 2- Damage of concrete.
- 3- Getting out of slab for shear connectors.

According to the standard bridge design of composite steel girder, the steel is approaching the yielding limit before the concrete starts to smash, which is caused by the failure of concrete that takes place at average strain scale of 0.00325 also, the ductile property in failure of steel has to consider. On other hand, getting out of slab for shear connectors is rare (no significant history).

Therefore, the type of failure will be steel yielding when reaching the ultimate capacity. The bending stress has been evaluated for the steel girder of composite section. The steel girder analytical model has been constructed with slab as a composite section behavior. The slab section needs to be converted as a steel component and required to find the effective width of the slab segment over the girder. The AASHTO-LRFD code provides three formulas for effective width of slab segment and the lower value will dominate. In addition, from the structure point of view, the assumption of plane section before bending remains plane after bending. The bending capacity has been calculated versus flange loss and the bending stiffness (EI) has been estimated in this study. The Figures 5.11 through 5.16 show the reduction in bending behavior.

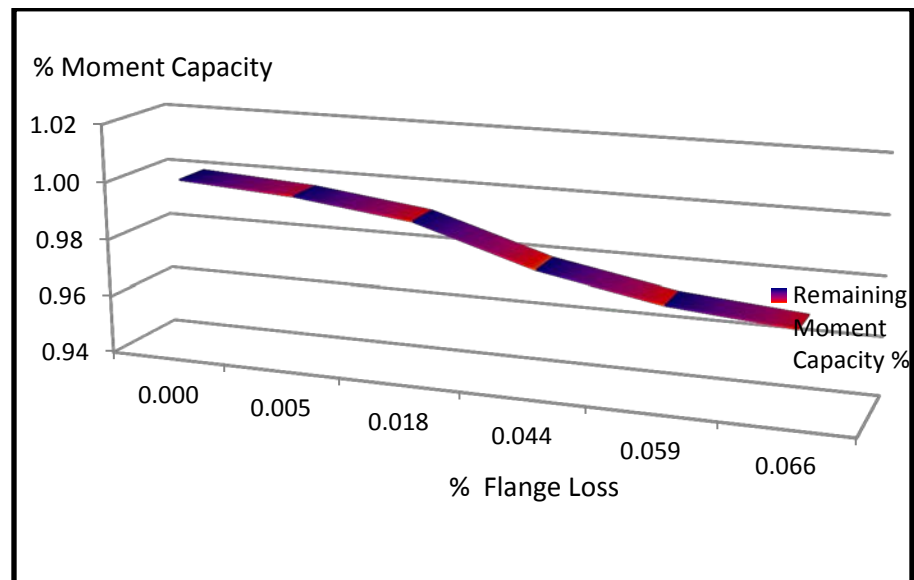


