Target Reliability Analysis for Structures

by

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Abstract

Structural performance depends on load and resistance parameters, in particular magnitude and frequency of load components and their combinations, strength of materials, modulus of elasticity, dimensions, rate of deterioration, and so on. The problem is that these parameters are time-varying random variables and the structure is expected to perform its function in an acceptable manner. Therefore, prediction of structural performance involves uncertainty. Reliability is defined as a probability of the structure to acceptably perform its function. There is a need to determine the optimum or target reliability level, as too low reliability can result in problems (e.g. cracking, vibrations, collapse) and too high reliability can be costly. The papers in this dissertation deal with selection criteria for the target reliability.

Acceptable performance has to be defined and this can be very subjective. The boundary between acceptable and unacceptable performance is called a limit state. Mathematical expression that defines a limit state is the limit state function. In general, a limit state function, g, is a function of load and resistance parameters,

$$g(X1, \dots Xn)$$

and $g > 0$ or $g = 0$ represents acceptable state of the structure and $g < 0$ represents unacceptable state, or failure to perform the expected function. Therefore, the objective of reliability analysis is to determine the probability of g being negative, or probability of failure. Once the limit state function is formulated as a function of load and resistance parameters, the

probability of failure and corresponding reliability index can be calculated using one of the available procedures.

There are several procedures available for performing the reliability analysis and they are presented in textbook, including Nowak and Collins (2013). They vary with regard to accuracy, required effort and simplicity of form. The direct calculation of probability of failure, P_F, is complicated as it involves a double integration and convolution functions. A practical procedure, allowing to avoid double integration, to determine the probability of failure and reliability is proposed in paper #1.

Therefore, the major developments were directed at calculation of the so called reliability index, β. If g is treated as a normal random variable then the relationship between β and P_F is

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

where Φ^{-1} is the inverse standard normal distribution. Otherwise, the formula involves a certain degree of approximation. This problem is considered in paper #1 in this dissertation.

There are four types of limit states: ultimate (or strength) limit state (ULS), serviceability limit state (SLS), fatigue limit state and extreme events limit state. Each limit state provides different acceptability criteria. Examples of ULS include moment or shear carrying capacity of a beam, column buckling, or overall stability of a beam. Examples of SLS include a limit on deflection, excessive vibrations, or cracking of concrete. Fatigue limit state is a limit on stress and/or number of load cycles. Extreme events include resistance to earthquakes and vessel/vehicle collision. The definition of what is acceptable performance and what is not can be very subjective. For example, what is acceptable deflection in a bridge due to traffic load? The answer to this question is very difficult, as the answer is not just one value but values of deflection and associated frequencies of allowable occurrence. Live load representing traffic on the bridge consists of a mixture of vehicles. Extremely heavy trucks can occur relatively infrequently. By measurements and structural analysis, it is possible to develop a cumulative distribution function (CDF) of deflection. So there is a need to select acceptable/tolerable CDF of deflection. This issue is a subject of paper #4.

In general, reliability has to be calculated separately for each of the considered limit states. The target reliability can also be different for each limit state as it depends mostly on consequences of failure. What happens if the limit state is exceeded? If there are serious consequences (closure for traffic, partial or total collapse, injuries, fatalities), the target reliability has to be increased to avoid major disaster. On the other hand, if not much happens, a lower reliability level can be tolerated. An example can be deflection limit state in bridges. If an extremely heavy truck crosses a bridge causing an excessive deflection, there no serious damage to the bridge, as it can still perform its function.

The other major factor that affects selection of the target reliability level is economics. If increasing safety is costly, a lower reliability can be acceptable. On the other hand, if safety is cheap, a large safety margin can be economically justified. For example, bolted connections have a very high reliability because safety is cheaper in bolts compared to safety in beams and columns these bolts connect.

There are also other factors that affect the selection of the target reliability such as past practice, risk perception by the society and political aspects.

A practical implementation of the target reliability concepts is presented in paper #3. The objective is calculation of load and resistance factors for concrete circular tunnels. This study involved formulation of the limit state functions for the considered structures. The statistical parameters were determined for the major variables, representative for the current practice. A reliability analysis procedure was developed and applied to calculate a wide spectrum of reliability indices. Using the current practice as a benchmark, the target reliability index was selected for each limit state, including moment, shear and compression capacities. Finally, the resistance factors were selected so as to minimize the closeness to the target reliability index.

Dedication

To my father, Seyed Masoud Ghasemi and my mother, Homa Zandi.

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Abbreviations

AASHTO	American Association of State Highway and Transportation Officials
ADTT	Average Daily Truck Traffic
AISC	American Institute of Steel Construction
ANSI	American National Standards Institute
ASCE	American Society of Civil Engineers
CDF	Cumulative Distribution Function
LRFD	Load and Resistance Factor Design
PDF	Probability Distribution Function
MTP	Multiple Truck Presence
SLS	Serviceability Limit State
TR	Target Reliability
ULS	Ultimate Limit State
WIM	Weigh In Motion

Symbols

c	constant parameter of the wave propagation speed
C	average corrosion penetration rate
C_F	failure cost
C_I	initial cost
C_M	maintenance cost
C_T	total cost
erf	error function
$erfc$	complimentary error function
$E[X]$	expectation operator
EI	bending stiffness
f_{mm}	the maximum mean value factor
$f_x(x)$	probability distribution function
$F_x(x)$	cumulative distribution function
F_y	yield stress
F_{y_n}	nominal yield stress
$F_y(t)$	time dependent yield stress, where here it is not considered
h_w	web height
i	inflation rate
L	bridge length

m	ratio of the maintenance cost to the initial cost
M_D	moment due to the dead load
M_L	moment due to the HL-93
M_W	moment due to the wearing pavement
M_{D_n}	nominal moment due to the dead load
M_{L_n}	nominal moment due to live load
M_{W_n}	nominal moment due to the wearing pavement
n	ratio of the failure cost to the initial cost
N_e	number of extreme events in a year
P_d	probability of damage at time of the maintenance
P_f	probability of the failure
P_r	probability of random variable (X)
Q	random variables of load
R	random variable of resistances
$R(t)$	time dependent resistance
S_I	structural importance factor
t	Time
t_o	construction time
t_m	maintenance time
t_{end}	failure time
T_a	time period
T_N	web thickness
x	first space dimension, alongside of the wave propagation
y	second space dimension, time dependent displacement

Z	plastic modulus of section
Z_n	nominal plastic modulus of section
Z_{XC}	plastic modulus of the corroded I section
Z_{XN}	plastic modulus of the new I section
$Z(t)$	time dependent plastic modulus of section
α_{def}	displacement modification factor
$\beta(t)$	time-dependent reliability
γ_1	Skewness
γ_D	dead load factor
μ_g	mean of the limit state function
γ_L	live load factor
γ_W	surface wearing load factor
μ_Q	mean value of load
μ_R	mean value of residence
${\mu^e}_R$	equivalent normal mean value
ξ_w	percentage loss of the thickness of web
ξ_f	percentage loss of the thickness of flange
σ	standard deviation
σ_g	standard deviation of the limit state function
σ_Q	standard deviation of load
σ_R	standard deviation of resistance
${\sigma^e}_R$	equivalent normal standard deviation
$\Phi_f(.)$	cumulative distribution of failure
$\Phi_d(.)$	cumulative distribution of maintenance

Φ^{-1}	inverse of cumulative distribution of standard normal function
${\Phi_f}^{-1}(.)$	inverse of cumulative distribution of standard normal failure
${\Phi_d}^{-1}(.)$	inverse of cumulative distribution of standard normal of maintenance
$\omega(x,t)$	displacement of any point (x) at any time (t)

1. Reliability Index for Non-Normal Distributions

Abstract I

Reliability analysis is a probabilistic approach to determine the safety level of a system (or a structure, in structural engineering). Reliability is defined as a probability of the system to functionally perform under given conditions. In the 1960s, Basler (1961) and Cornell (1969) defined the reliability index as an indicator to represent the safety level of the system, which until today is a commonly used parameter. However, the reliability index was formulated based on the normally distributed probability of failure. Nevertheless, it is not guaranteed that the probability of failure of the system follows a normal distribution; therefore, there is a need to determine the reliability index for non-normal distributions.