Figure 5.11: Reduction in Moment Capacity with Flange Loss for Low Corrosion

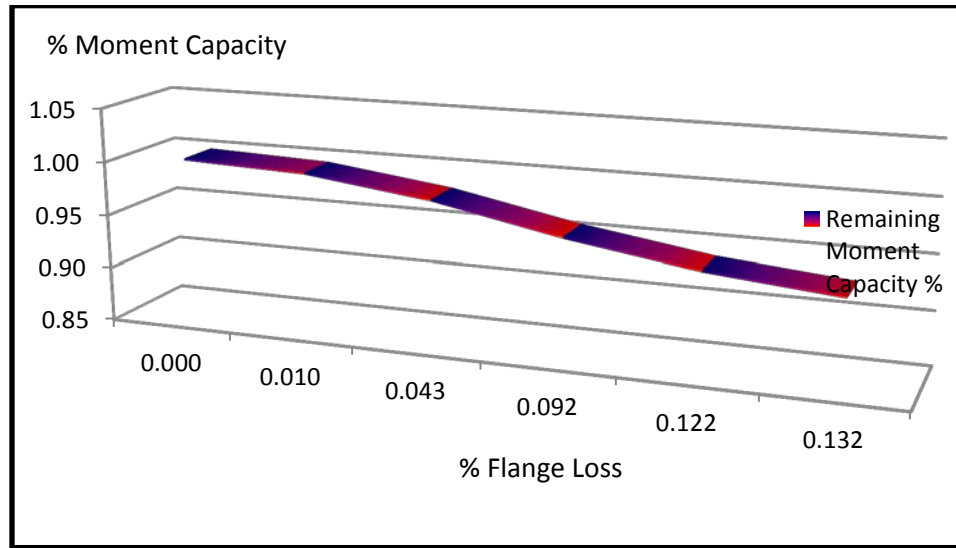


Figure 5.12: Reduction in Moment Capacity with Flange Loss for Medium Corrosion

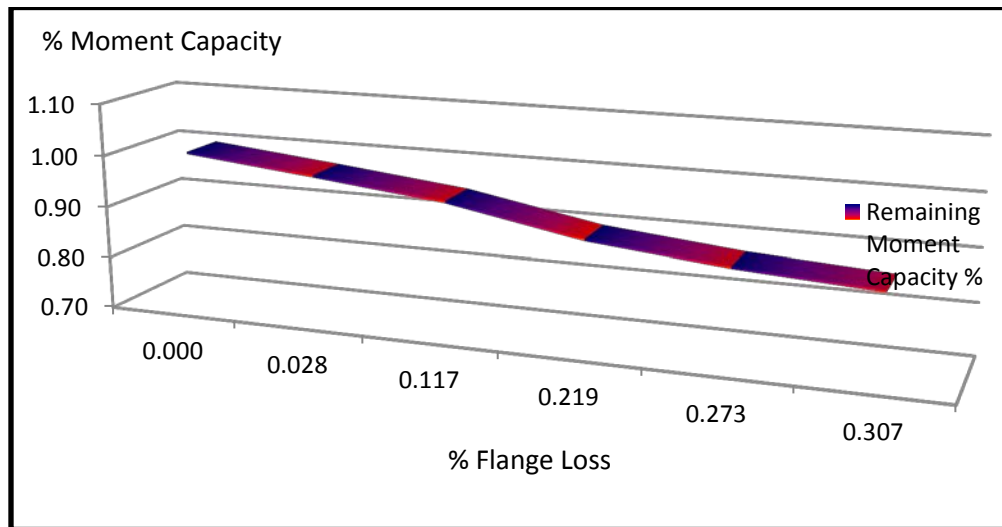


Figure 5.13: Reduction in Moment Capacity with Flange Loss for High Corrosion

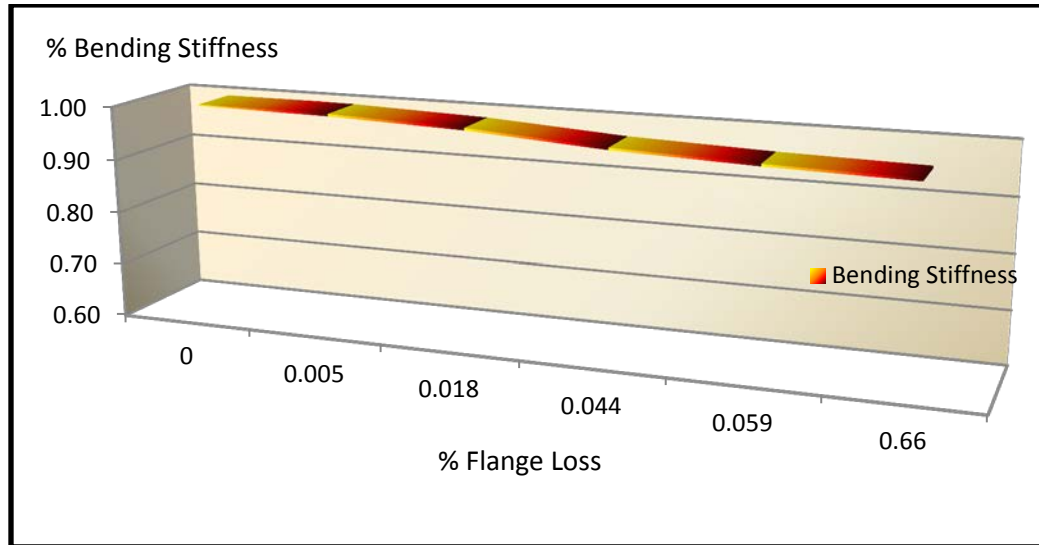


Figure 5.14: Reduction in Bending Stiffness Versus Flange Loss for Low Corrosion

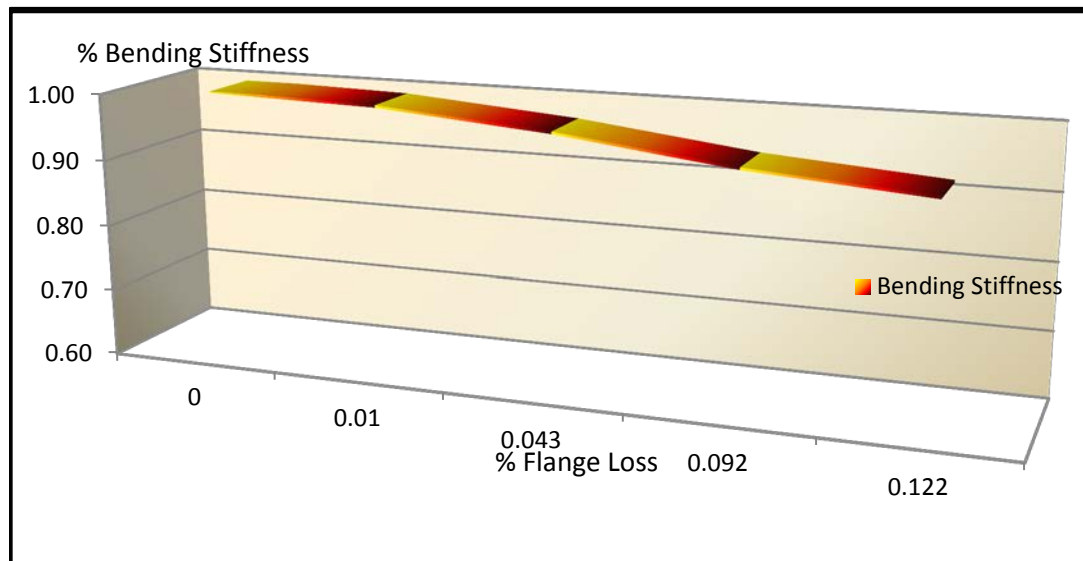


Figure 5.15: Reduction in Bending Stiffness Versus Flange Loss for Medium Corrosion

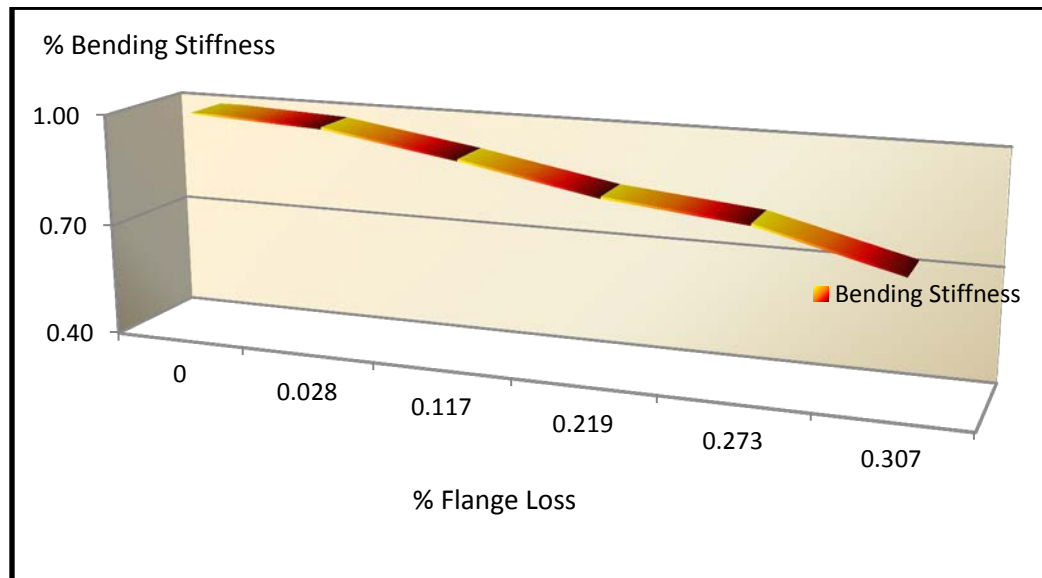


Figure 5.16: Reduction in Bending Stiffness Versus Flange Loss for High Corrosion

5.8 EVALUATION OF THE BRIDGE

The HS rating factor of AASHTO-LRFD code has been applied to evaluate the bridge for low, medium and high corrosion using HS20-44 truck as live load vehicle. The bridge is a simple span of 50 ft. The girder type is A36 steel with cross section of W27x94 with 5 girders in the bridge cross section. The concrete slab thickness is 6.5 inches. The capacity was determined based on the pervious composite steel girder corrosion model that was used for moment and shear behavior calculations (for composite section). Figures 5.17 and 5.18 represent the obtained results.

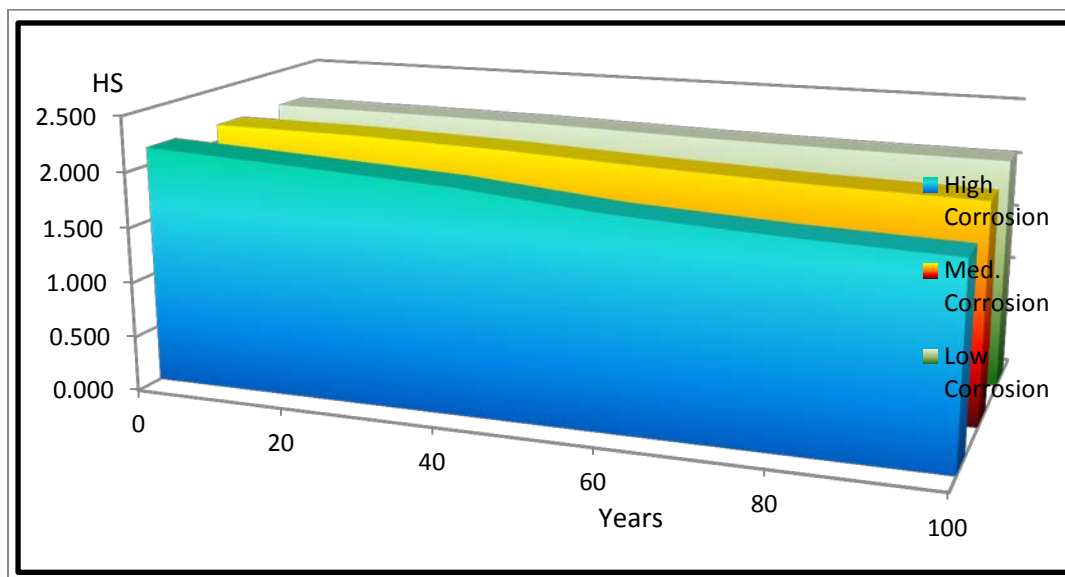


Figure 5.17: HS Rating Factor Versus Years of Exposure of Moment.

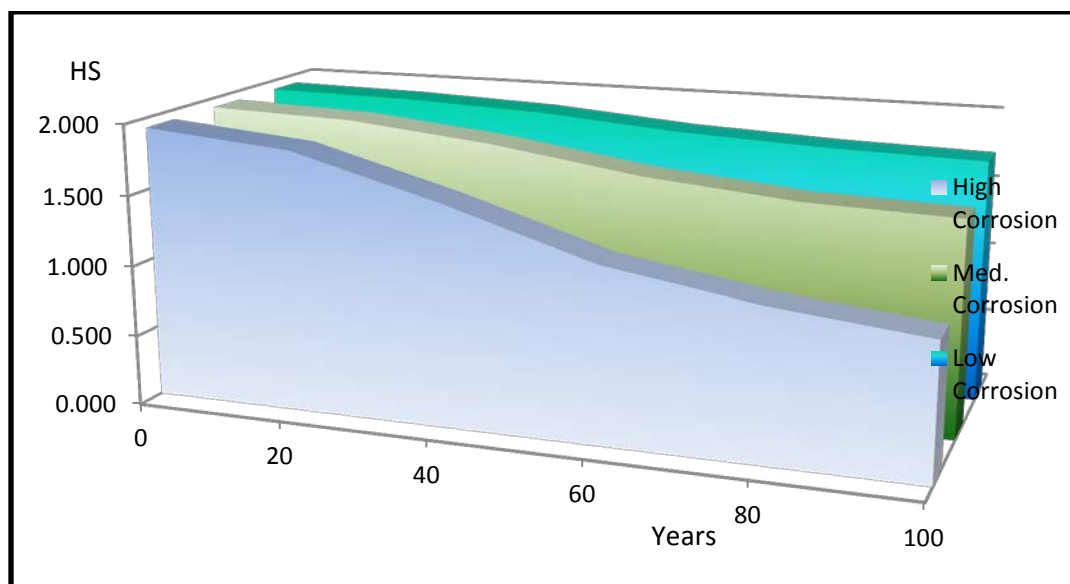


Figure 5.18: HS Rating Factor Versus Years of Exposure of Shear.

CHAPTER 6

ABAQUS STRUCTURE MODELING

6.1 INTRODUCTION

The modeling for a function of an existing structure, assumed to be close to the real responses as in the available structural situation as possible as it can be done. The objective of analysis using ABAQUS finite elements programing is to examine the assumed behavior of analyzed bridge structure or component due to its applied loads. Consequently, the stresses and deformations of structures under various load effects will be determined. In addition, it is an important procedure to figure material properties as they appear in the target structure. As a result, the cross section modeling will be highly influenced by material modeling. Elastic material returns to its original shape when releasing the loads. Otherwise, it can be called as an inelastic material.

For an elastic behavior, the stresses usually fellows the state of deformation while, in an inelastic behavior, residual deformation and stresses remain in the structure or element even when all external forces are removed. The elastic material may show linear or nonlinear behavior depending on the allowable capacity of this material or on a composite section material.

6.2 MODELING SYSTEM

The design of the model should reflect closely the structure behavior and approaching exactly in addressing a problem. The specifications of a model has represented by many attributes such as the targeted structure type, the form of loads and the resistance under investigation, the analysis pattern (design or an existence structure), and the expecting results that will be determined from modeling, which refers to the accuracy needed from this task and a user friendly model.

In this study, the corrosion rate of composite steel girder has been estimated depending on the methodology explained in chapter 3 (corrosion penetration). The corroded composite steel girder bridge has been represented by a girder model as structure component that exactly holds the corrosion problem. The complexity of the model (including all the bridge) has been avoided to obtain the stresses regarding the composite steel girder specifically. Furthermore, the model depends on the design of composite steel girder procedure as an independent element according to AASHTO-LRFD. On the other hand, the analysis of the entire bridge required more available information such as conducting a real test by passing truck and measuring the deflection of the bridge.

6.3 ENGINEERING SHAPE (GEOMETRY)

The model has been determined from the above section (Modeling System). This is a composite steel girder. The girder dimensions represented by the cross section with the span length will be considered as a model. In addition, the time dependent corrosion penetration will be modeled as 0, 20, 40, 60, 80, and 100 years. That means 6 models were needed to include the effect of these years.

6.4 MATERIAL PROPERTIES

A composite section of steel girder plus concrete section from slab with shear connectors between them has become the model of this study. Changing the concrete section to steel is required as composite steel girder design. The steel is A36 type and the concrete resistance is 4000 psi and this will give as $n = 8$. Otherwise, the concrete will be modeled as a dead load effect.

6.5 BOUNDARY CONDITIONS

This is an important factor to integrate the structure model. The structure analysis is an accuracy key that depends on its boundary conditions at supports. The real boundary conditions represented by the nature of girder supports (piers and abutments). These supports are represented as fixed, rollers, and pins. In this study, the girder was supported by two abutments.

6.6 LOADS

To evaluate the structure, the load effect should be considered. The bridge contains various types of load such as traffic, weight of components, wind, thermal expansion and others. In this study, the traffic load has been represented by HS20-44 AASHTO- LRFD truck with other dead loads. The girder loads have been determined depending on structure analysis according to the location of truck wheels.

6.7 ABAQUS COMPOSITE STEEL GIRDER MODEL

In this study, the composite steel girder has been modeled. To reflect the real condition of corrosion attack as it will cause a material loss to the girder itself. Furthermore, the response will be not complicated comparing with modeling of the whole bridge. In addition, this is an existing structure, which has been built and designed since 1953 which was no high level software used for design aid. In other words, the design was based on hand calculation, which means the bridge was designed by its components. In addition, the bridge is a simple span with relatively short length. On the other hand, the real response of the entire deck slab in reality may differ due to age as it is a composite material. Finally, previous research showed that the results of modeling the composite steel girder is close to real conduct when using non-destructive equipment. The girder has been modeled using 432 finite elements and 833 nodes. The span is 50 ft and the girder

type is W27x94. The slab effective width is 70.5 inches and has been converted to steel section equal to 8.8 inches. The load used in this model is HS20-44 truck as live load. Figures 6.1 and 6.2 show the deflection and maximum moment of the steel girder from ABAQUS. While, Figures 6.4 through 6.6 show the girder model before loading, after loading, and effective stress induced by loading respectively.

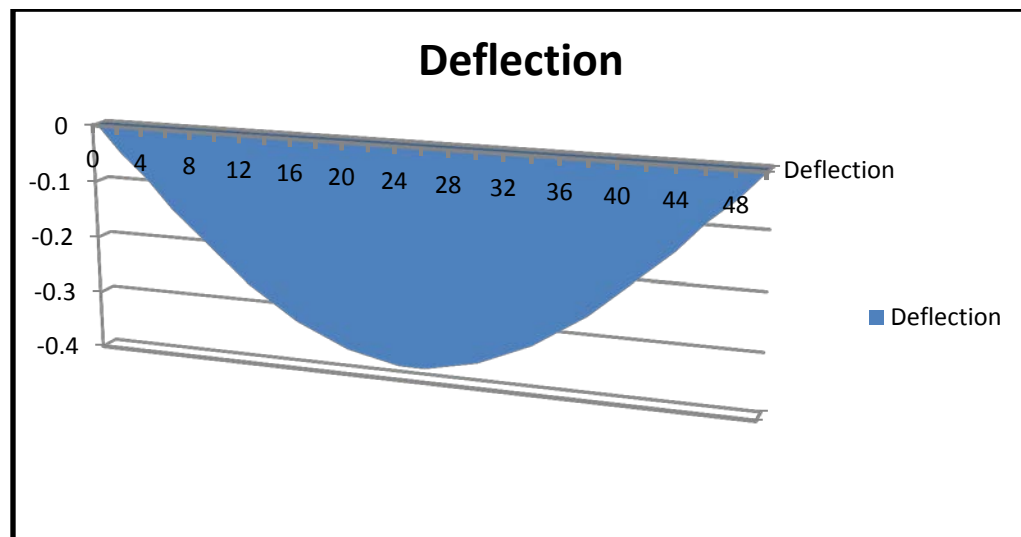


Figure 6.1: Deflection Versus Span from ABAQUS Girder Model.