The objective of this research is to define the reliability index for non-normal distributions. In order to introduce the reliability index for a non-normal distribution, the probability of failure is defined first based on the convolution operation and then it is calculated for that probability failure function.
Keywords: *Reliability Analysis, Convolution Theory, Non-normal Distribution, Probability of Failure.*

1.1.Introduction

To perform the reliability analysis of the structures, instead of using the deterministic capacity of the structure and applied load, it is required to utilize the statistical parameters of the load and/or resistance.

The objective of the reliability analysis is to calculate the safety level of the structures or its components. The definition of reliability index was first given by Basler (1961) and Cornell (1969) as:

$$\beta = -\Phi^{-1}\left(P_f\right) \tag{1.1}$$

where,

$\beta =$ reliability index
$\Phi^{-1} =$ inverse of the cumulative distribution of the standard normal function

Based on that definition, the reliability index is related to the probability of failure of a normal distribution.

Reliability Analysis

In structural engineering, the probability failure is defined as the probability of structural failure during its design period (lifecycle). Accordingly, reliability is defined as a probability of structural performance during its lifecycle. To compute the level of reliability of structures, the reliability index, β, is a commonly used parameter. The reliability index is defined as the inverse of the coefficient of variation of the Cumulative Distribution Function (CDF) of the limit state function. The graphical definition of the reliability index is the shortest distance from the origin in the reduced variables space state to the limit state function. For instance, if R represents the reduced variables of resistance, Q indicates the reduced variables of load and the limit state is defined as:

$$g = R - Q \tag{1.2}$$

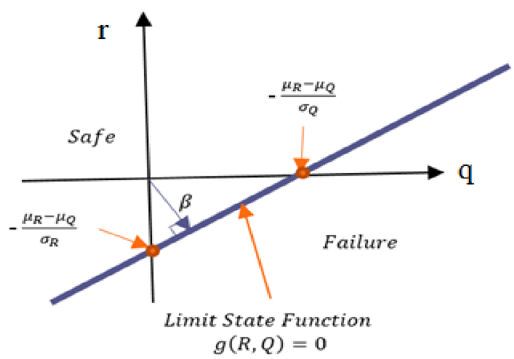

Figure 1.1. Graphical Definition of the Reliability Index, by Nowak and Collins (2013)

r represents the reduced variables of resistance, q indicates the reduced variables of loads and the limit state is defined as $g = R - Q$

Nowak and Collins (2013) summarized the procedure to determine the reliability index as follows

1. Structural loading and load effect
   - Consider the applied load on the structure
   - Determine the statistical parameters of load
2. Structural resistance
   - Consider the structural resistance
   - Determine the statistical parameters of resistance
3. Balance between load effect and structural resistance
   - Establish the limit state function $g = R - Q$
   - Determine the reliability index

Depending on the limit state function, several approaches were introduced to compute the reliability index. For instance, if the variables follow a normal distribution, the Hasofer-Lind (1974) method is one of the appropriate approaches; however, if one of the variables is treated as a non-normal distribution, the Rackwitz-Fiessler (1978) method is an alternative approach. If the limit state consists of several random variables with different distributions, the Monte Carlo method is recommended.

The difference between the Probability Density Function (PDF) of the resistance and the PDF of the load leads to the probability distribution of failure, namely (P_f). The state of the failure is the condition when $g < 0$.

$$P_f = P(R - Q < 0) = P(g < 0) \quad (1.3)$$

Assuming statistical independence between R and Q, the joint PDF can be considered as the multiplication of the PDFs of R and Q ($f_{RQ}(\mathrm{r}, \mathrm{q}) = f_Q(q) f_R(\mathrm{r})$). Then, by taking the integration of $f_Q(q)$ with respect to q from r to infinity and finally computing the integration with respect to r over the entire possible domain, the probability of failure (P_f) can be determined. Figure 1.2 illustrates the graphical approach to evaluate P_f.

$$P_f = 1 - \int_{-\infty}^{+\infty} f_R(\mathrm{r}) F_Q(r) d\mathrm{r} \quad (1.4)$$

Alternatively, first, by taking the integration over $f_R(\mathrm{r})$ from negative infinity to q, P_f can be represented as follows:

$$P_f = \int_{-\infty}^{+\infty} f_Q(\mathrm{q}) F_R(q) d\mathrm{q} \quad (1.5)$$

Based on the reliability function definition, the reliability is the complementary function of the probability function; therefore, the reliability function can be demonstrated as follows (Thoft-Christensen, and Baker (1982)):

$$\mathrm{R} = \int_{-\infty}^{+\infty} f_R(\mathrm{r}) F_Q(r) d\mathrm{r} = 1 - \int_{-\infty}^{+\infty} f_Q(\mathrm{q}) F_R(q) d\mathrm{q} \quad (1.6)$$

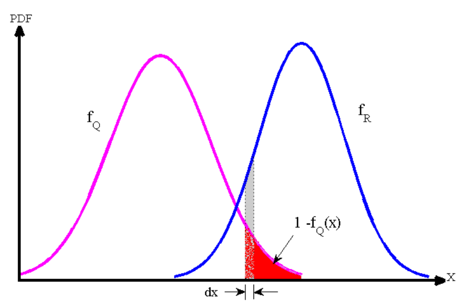

Figure 1.2. Integration Approach to Evaluate P_f, by Nowak and Collins (2013)

where f_R and f_Q are the probability density functions, and F_Q and F_R is the cumulative distribution functions (CDF). r and q are the random variables of the load and resistance, respectively. Mathematically, if both functions are normally distributed, it is proven that the probability distribution of failure also follows the normal distribution and the reliability index can be computed using Hasofer-Lind approach, which was also shown by Ditlevsen and Madsen (1996) and Nowak and Collins (2013).

$$\beta = \frac{\mu_R - \mu_Q}{\sqrt{\sigma_R{}^2 + \sigma_Q{}^2}} \quad (1.7)$$

where,

mean value of resistance
μ_Q = mean value of load
σ_R = standard deviation of resistance
σ_Q = standard deviation of load

Another way to determine the reliability index of normal distribution is the graphical method. Basically, as long as the distribution of the probability of failure is given and follows the normal distribution, the graphical method would be beneficial (Figure 1.3). Based on the graphical method, the reliability index is defined as the distance between the mean value and the safety margin ($g = 0$) in terms of the standard deviation. Equation 1.8 shows the formula to compute the reliability index using the graphical method:

$$\beta = \frac{\mu_g}{\sigma_g} \tag{1.8}$$

where,

μ_g = mean of the limit state function
σ_g = standard deviation of the limit state function

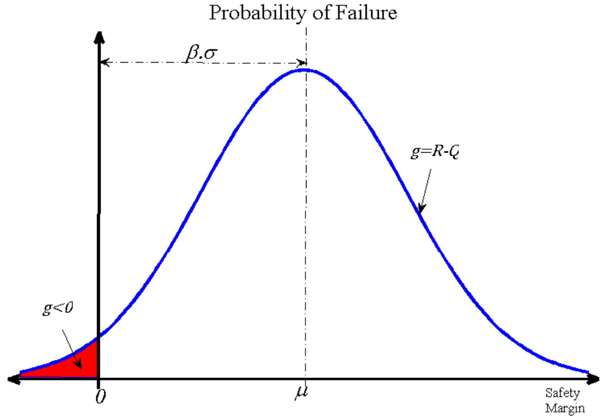

Figure 1.3. Graphical Relationship between the Reliability Index and Statistical Parameters of P_f, Nowak and Collins (2013)

However, if any of the aforementioned distributions (load and resistance) do not behave as a normal distribution, it is necessary to apply another method to find the reliability index. For instance, the Rackwitz-Fiessler (1978) method is a commonly used technique to determine the reliability. For example, if the resistance has a log-normal distribution and the load has a normal distribution, the reliability index can be written as follows (Nowak and Collins (2013)):

$$\beta = \frac{\mu_R\left(1-k\frac{\sigma_R}{\mu_R}\right)\left[1-\ln\left(1-k\frac{\sigma_R}{\mu_R}\right)\right]-\mu_Q}{\sqrt{\left(\mu_R\left(1-k\frac{\sigma_R}{\mu_R}\right)\left(\frac{\sigma_R}{\mu_R}\right)\right)^2+\sigma_Q{}^2}} \tag{1.9}$$

where k is the multiplication factor of the standard deviation, indicating the distance between the design point, x^*, and the mean value (Equation 1.10).