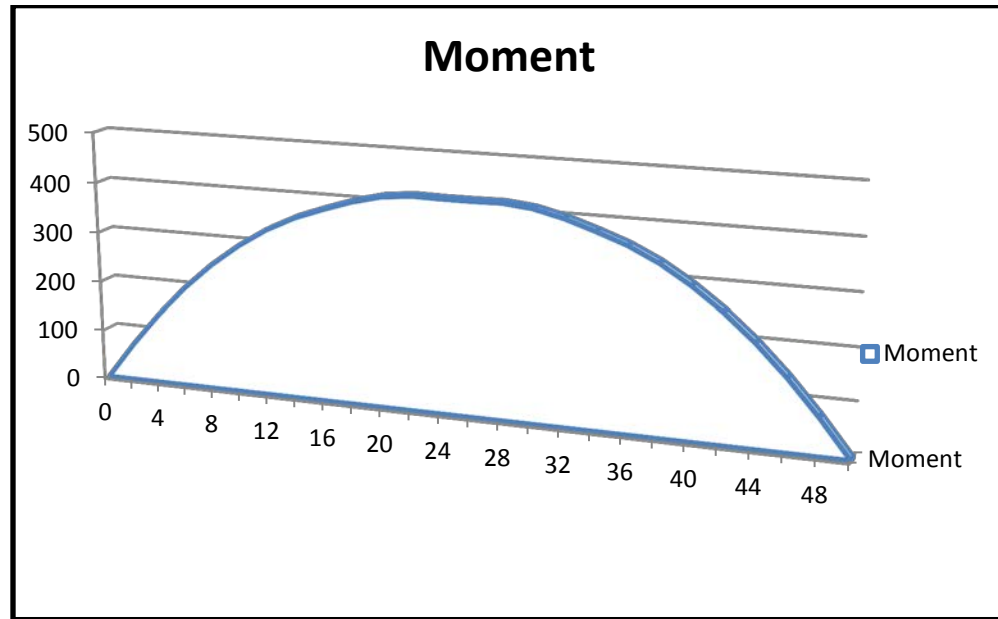


Figure 6.2: Moment Versus Span from ABAQUS Girder Model.

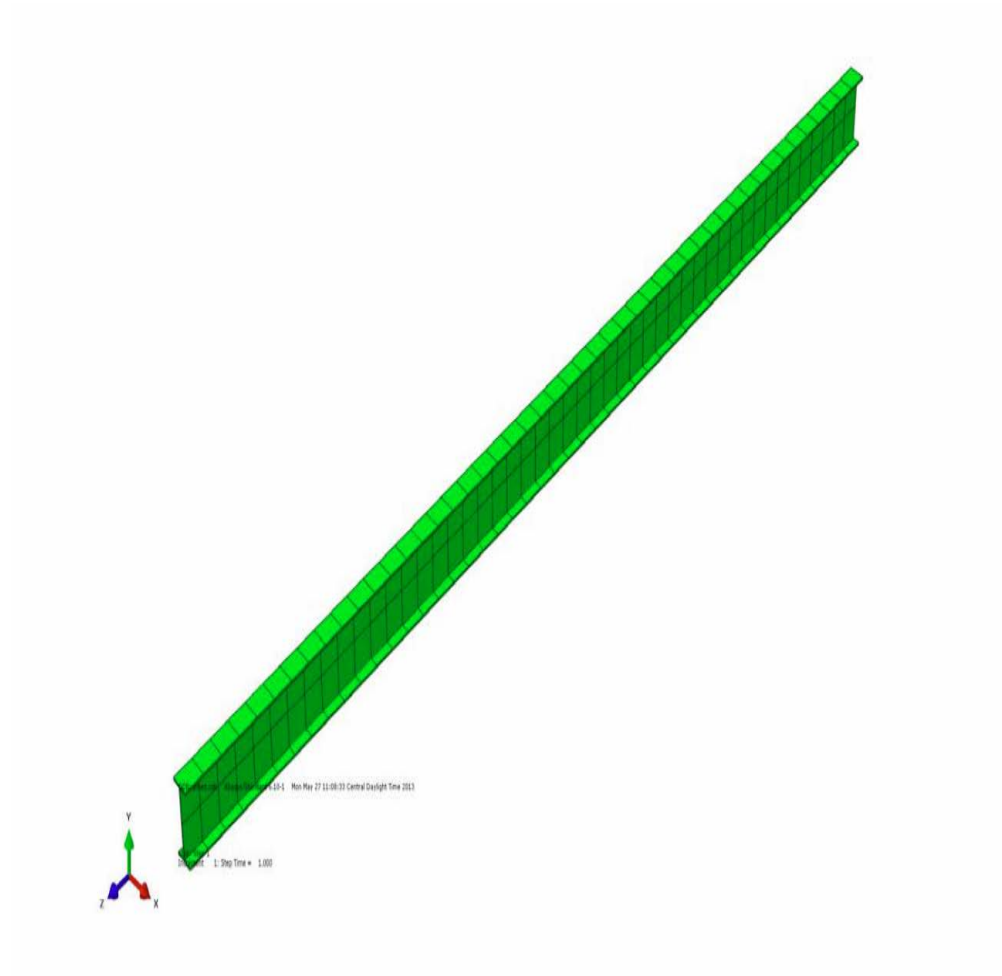


Figure 6.3: ABAQUS Girder Modeling Showing the Mesh

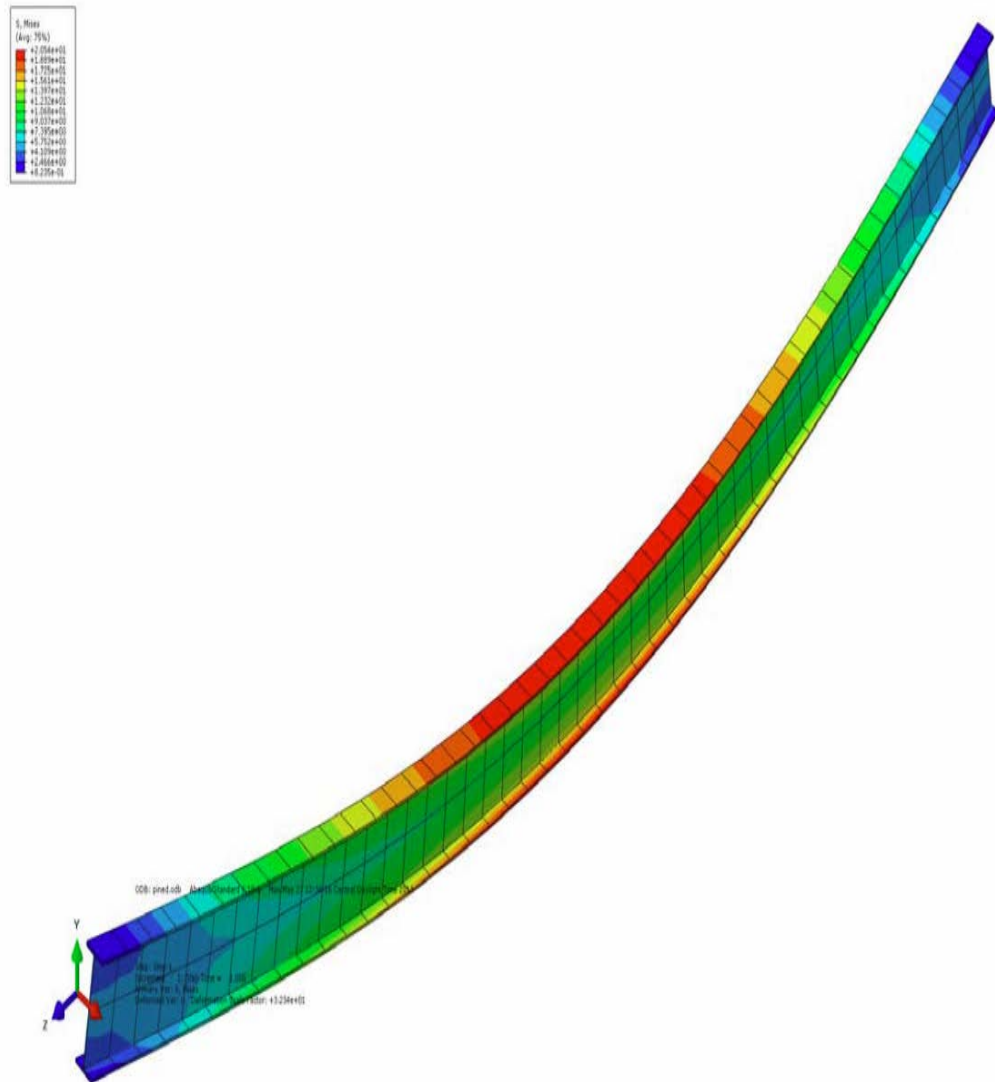


Figure 6.4: ABAQUS Girder during Loading Showing the Deflection

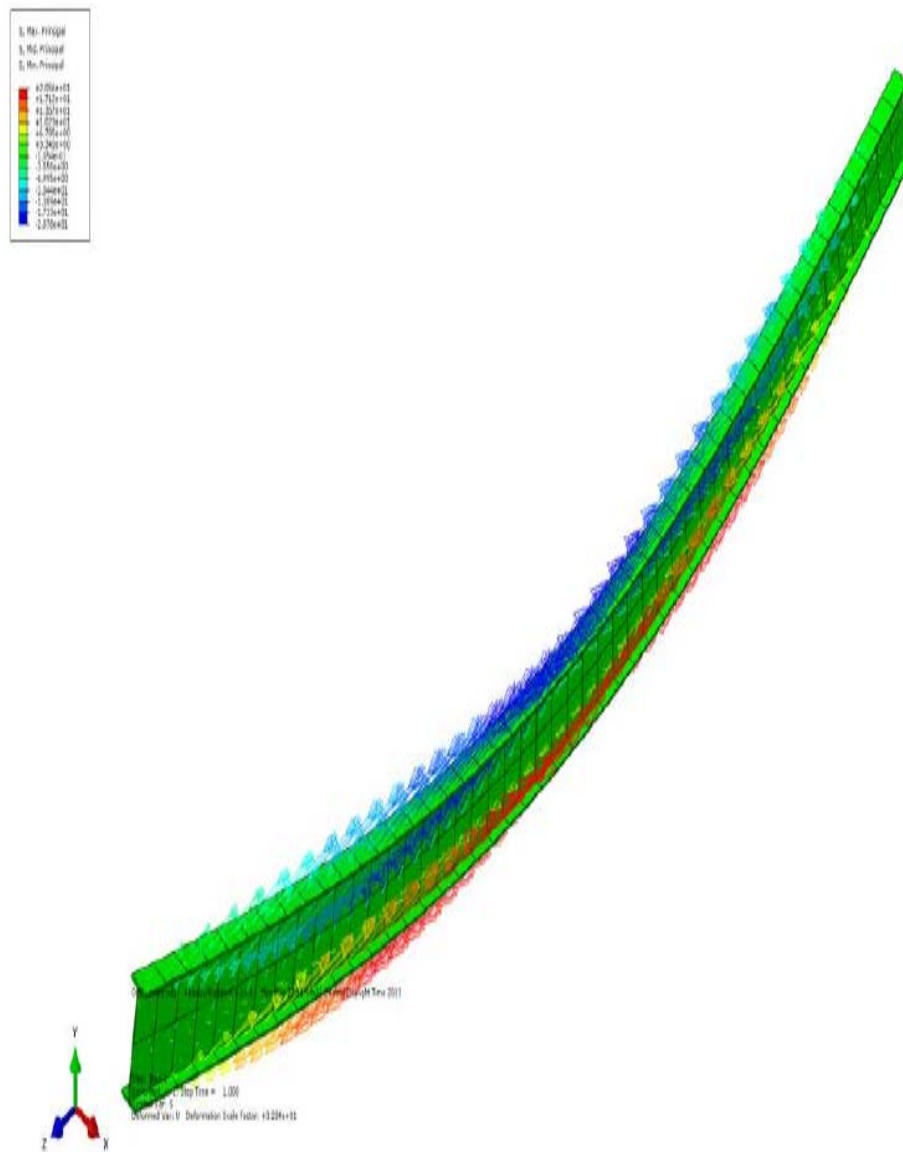


Figure 6.5: ABAQUS Girder Stresses Distribution during Loading

CHAPTER 7

RELIABILITY ANALYSIS

7.1 INTRODUCTION

The reliability analysis procedure has been conducted in this chapter to evaluate the composite steel girder bridge according to the function performance level represented as reliability indices of its corroded steel girder. The sample bridge is simply supported with two lanes and no pedestrian routes. The corrosion penetration models for moment and shear have been discussed in chapter 5 depending on the location of steel section at mid span or at its supports respectively. This cross section model will be used in calculating the resistance of the girder. The reliability theory has been used in many scientific researches that submitted reasonable realistically results which made it more frequently used in the design and the risk analysis. The reliability indices will be determined by using the approach explained in chapters 2, which introduce the limit states function (as performance scale) that controls the decision made, which will be defined approximately by the reliability index.

The corroded composite steel girder bridge has serious moment capacity reduction. The shear capacity reduction and deflection increased due to decreasing of the steel stiffness. The AASHTO-LRFD code limit state functions have been used in this study and the target reliability is 3.5. The serviceability limit state (deflection) of steel girder is equal to $L/800$. The live load used in the testing software is HS20-44, which a moving truck is crossing the bridge.

7.2 RELIABILITY ANALYSIS

The calculations have been determined using the results obtained from ABAQUS finite element programming. The distribution of loads per girder can be modeled in several ways. One of the most is using the distribution factors from the weigh-in-motion model (Kayser, 1988). For this study the finite elements method using ABAQUAS software employed AASHTO-LRFD with HS20-44 moving load truck to examine the girder resistance by distributing the load depending on the structure analysis on each girder and then, running the program and recording the deflection for each case which, required changing the distribution of forces several times to get the deflection value approximately. The deflection scale is 0.37 in. with no corrosion. The value of deflection limit state according to AASHTO-LRFD code is equal to 0.75 in. Consequently, the bending moment and shear has been determined. Tables 7.1 through 7.3 show the reliability indices versus exposure year and corrosion penetration for moment, shear and deflection.