where,

$$k = \left(\frac{\mu_r - x^*}{r}\right) \tag{1.10}$$

It is worth mentioning that in this approach, the design point is a point on the failure distribution's boundary; the CDF, F_X, and the PDF, f_X, of the investigated distribution are approaching the CDF and the PDF of the normal distribution, respectively (Equations 1.11 and 1.12, Nowak and Collins 2013).

$$F_X(x^*) = \Phi\left(\frac{x^* - \mu^e{}_X}{\sigma^e{}_R}\right) \tag{1.11}$$

$$f_X(x^*) = \frac{1}{\sigma^e{}_X}\varphi\left(\frac{x^* - \mu^e{}_X}{\sigma^e{}_X}\right) \tag{1.12}$$

where,

$\mu^e{}_X$ = equivalent normal mean value
$\sigma^e{}_X$ = equivalent normal standard deviation

The Rackwitz-Fiessler method's result is sensitive to the k value because the distribution depends on the probability failure function, and there is no specific approach to recommend the k value.
The other popular method to compute the reliability index is Monte Carlo simulation (Nowak and Collins 2013). There are two ways to determine the reliability index from the Monte Carlo simulation:

Approach 1: The probability of the failure is the ratio of the number of failures, where $g < 0$, to the total number of samples, therefore, the reliability index is the inverse function of the CDF with respect to obtained probability.

Approach 2: The reliability index is the intersection of the limit state and the vertical coordinate.

According to the definition of the reliability index in Equation 1.1, it was defined based on the negative inverse function of the CDF of the normal distribution with respect to the probability. However, the probability function does not necessarily behave as a normal distribution function. Therefore, there is a need to define the reliability index for non-normal distributions. Hence, the authors of this paper introduce a new equation to compute the reliability index for a non-normal distribution as a negative inverse function of the CDF of the probability of failure of the non-normal distribution as:

$$\beta = -F_X{}^{-1}(P_f) \tag{1.13}$$

where, ${F_X}^{-1}$ is the inverse function of the CDF of the non-normal distribution of probability of failure. In the next section based on the convolution theory the probability of failure is first defined, and then a formula is proposed to determine the reliability index of a non-normal distribution.

1.2.Convolution of Probability Distributions

If the Probability of the failure, $P_f = \iint_{g(R,Q)<0} f_{RQ}(\mathrm{r},\mathrm{q})dqdr$, does not follow the normal distribution, the current available methods do not conclude to the accurate reliability index.

The reliability index is calculated based on the assumption that the distribution of values of the limit state function is normal. However, if the PDF of the probability of failure does not follow the normal distribution, the obtained reliability index is not rational. Therefore, in order to compute the reliability index of non-normally distributed variables, it is required to establish a closed-form formula. In doing so, the probability of failure first should be defined. This research applies the convolution operator to define the probability of failure.

Convolution, $f * g$, is defined as the integration of the product of the two functions after reversing and shifting one of them (Dimovski 1990).

$$(f * g)(t) \stackrel{\text{def}}{=} \int_{-\infty}^{+\infty} f(\tau)g(t-\tau)d\tau = \int_{-\infty}^{+\infty} f(t-\tau)g(\tau)d\tau \tag{1.14}$$

Based on the definition of convolution, in probability analysis, the convolution can be interpreted as the combination of two distribution functions. By changing the variables, the authors of this paper introduced the probability of failure based on the convolution theory. Therefore, for given limit state in Equation 1.2, the probability of failure can be determined by the following equation.

$$P_f = \big(R(x) * Q(-x)\big) \stackrel{\text{def}}{=} \int_{-\infty}^{+\infty} R(\tau)Q(x+\tau)d\tau = \int_{-\infty}^{+\infty} R(x-\tau)Q(-\tau)d\tau \tag{1.15}$$

Eventually, in order to find the reliability index for a non-normal distribution, this research recommends Equation 1.16.

$$\beta = -inv\left(\left(\int_{-\infty}^{+\infty}\big(R(x) * Q(-x)\big)d\tau\right)\right)\Big|_{(R>Q)} \tag{1.16}$$

Equation 1.16 is a proposed closed-form solution to compute the reliability index for non-normal distributions. To implant the proposed formula for probability of failure (Equation 1.15) and reliability index (Equation 1.16) this paper represents two examples. The *first* example assumes that the load behaves as a normal distribution and the resistance is also normally distributed.

$$Q = Normal(x, 2, 1)$$

$$R = Normal(x,5,1)$$

where,

$$Normal(x,\mu,\sigma) = \frac{1}{\sigma\sqrt{2\pi}} exp\left(-\frac{1}{2}\left(\frac{x-\mu}{\sigma}\right)^2\right)$$

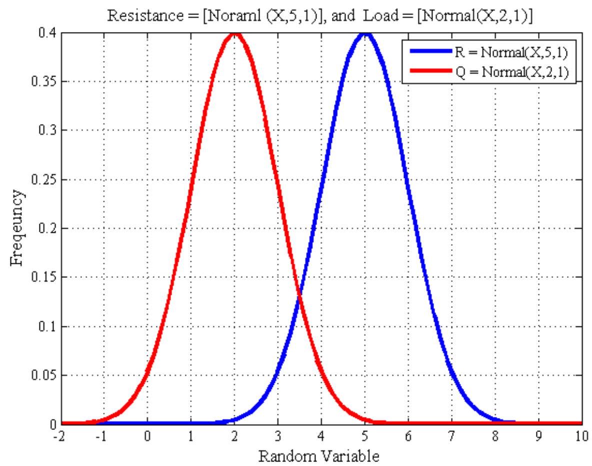

Figure 1.4A. Load and Resistance Distributions

The probability of failure is determined based on the Equation 1.15.

$$P_f = 0.28 \exp\left(-\left(\frac{x-3}{2}\right)^2\right)$$

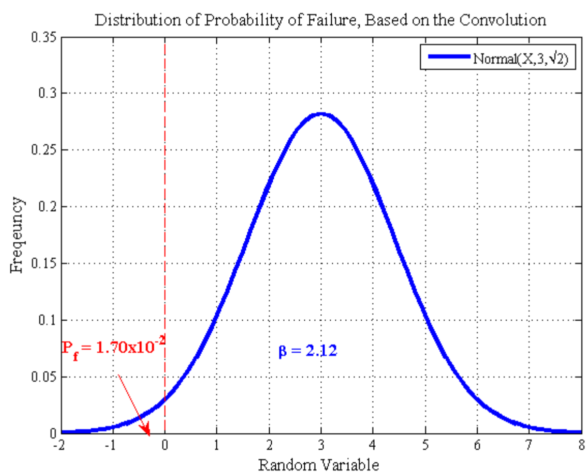

Figure 1.4B. Graphical P_f and Reliability index of $Normal(x,5,1)$ and $Normal(x,2,1)$ distributions

And the reliability index equals to $\beta = 2.12$

The *second* example assumes that the load behaves as a Gamma distribution, and resistance is normally distributed..

$$Q = Gamma(x, a = 2, b = 1) = \frac{1}{b^a\Gamma(a)} x^{a-1} \exp\left(\frac{-x}{b}\right),$$

$$R = Normal(x, 5, 1)$$

where,

Gamma function = $\Gamma(a) = \int_0^{\infty} \exp(-t) t^{a-1} dt = \Gamma(a) = (a-1)!$

! = factorial operator

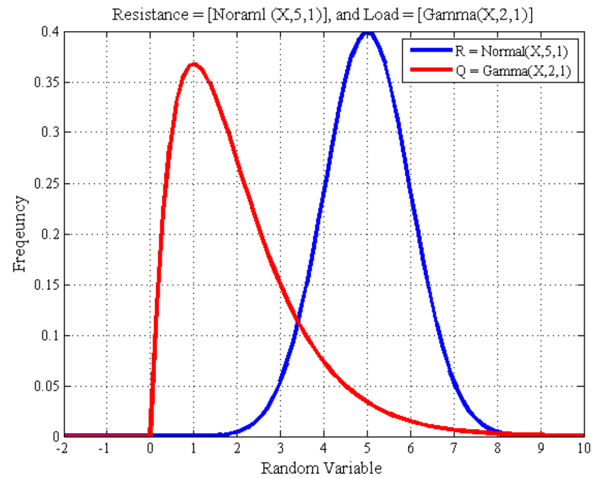

Figure 1.5A. Load and Resistance Distributions

Therefore using Equation 1.15 and Equation 1.16 respectively give the probability of failure and reliability index.