Table 7.1 Reliability Indices for Deflection

Years	Nominal Low Corrosion Penetration x 0.000039 in	Reliability Index β	Nominal Medium Corrosion Penetration x 0.000039 in	Reliability Index β	Nominal High Corrosion Penetration x 0.000039 in	Reliability Index β
10	0	4	0	4	0	4
20	42	3.89	85	3.78	240	3.55
40	150	3.6	370	3.44	1000	3.15
60	375	3.1	784	2.69	1875	2.33
80	500	2.84	1042	2.4	2330	1.84
100	563	2.5	1125	2.1	2635	1.5

Table 7.2: Reliability Indices for Moment Capacity

Years	Nominal Low Corrosion Penetration x 0.000039 in	Reliability Index β	Nominal Medium Corrosion Penetration x 0.000039 in	Reliability Index β	Nominal High Corrosion Penetration x 0.000039 in	Reliability Index β
10	0	4.2	0	4.2	0	4.2
20	42	4.16	85	4.15	240	4.04
40	150	4.14	370	4.08	1000	3.9
60	375	4.07	784	3.96	1875	3.63
80	500	4.03	1042	3.86	2330	3.48
100	563	4.01	1125	3.8	2635	3.39

Table 7.3: Reliability Indices for Shear Capacity

Years	Nominal Low Corrosion Penetration $\times 0.000039$ in	Reliability Index β	Nominal Medium Corrosion Penetration $\times 0.000039$ in	Reliability Index β	Nominal High Corrosion Penetration $\times 0.000039$ in	Reliability Index β
10	0	3.87	0	3.87	0	3.87
20	42	3.84	85	3.8	240	3.68
40	150	3.75	370	3.56	1000	2.99
60	375	3.56	784	3.19	1875	2.08
80	500	3.46	1042	2.97	2330	1.52
100	563	3.4	1125	2.88	2635	1.16

7.3 GRAPHS OF RELIABILITY ANALYSIS

Figures 7.1 through 7.3 show the reliability indices of moment capacity versus exposure years to corrosion. While, Figures 7.4 through 7.6 show the reliability indices of maximum deflection versus exposure years to corrosion. Finally, Figures 7.7 through 7.9 present the reliability indices of shear capacity versus exposure years to corrosion.