$$P_f = \int_{-\infty}^{0} \left(R(x) * Q(-x)\right) = 0.0555$$

$$\beta = -inv\left(\left(\int_{-\infty}^{+\infty} \left(R(x) * Q(-x)\right) d\tau\right)\right) = 2.2$$

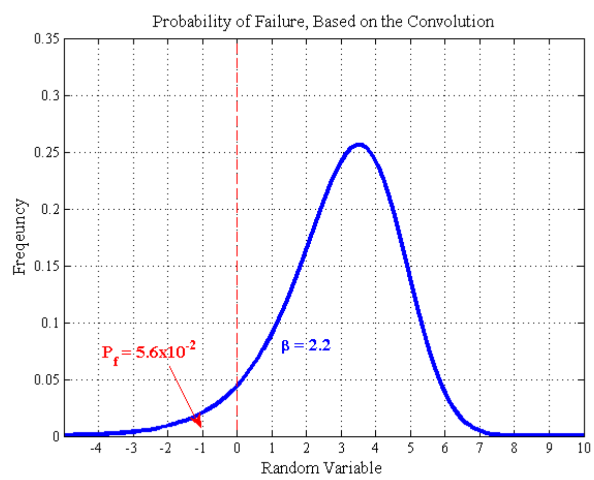

Figure 1.5B. Graphical Reliability index of $Normal(x, 5, 1)$ and $Gamma(x, a = 2, b = 1)$ distributions

Comparison between example one and example two shows that the resistance distribution is the same in both examples, but the load distribution in example one follows the normal distribution, which does not have any skewness, and the load distribution in example two follows the gamma distribution. Since most of the frequency content of the gamma distribution shifted to the left side, it is expected that the gamma distribution concludes to the higher value of the reliability index, therefore, the obtained results would be rational. These two examples are also examined by the Monte Carlo approach (see Nowak and Collins 2013).

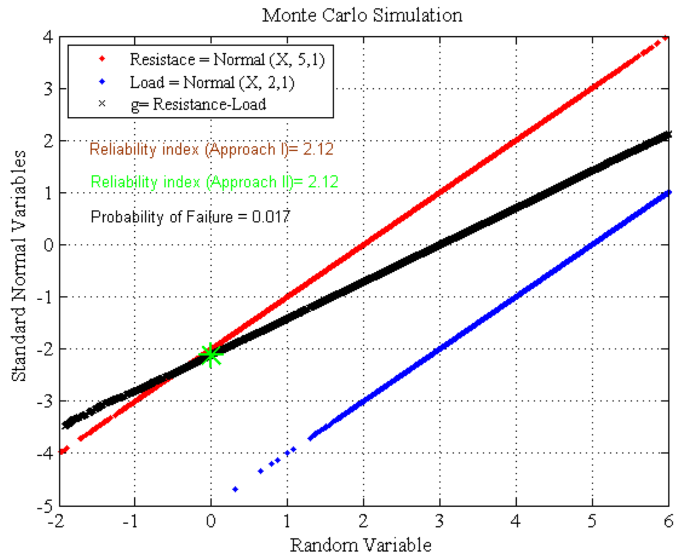

Figure 1.6A. Reliability index based on the Monte Carlo simulation, R=$Normal(x, 5, 1)$ and $Q = Normal(x, 2, 1)$

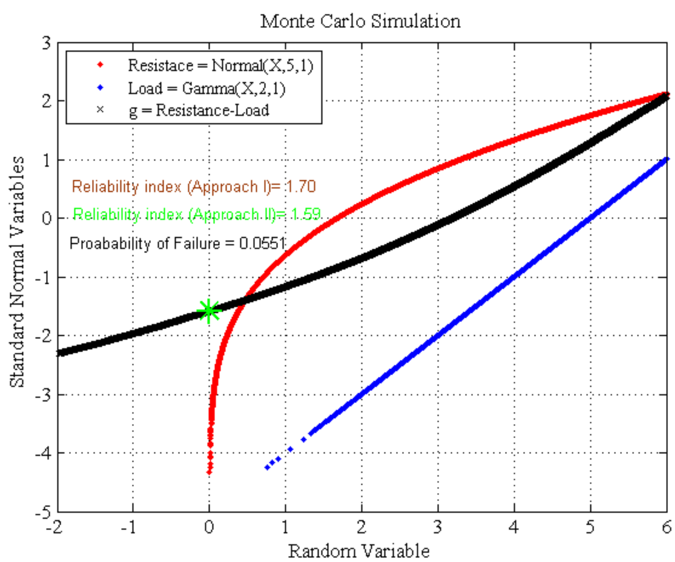

Figure 1.6B. Reliability index based on the Monte Carlo simulation, R=$Normal(x, 5, 1)$ and $Q = Gamma(x, 2, 1)$

As seen, the outcome of the Monte Carlo simulation for the normal distribution (for 100,000 samples) is the same as the convolution method, which could be claimed as a proof to the proposed method in Equation 1.15 and Equation 1.16.

In the second case, where, the load distribution behaves as a Gamma distribution, the result of the obtained probability of failure from the Monte Carlo simulation equals to the probability of failure resulting from the convolution method. Although the probability of the failure of the Monte Carlo simulation is reliable, the reliability index from this method could not be trusted, since Equation 1.1 determines the reliability index for normal distribution. Therefore, in order to consider the actual failure distribution it is recommended to utilize the proposed equation in this paper (Equation 1.16).

However, computing the reliability index using Equation 1.16 requires the advanced mathematics analysis. Therefore, there is a need to introduce a simplified proposed method.

1.3.Simplified Procedure to Calculate Reliability Index based on the Proposed Method

This section presents a procedure to calculate the reliability index for a non-normal distribution in a simple way. As a creative idea, it is possible to formulate any function in terms of the infinite sum of known functions. In 1715, Taylor developed an approach to formulate the function, which today is called as Taylor's series. Based on the Taylor's series, under certain conditions (differentiable function), it is possible to formulate any function in terms of a polynomial function. Also, in 1822, Fourier expressed that under particular conditions (periodic function), any function can be formulated with regard to the summation of the series of the periodic functions (such as: sin and cos). Here, It is proposed, under the certain conditions (continuous distribution), it is possible to formulate any distribution in terms of the sum of the Gaussian functions, Equation 1.17, (see Weisstein, (2014) to find the properties of the Gaussian function).

$$f_X(x) = \sum_{i=1}^{n} a_i Normal(x, \mu_i, \sigma_i) \quad (1.17)$$

where,

a_i = constant coefficient, which can be positive or negative

Therefore, the probability of failure is written in the form of the series of Gaussian functions. Accordingly, a Gaussian function can be converted to the fraction of a normal distribution, then, the reliability index is defined based on the summation of the reliability indices of the normal distributions.

$$\beta = a_1\beta_1 + a_2\beta_2 + \cdots + a_i\beta_i + \cdots + a_n\beta_n \quad (1.18)$$

Currently, there is an accessibility to the advanced mathematical software, such as MATLAB, the procedure to determine the reliability index is summarized as follows

1- Using the convolution operator to compute the probability of the failure based on the proposed Equation 1.15.
2- Fitting a distribution by utilizing the Gaussian function, which was defined in MATLAB curve fitting documentary.
3- Calculate the reliability index based on the Equation 1.18.

To illustrate the proposed simplified method, herein, the examples in the previous section are examined. Considering the first example, where both load and resistance are normally distributed, leads to a normally distributed probability of failure. Therefore, there is no need to fit Gaussian series over the Normal distribution. However, in other case where the resistance has a normal distribution and load follows the gamma distribution, the obtained probability of failure from convolution theory does not follow a normal distribution, therefore, it is possible to apply the proposed simplified approach to determine the reliability index.

The first step is to determine the probability of failure by utilizing the convolution operation.

$$P_f = Gamma(x,2,1) * Normal(x,5,1) = \int_{-\infty}^{+\infty} \tau\, exp(-\tau) \frac{1}{\sqrt{2\pi}} exp\left(-\frac{1}{2}(x+\tau-5)^2\right) d\tau$$

The equation above is implicitly solved by MATLAB. The second step is to fit a curve over the obtained probability function, using the Gaussian series. This step is also accomplished by MATLAB. As can be seen in Figure 1.7, using the just combination of two Gaussian functions can sufficiently provide an appropriate fitting curve.

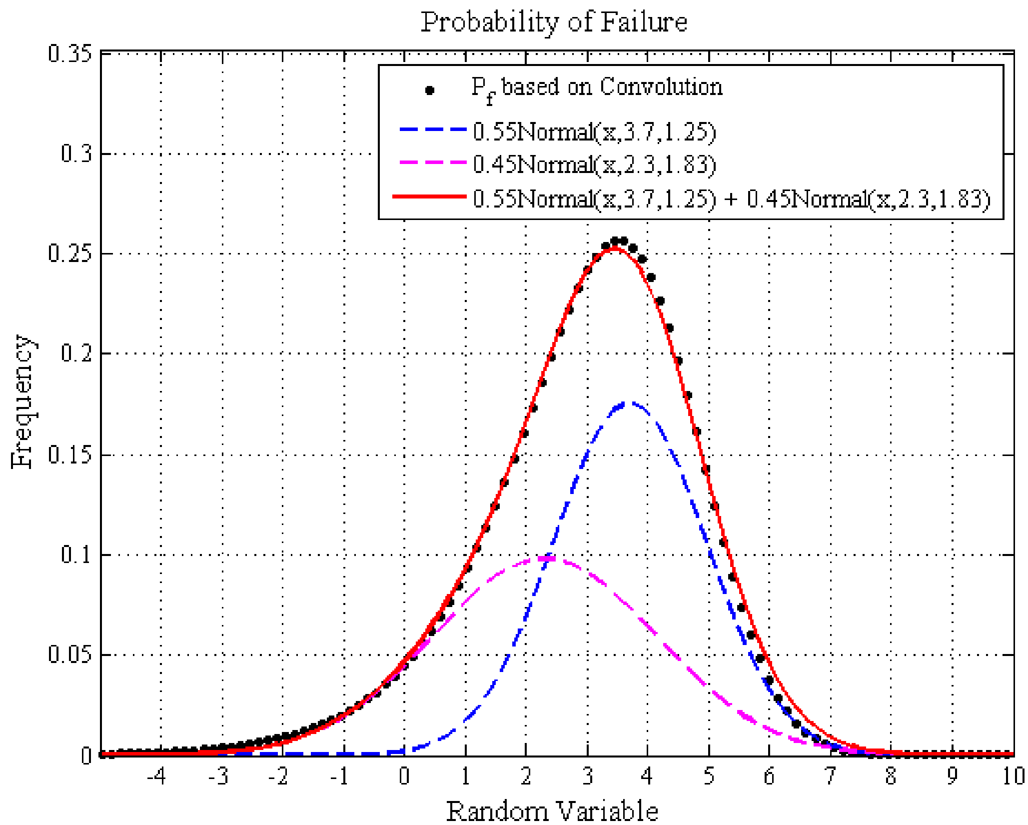

Figure 1.7. Curve Fitting Over the Probability of Failure of while R=$\boldsymbol{Normal(x,5,1)}$ and $\boldsymbol{Q = Gamma(x,2,1)}$

Eventually, the probability of failure and the reliability index is computed. The reliability index is calculated based on summation of the reliability index of two normal distribution as follows:

$$f_X(x) \approx 0.55Normal(x, 3.7, 1.25) + 0.45Normal(x, 2.3, 1.83)$$

$$P_f = 0.056$$

$$\beta = a_1\beta_1 + a_2\beta_2 = 0.55\left(\frac{3.7}{1.25}\right) + 0.45\left(\frac{2.3}{1.83}\right) = 2.19$$

1.4.Conclusion

The reliability index was defined based on the normal distribution. However, if the distribution of the probability of failure does not follow the normal distribution, there is a need to define a new equation to determine the reliability index for non-normal distributions. In this research, by using the convolution theory, a new methodology was introduced to determine the probability of failure for non-normal distribution (Equation 1.15). Furthermore, a closed-form formula was introduced to compute the reliability index. The proposed approach was verified by two examples. It was observed, although that the proposed formula conclude to the exact result, because of the need for the advance mathematical knowledge, this approach is not convenient for engineering applications. Therefore, this paper introduced a simplified method to calculate the reliability index. The simplified method was established based on the fitting curve over the distribution of the probability of failure. The fitting curve was generated with regard to the series of the Gaussian functions, accordingly, the probability of the failure and reliability index was proposed in Equation 1.16 and Equation 1.18.

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2. Target Reliability for Ultimate Limit State in Bridges

Abstract II

The target reliability is a design constraint that assures the required safety level of structures. Derivation of the target reliability level is a complicated and challenging task. To determine the target reliability, two main approaches are considered. The first approach is based on engineering judgment with respect to past observations of structural failure, while the second method is based on optimization theory. The objective of the optimization approach is to find the target reliability based on the cost-failure function. In the past, structural design codes implicitly assumed target reliability that was based on past experiences. The optimization approach requires consideration of the structural cost and the lifecycle, which is the expected design life of the structure.

This paper establishes the relationship between the cost function and reliability index. Then, by using the reliability analysis, the probability of failure of steel girder bridges is considered for the strength limit state. Eventually, for various lifecycle (expected design life of bridges) the minimum of the structural cost (construction and failure costs) is considered and, the target reliability is determined. Furthermore, the contour concept is used to find the relationship between the structural importance factor and reliability.
***Keywords**: Target reliability, Structural cost, Probability failure Function, Limit state function.*

2.1. Introduction

The fundamental characteristics of failure mechanisms in engineering systems were studied by Dasgupta and Pecht (1991). These mechanisms are affected by material defects, assembly damages, and storage conditions. Stress-strength, damage-endurance, challenge-response, and tolerance-requirement are the four conceptual failure models. Considering the nature of stresses, on the other hand, the failure mechanisms can be grouped into mechanical, thermal, electrical, radiation, and chemical failures. They concluded that geometry and damage characteristics of materials were defined by production parameters and environmental consideration impact on stresses. This study would be useful for consideration of the structural failure modes to determine the limit state function and compute the reliability level of the structure based on the assumed limit state.

In general, Lee and et al. (2012) categorized the methods of the target reliability evaluation into two main classes as follows

A) Methods without economical considerations

1- Method based on past accident statistics

2- Method based on comparison with other disasters (Hoshitani and Ishii, 1986)

3- Method based on comparison with a deterministic design method (Nagao et al., 2005, S Enevoldsen and Sørensen, 1994)

B) Methods with economical consideration

4- Method based on analysis of the investment cost required to avoid causalities. (Losada and Benedicto, 2005; Trbojevic, 2009)

5- Method based on minimization of the expected total lifetime cost (Enevoldsen and Sørensen 1994; Suh et al., 2010)

6- Method based on the benefit-cost analysis (Rackwitz, 2000; Rackwitz and Joanni, 2009)

A study conducted by the American Bureau of Shipping (2000) represented a general risk-based methodology to identify important limit state functions and target reliabilities for novel structures considering the failure modes and their consequences. The existing design standards for novel structures do not provide a rational safety level of the structure. In order to determine what is the reliability level of the structures that are designed using the current design standards, a survey questionnaire was distributed among experts.

Wen (2001) considered the uncertainty involved in the loading process and structural response for buildings, in the design of new structures and evaluation of existing ones. He calculated the safety leve using a reliability-based approach. He tried to minimize the expected lifecycle cost as it corresponds to the target reliability.

Mori and Ellingwood (1992) used importance sampling, as a Monte Carlo method, to calculate the time-dependent reliability of a deteriorating reinforced concrete structure. They found that the standard deviation and mean value of the load and resistance parameters are important sampling variables, but deterioration can be treated as a deterministic parameter because of the second-order effect on the structural reliability.