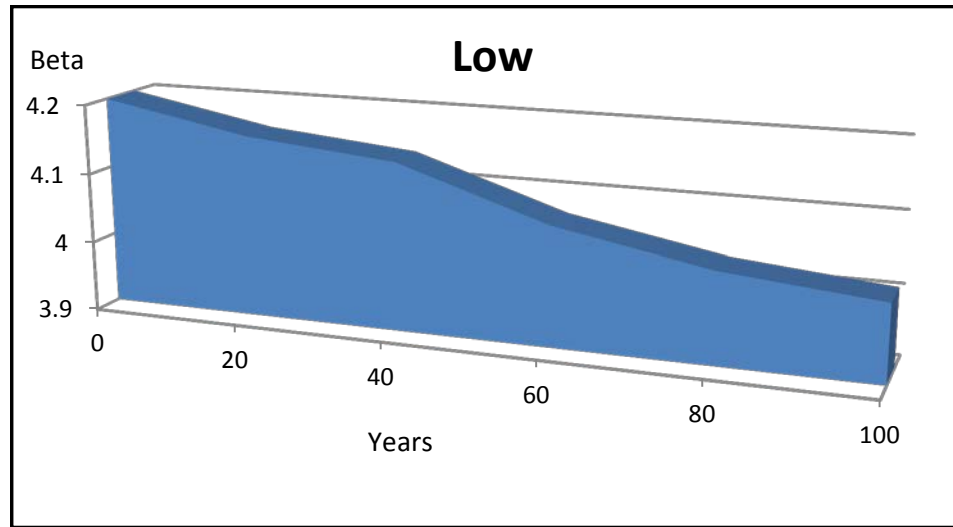


Figure 7.1: Reliability of Moment capacity Versus Exposure Years

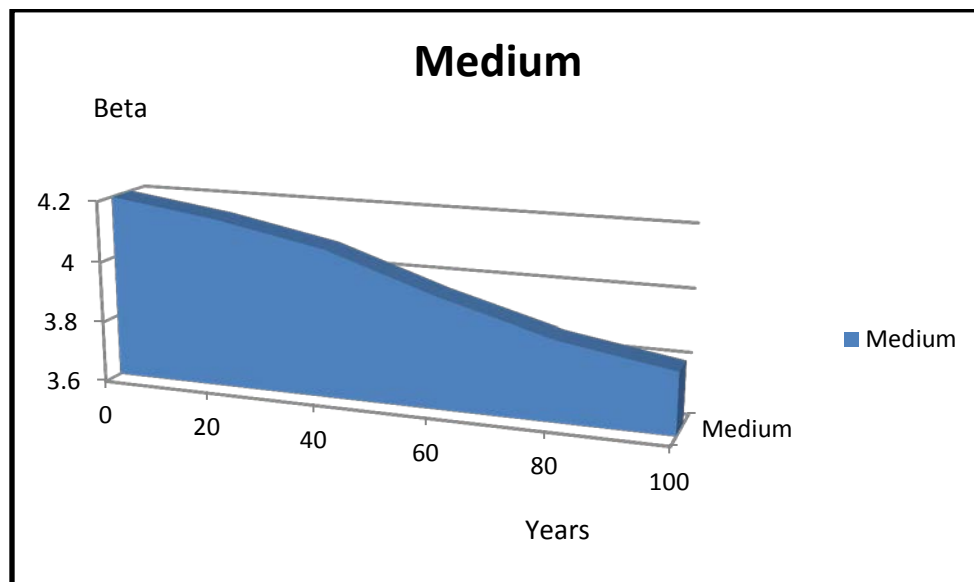


Figure 7.2: Reliability of Moment Capacity Versus Exposure Years

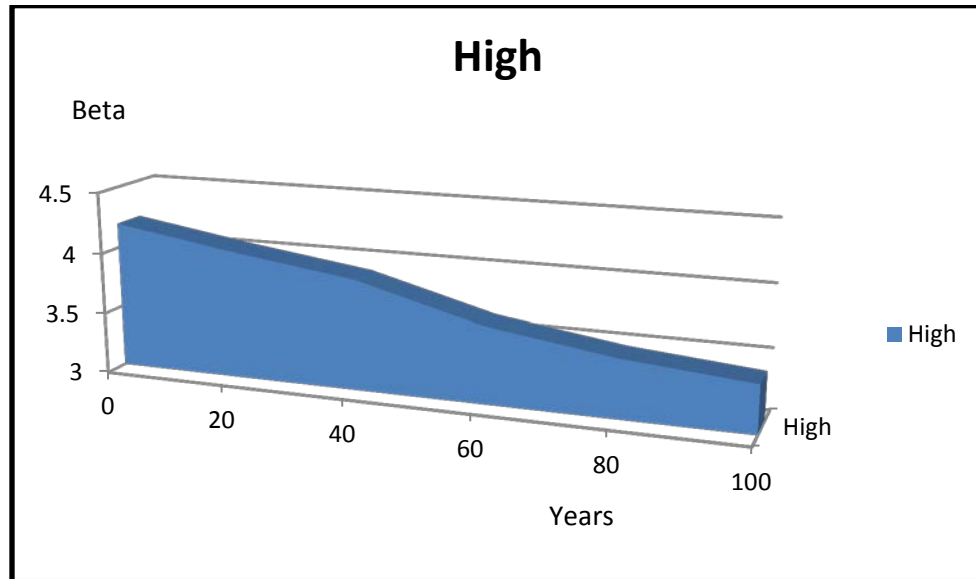


Figure 7.3: Reliability of Moment Capacity Versus Exposure Years

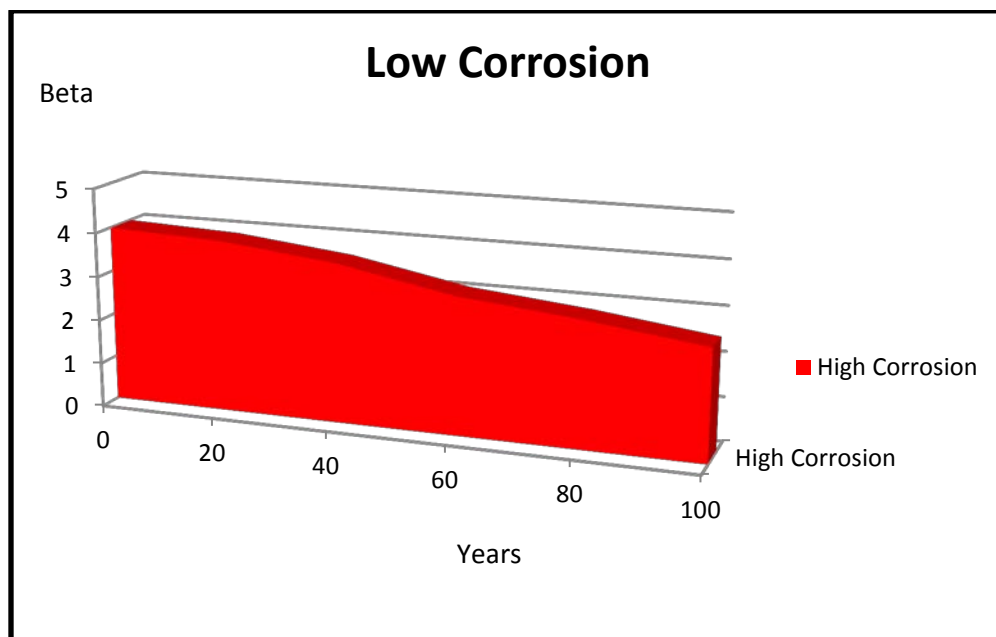


Figure 7.4: Reliability of Maximum Deflection Versus Exposure Years

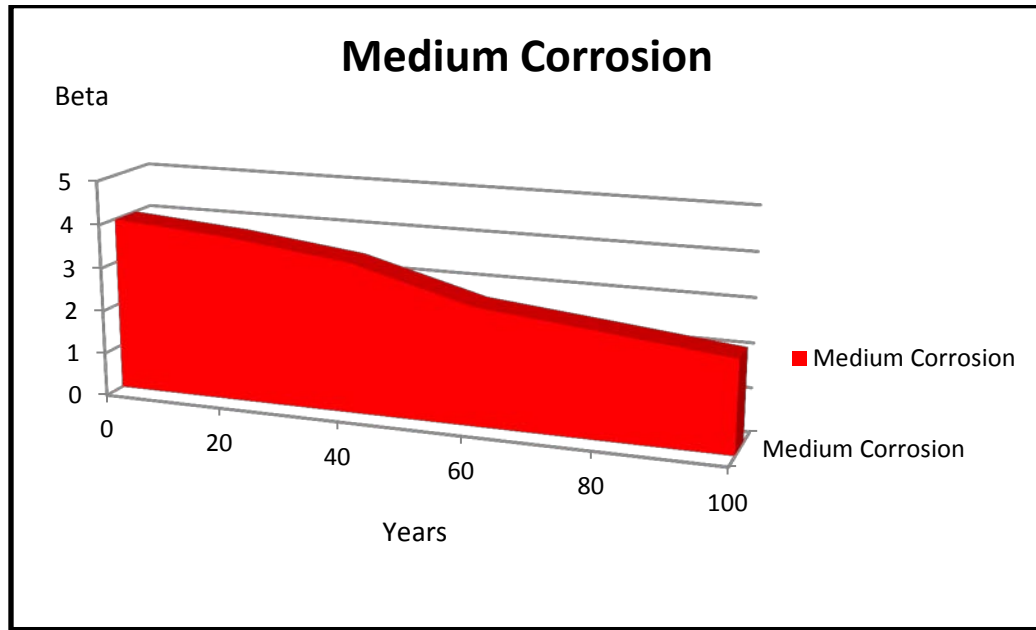


Figure 7.5: Reliability of Maximum Deflection Versus Exposure Years

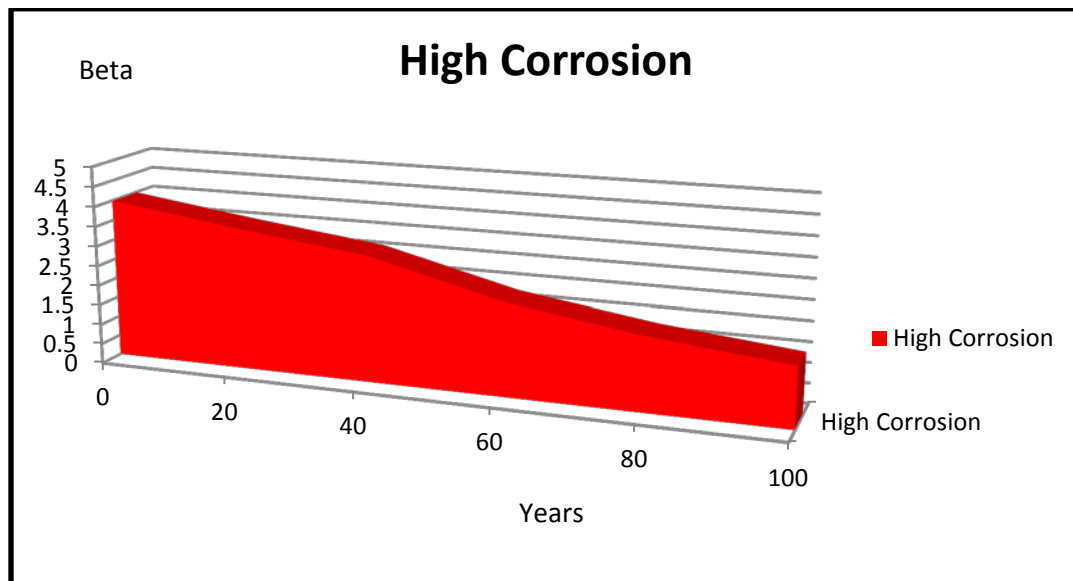


Figure 7.6: Reliability of Maximum Deflection Versus Exposure Years

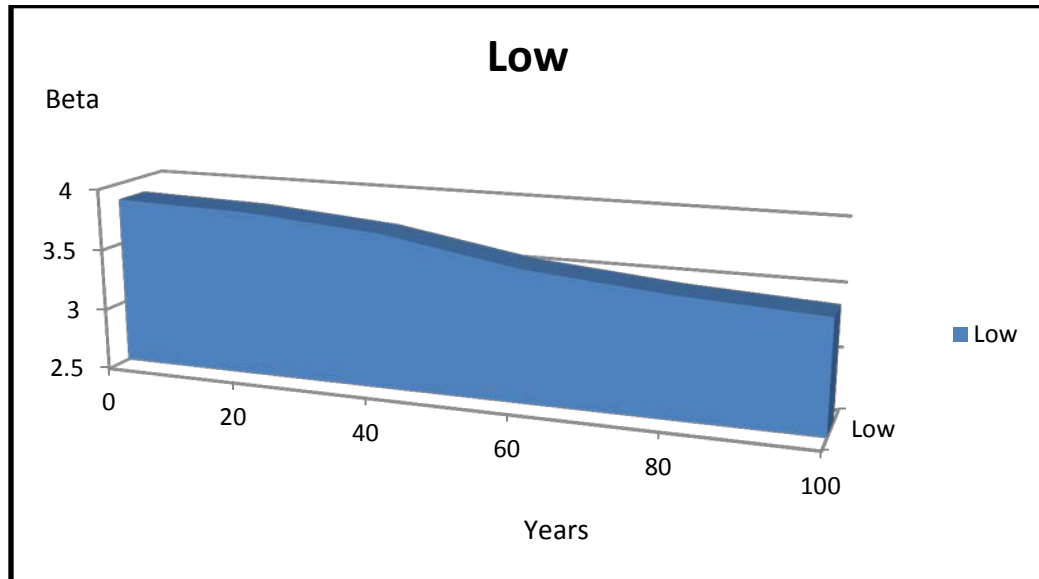


Figure 7.7: Reliability of Shear Capacity Versus Exposure Years

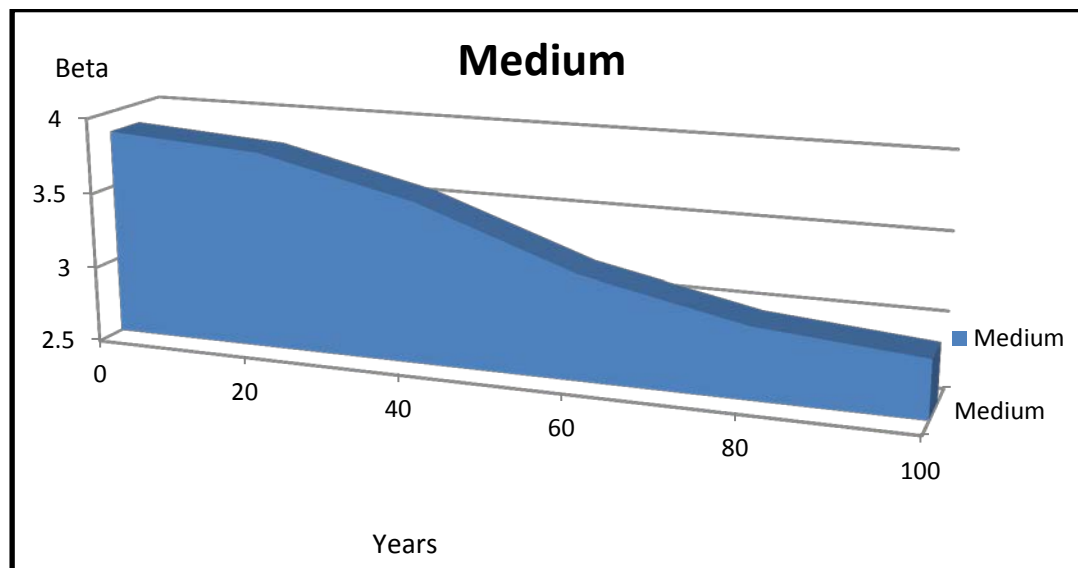


Figure 7.8: Reliability of Shear Capacity Versus Exposure Years

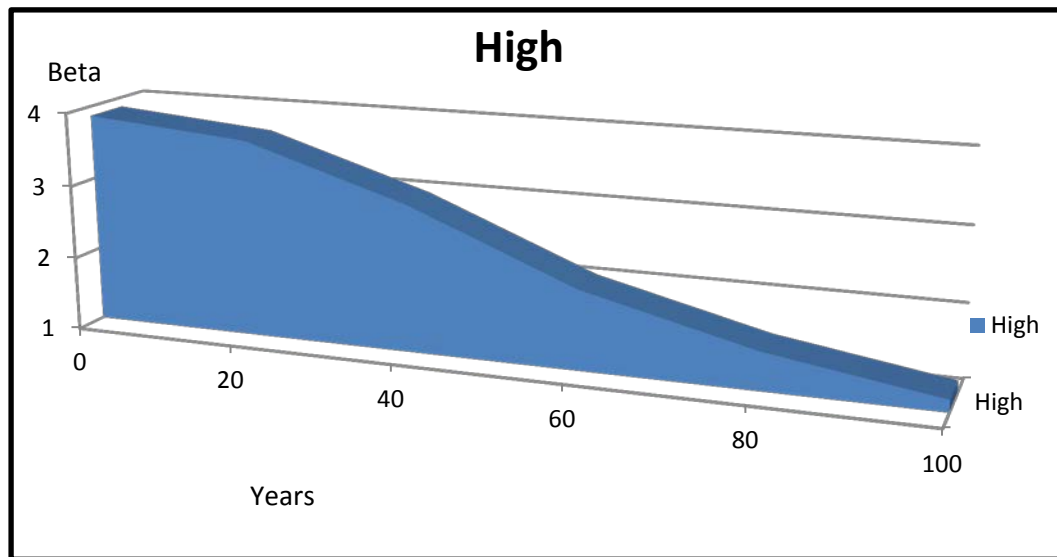


Figure 7.9: Reliability of Shear Capacity Versus Exposure Years

CHAPTER 8

CONCLUSIONS AND FUTURE WORK

8.1 SUMMARY

This study is focused on evaluating of a composite steel girder bridge structural. The reliability indices have been calculated for a specified bridge in Nebraska State which had corroded girders. The problem of corrosion was dealing with as outcome of a long time exposure where the environmental impacts take place. The increasing number of deteriorating infrastructures (bridges) needs more attention from the designer's aspect. In parallel action, the evaluation procedure and rehabilitation process are becoming important topics in bridge field. The steel girder bridges are the typical structures in the highways system. The targeted steel girder bridge has been evaluated in this research through ultimate and service limit states. According to results, the designed live load capacity may be influenced; it is simply depend on the size of the affected area and corrosion rate. The corrosion penetration was determined according to (Park, 1999) data for the steel girders.

8.2 CONCLUSIONS

The main conclusions of this research are outlined as follows:

- This research showed that the reliability can be considered as a rational measure of performance for structures in the bridge design and for the evaluation of an existing bridge.
- ABAQUS software girder simulation presented the large stresses effecting the lower flange of the steel girder Therefore, using the steel plate is an economic way for the design and maintenance.
- The reliability analysis procedure was demonstrated on a corroded composite steel girder of a specified bridge in Nebraska.
- Load and resistance parameters were defined, the limit state function for the girder was formulated, and the reliability analysis was performed.
- The reliability indices for moment had a range of (3.39-4.2) depending on low, medium, and high corrosion respectively.
- The reliability indices for shear had a range of (1.16-3.87) depending on low, medium, and high corrosion respectively.
- The reliability indices for deflection had a range of (1.5-4) depending on low, medium, and high corrosion respectively.
- The shear had the lowest reliability index and it had a large effect on the shear capacity and the bearing capacity of the steel girder.
- The high corrosion effect if occurred needed to be monitored and estimated in order to avoid more maintenance cost.

- The deflection of this bridge after 50 years of corrosion was more than AASHTO limit state that equal (0.75 in). That means the problem of vibration may start and the deck will face more cracking.
- This bridge may need to redesign the bearing area or adding bearing stiffeners according to the decrease of bearing resistance.
- The announced live load capacity of the examined bridge may be reduced according to the results of this research to maintain a safety operation.

8.3 FUTURE WORK

- Further work related to corrosion with reliability analyses, the system reliability of bridges can be considered instead of component reliability analysis.
- In addition, as an advantage of this analysis the live cycle cost needs to be considered.
- For evaluating the effect of corrosion accurately, the non-destructive testing method of bridges should be recommended in this analysis.

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