Mori and Ellingwood (1994) combined a crack growth model in concrete with time-dependent reliability to evaluate the strength deterioration of the considered concrete structures. Using experimental data and the proposed approach, a strategy was developed to optimize the cost of inspection/maintenance. A significant computational effort was made to solve the non-linear optimization problem and obtain the optimum inspection and repair cost while keeping the reliability above the target value.

Katsuki and Katade (2009) presented a new method to determine the target reliability of structures. They calculated the reliability of the structures based on the probability of failure and the structural cost.

In an attempt to minimize the related cost of bridge foundations, Huaco et al. (2012) defined the target levels of reliability for strength and serviceability limit states. In doing so, they took into account both socially acceptable risk and economic concerns. Frequency of failure and consequences of failure can determine the socially acceptable risk through FN curves (F= frequency of failure, N=number of lives lost).

Economic concerns, on the other hand involves the lifecycle cost of a bridge depending on reliability of the foundation and consequences of failure. They provided procedure for minimization of the bridge cost for a preselected reliability level.

This paper establishes the new procedure to determine the target reliability with consideration of the maintenance cost. Also, the relationship between the cost function and reliability index is modified. Then, based on the minimization of the structural cost the target reliability is computed. Furthermore, the structural importance factor is defined based on the contour concept. Eventually, as practical example, the target reliability of the steel girder bridges is determined for various lifecycle.

2.1.1. Limit State Function

Structures can be defied in two states: safe and unsafe. The boundary between these two states is represented by a limit state function (g). The limit state function is usually expressed in the form of an equation (Nowak and Collins (2013), and Retief and Dunaiski (2009)).

$$g(X_1, .., X_n) \tag{2.1}$$

where X_i are state variables represent load and resistance parameters. Usually, $g(X_1, .., X_n) > 0$ represents safe structures and $g(X_1, .., X_n) < 0$ represents the unsafe structures.

For instance, when considering a steel beam the unsafe region can be defined by reaching to plastic hinge. Therefore, each failure mode of the structure is associated with limit state functions. Then, based on the failure mode, it is possible to specify the limit state function. In structural engineering, the simplest form of limit state function is

$$g = R - Q \tag{2.2}$$

where,

R = random variable of resistance
Q = random variable of load

Fig. 2.1 shows the safe domain and failure domain for the limit state function in Equation 2.1.

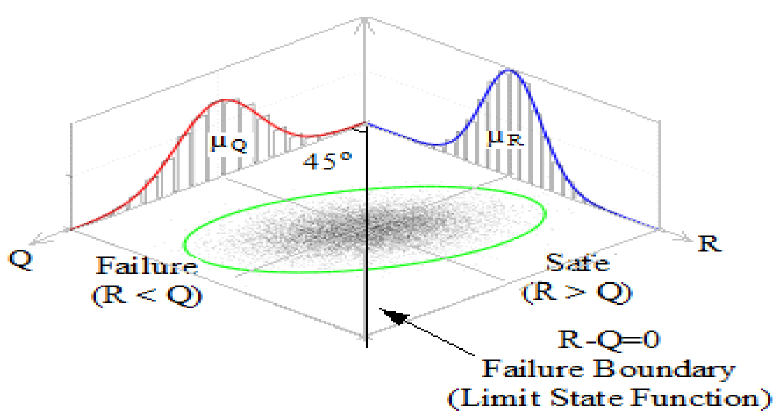

Figure 2.1. Safe Domain and Failure Domain in a Two-Dimensional State Space (Wikipedia-Joint Distribution Function by IkamusumeFan (2014))

2.1.2. Objective Function to Determine the Target Reliability by Lind and Davenport

Target reliability can be determined by

1- Minimization of the total expected cost

2- Maximization of utilities

3- Minimization of human casualties and environmental damage

In all optimization problems, it is necessary to establish the objective function. The objective function in this study is a cost function. Based on the study by Lind and Davenport (1972), the cost function can be defined in terms of the initial cost and the failure cost, which is related to the probability of failure. Lind and Davenport proposed the following model for the objective function.

$$C_T = C_I + C_F\{P_f\} \tag{2.3}$$

Basically, the failure cost can be categorized into the costs at the moment of the failure (*General Cost*) and the cost of the consequences of the failure (*Operational Cost*). Also both mentioned costs, can be divided into economic cost and social and environmental cost (Losada and Benedicto 2005).

In this research, the maintenance cost (C_M) is added to the total cost. Maintenance is required when the structure is exposed to expected or unexpected severe conditions that reduce the structural capacity or functionality. Expected causes are related to material deterioration or reduction of capacity due to severe conditions such as corrosion, fatigue or vibrations. In other words, the main reason for maintenance is to return the structural performance capacity, to its original condition. Therefore, this research proposes the following object function to calculate the target reliability.

$$C_T = C_I + \sum_{i=1}^{n}(C_M)_i\{P_d\}_i + C_F\{P_f\} \tag{2.4}$$

where, $(C_M)_i =$ maintenance cost and $\{P_d\}_i$ is the probability of the structural deterioration at the ith sequences out of the n required maintenance periods during the structural lifecycle.

2.2. Proposed Objective Function to Calculate the Target Reliability

The target reliability is a limit of the safety margin of the structure that indicates the minimum structural cost. The objective function consists of several parameters, such as an initial cost, failure cost and maintenance cost. The initial cost can be directly determined based on the construction cost. The maintenance and failure costs, however, depend on probabilities of maintenance and failure, respectively, and those costs can be considered as an expected cost value. Moreover, during the lifecycle of the structure, failure cost and maintenance cost are changed based on the inflation rate. The objective function is the time-dependent function, therefore, the maximum lifecycle of the structure can be determined. Hence, in this research the double-optimization equation is proposed as an objective function to compute the target reliability.

$$\underset{t}{Max}\,\underset{\beta}{min}\left\{C_I(\beta)+\sum_{i=1}^{n}\big(C_M(\beta,t)\big)_i\{P_d(\beta,t)\}_i+C_F(\beta,t)\big\{P_f(\beta,t)\big\}\right\} \quad (2.5)$$

where β represents the reliability index. If the probability of failure and maintenance behave as normal distributions, the relationship between the probability of failure and the reliability index is expressed as:

$$\text{failure: } \beta=-{\Phi_f}^{-1}(P_f)\ \rightarrow\ P_f=-\Phi_f(\beta) \quad (2.6)$$

$$\text{Maintenance: } \beta=-{\Phi_d}^{-1}(P_d)\ \rightarrow\ P_d=-\Phi_d(\beta) \quad (2.7)$$

where,

$\Phi_f(.)$ = cumulative distribution of failure,
$\Phi_d(.)$ = cumulative distribution of maintenance,
${\Phi_f}^{-1}(.)$ =inverse of cumulative distribution of standard normal failure, and
${\Phi_d}^{-1}(.)$ = inverse of cumulative distribution of standard normal maintenance.

2.2.1. Initial Cost Function

It is clear that by increasing the reliability level of the structure, the construction (initial) cost will be increased. In 1972 Lind and Davenport proposed a linear relationship between reliability index and initial cost.

$$C_I=[a+b\beta] \quad (2.8)$$

However, in this research the relationship between initial cost and of the reliability index is explicitly estimated. As a creative idea, in order to determine the initial cost function with respect to the reliability index, in this research several structures are considered with different levels of reliability. To determine the initial cost.

2.2.2. Maintenance Cost Function

Estimation of the maintenance cost depends on the structural material, its application, the load condition, the environmental conditions around the structure, and the method of repair and maintenance. However, from a simplification point of view, it can be assumed that the maintenance cost merely refers to the general economic costs. Hence, it is possible to consider the maintenance cost as a fraction of the initial cost. However, in order to estimate the maintenance cost at maintenance time, t_m, it is necessary to consider the inflation rate. Accordingly, the following function is proposed to calculate the maintenance cost of the structure.

$$C_M=m[a+b\beta(t_m)](1+i)^{t_m} \quad (2.9)$$

where,

$m = \frac{C_M}{C_I}$ = ratio of the maintenance cost to the initial cost

2.2.3. Failure Cost Function

As mentioned earlier, approximation of the failure cost depends on general costs (economic, social and environmental cost) and operational costs (economic, social and environmental cost). The general economic cost represents the cost resulting from the reestablishment of the structure at the time of the failure, t_{end}; therefore, the general economic cost can be considered as an initial cost with respect to the inflation rate. Nevertheless, consideration of the social and environmental cost is very complicated, and sometimes it seems to be impossible to predict.

However, it is feasible to assume that all side-effect costs can relatively depend on the safety level of the structure. If the structure was designed in a vulnerable social and environmental condition, it is rational to consider a higher safety level for that structure. Therefore, it can be possible to propose that the failure cost relatively depends on the reliability index of the structure. Accordingly, in order to model the failure cost function, this research proposes the following failure cost function.

$$C_F = n[a + b\beta(t_{end})](1 + i)^{t_{end}} \tag{2.10}$$

where,

$n = \frac{C_F}{C_I}$ = ratio of the failure cost to the initial cost

After proposing the cost functions, the objective function can be expressed as follows

$$\underset{t}{Max}\ \underset{\beta}{min}\left\{[a + b\beta(t_o)] + \sum_{i=1}^{n}(m[a + b\beta(t_m)](1 + i)^{t_m})_i\{P_d(\beta, t_m))\}_i \right.$$
$$\left. + n[a + b\beta(t)](1 + i)^t\{P_f(\beta, t)\}\right\} \tag{2.11}$$

where,

t =time (year),
t_o = construction time (year),
t_m = maintenance time (year),
i = inflation rate,
$\beta(t)$ = time-dependent reliability,
m & n = constant parameters.

However, minimization of the reliability index with respect to the time variant is a complicated task, and it requires stochastic analysis. To avoid complex analysis, this research proposes a simple way to determine the target reliability based on the maximum lifecycle by discretization of the objective function over the lifecycle,

$$\underset{t}{Max}\begin{cases} t = t_1 \rightarrow \underset{\beta}{min}\left\{C_I(\beta) + \sum_{i=1}^{n}\big(C_M(\beta, t_i)\big)_i\{P_d(\beta, t_i)\}_i + C_F(\beta, t_1)\{P_f(\beta, t_1)\}\right\} \\ t = t_2 \rightarrow \underset{\beta}{min}\left\{C_I(\beta) + \sum_{i=1}^{n}\big(C_M(\beta, t_i)\big)_i\{P_d(\beta, t_i)\}_i + C_F(\beta, t_2)\{P_f(\beta, t_2)\}\right\} \\ \vdots \\ t = t_n \rightarrow \underset{\beta}{min}\left\{C_I(\beta) + \sum_{i=1}^{n}\big(C_M(\beta, t_i)\big)_i\{P_d(\beta, t_i)\}_i + C_F(\beta, t_n)\{P_f(\beta, t_n)\}\right\} \end{cases} \tag{2.12}$$

As seen in system Equation 2.12, at any intended lifecycle the target reliability can be computed. Then based on the maximization point of view and the structure (bridge) owner's decision, the most appropriate lifecycle can be determined.

2.3. Relationship between the Probability of Failure and Reliability index

The probability of failure (P_f) is an important term in the objective function. The probability of failure is the function that illustrates the distribution of the failure function. In fact, this function is a time-dependent one. To discover the probability of failure, it is necessary to establish the limit state function, and then, by consideration of the state variables the probability of failure can be obtained. If the probability of failure behaves as a normal probability distribution function, the relation between the reliability index and the probability of failure is defined based on the inverse function of the cumulative function of the normal distribution (Basler (1961) and Cornell 1969).

$$\beta = -\Phi^{-1}(P_f) \tag{2.13}$$

In the literatures review (Nowak and Collins (2013)), the relationship between reliability index and probability of failure was simplified as an exponential function. However, the exponential assumption merely works in a deceptive way.

The common used simplified function has been introduced in form of $c * e^{\left(\frac{-\beta}{d}\right)}$, where the best fitted coefficients are $c \approx \frac{1}{2}, d \approx \frac{1}{\sqrt{2}}$. This exponential approximation works decently as long as the range of β is restricted to less than or equal to 2, and, beyond that range, the differentiation is extremely revealed. This error cannot be observed in linear coordinates; however, plotting on a semi-logarithmic coordinates shows this considerable difference.

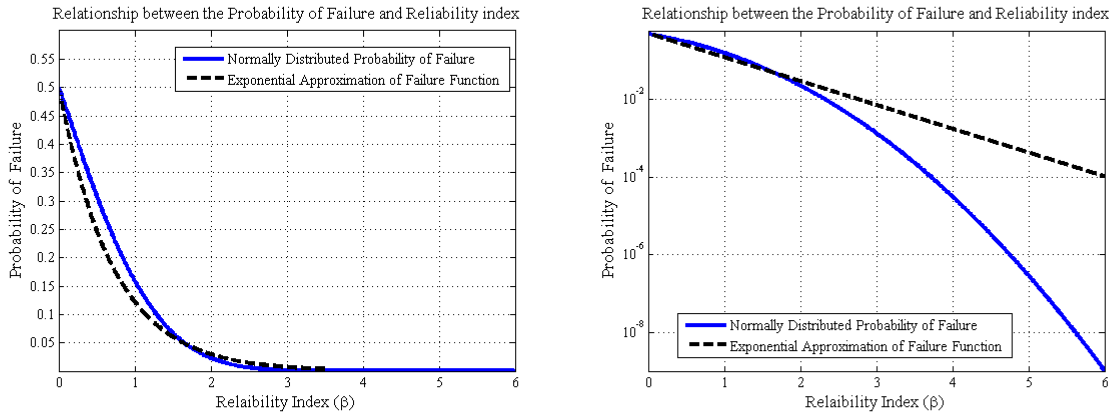

Figure 2.2. Comparison between the Normal Distributed Probability of Failure Function and the Proposed Approximation in the Past (which was cited by Nowak and Collins (2013)), in Linear Scale (Left Side) and Log Scale (Right Side)

As can be seen, the proposed relationship between reliability index and probability of failure is not a proper model. Therefore, there is a need to figure out the more exact relationship between reliability index and probability of failure. In this paper it is assumed that probability of failure behaves as a normal distribution, based on the mathematics fitting curve, a new function is proposed. Equation 2.14 shows a proposed relationship between reliability index and probability failure of the normal distribution function.

$$P_f = 0.5 * exp\left(\left[\frac{-\beta}{1.9}\right]^2\right) \quad (2.14)$$

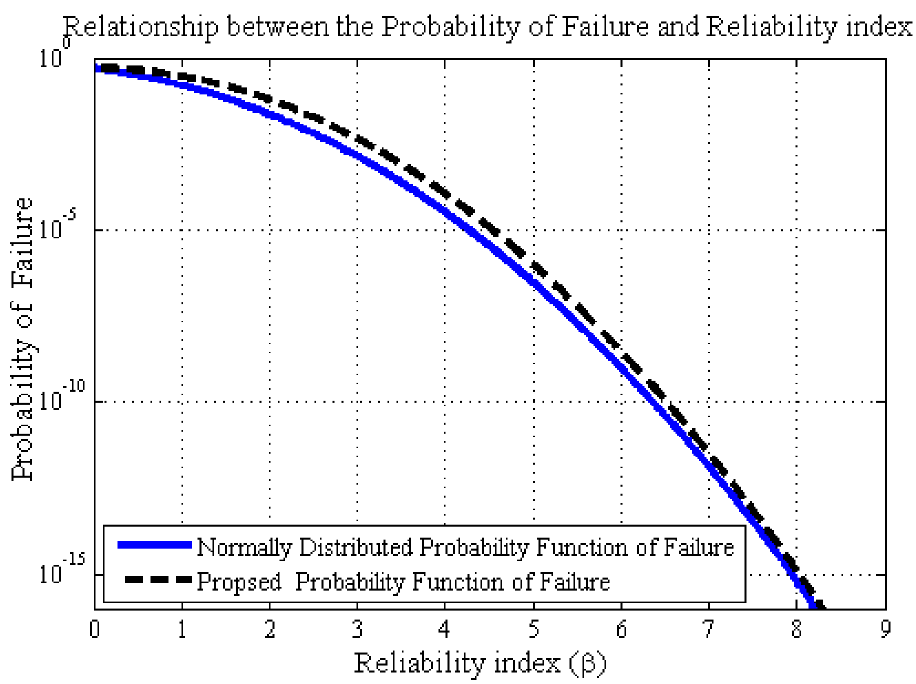

Figure 2.3. Comparison between the Normal Distributed Probability of Failure Function and the Proposed Failure Function in This research, in Semi-Log Scale

2.4. Structural Importance Factor versus the Reliability Index

In American Society of Civil Engineering (ASCE 7-10) Minimum Design Loads for Buildings and Other Structures (2010), the structural importance factor was considered as a modification factor to the applied load. Nevertheless, it is evident that the impact of the structural importance factor refers to the failure cost. Therefore, in this research, the structural importance factor, S_I, is defined as a modification factor that represents the consequences of structural failure. Generally, if the consequences of structural failure are greater than an ordinary structure, there is a need to consider a higher safety. Based on this definition, the structural importance factor can be considered as a modification factor to the failure cost. To demonstrate the relationship between the structural importance factor and the target reliability, the contour concept would be an appropriate approach.

A contour plot shows the objective function in shape of the isopleths (isolines), where every single line refers to the particular objective function with a consideration of the certain structural importance factor. For instance, Figure 2.4 displays the contour of the target reliability with respect to the structural importance factor for a bridge with 40 ft. length.

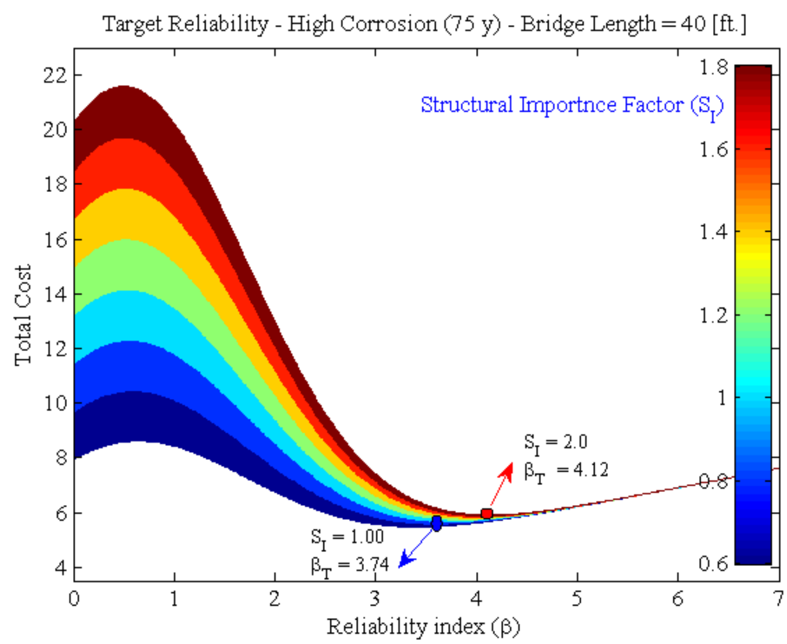

Figure 2.4. Variation of the Target Reliability with Respect to the Structural Importance Factor

As can be seen, and as was expected, by consideration of the greater structural importance factor for a structure, it is required to consider the higher safety level for that structure.

2.5. Target Reliability for Steel Girder Bridges based on the Ultimate Limit State

In this section, the target reliability of the non-composite steel girders is determined based on the ultimate limit state. Based on the design recommendations, the flexural capacity is the most important design criteria for general bridges. Therefore, this research considers the moment carrying capacity of the steel girders as the ultimate limit sate of the bridges. Nowak and Hong (1991), and Nowak and et.al (1987) used the following limit state function to determine the reliability index of the bridges

$$g = F_y Z - (M_D + M_W + M_L) \quad (2.15)$$

where,

Z = plastic modulus of the section
F_y = yield stress
M_D = moment due to the dead load
M_W = moment due to pavement wearing
M_L = moment due to HL-93

Based on Nowak's studies (1993 and 1999), the design formula for a bridge subjected to vehicular live load (HL-93) is

$$F_{y_n} Z_n > \left(\gamma_D M_{D_n} + \gamma_W M_{W_n} + \gamma_L M_{L_n}\right) \quad (2.16)$$

where,

Z_n = nominal plastic modulus of the section
F_{y_n} = nominal yield stress
γ_D = dead Load Factor
γ_W = surface wearing Load Factor
γ_L = live Load Factor
M_{D_n} = nominal moment due to the dead load
M_{W_n} = nominal moment due to pavement wearing
M_{L_n} = nominal moment due to live load

2.5.1. Initial Cost Study for Non-Composite Steel Girders

The initial cost can be expressed in terms of the reliability index. The initial cost represents the total required cost to build a structure. To determine the relationship between the initial cost and the reliability index, it is required to design the structure based on the assumed reliability index and then calculate the cost of the construction. To attain this goal, the best way is to virtually design the structure with the different levels of reliability to determine the construction cost, and then the relationship between the cost and reliability index can be demonstrated.

One of the major purposes of this research is to compute the target reliability for bridges. Here, the non-composite steel girder bridges are investigated and the initial cost functions are computed.

As declared, this research aims to determine the target reliability of non-composite steel girder bridges for the ultimate limit state due to the flexural strength of the girders. Therefore, in this research, the initial cost refers to the cost of girders. The costs of the girders were estimated based on the list price of Nucor-Yamato Steel Company (online price). From the preliminary study, it was observed that the initial cost is linearly related to the reliability index. Figures 2.5 to 2.8 show the relationship between the reliability index and the initial cost of the non-composite steel girders with respect to the span lengths.

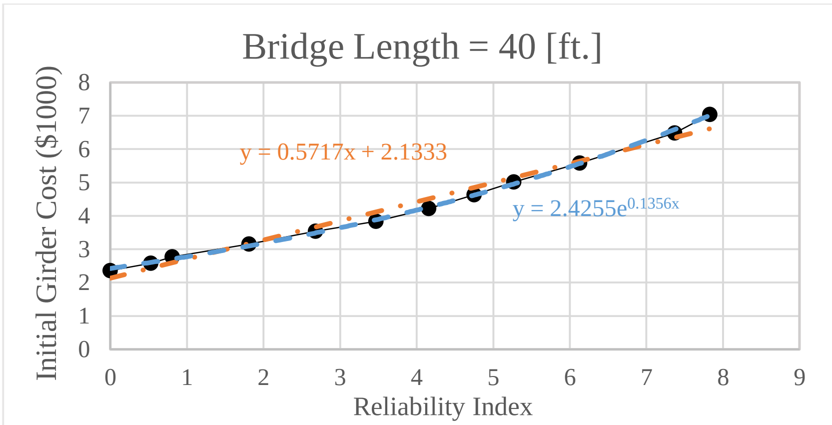

Figure 2.5. Initial Cost vs. Reliability Index, Bridge Length=40 [ft.]

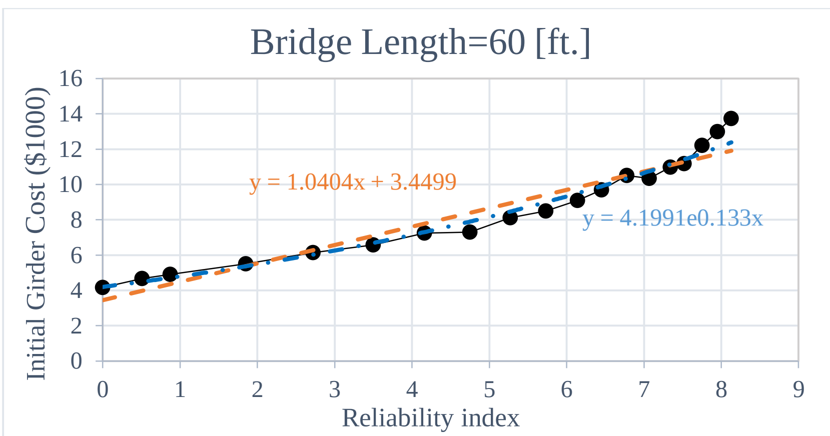

Figure 2.6. Initial Cost vs. Reliability Index, Bridge Length=60 [ft.]

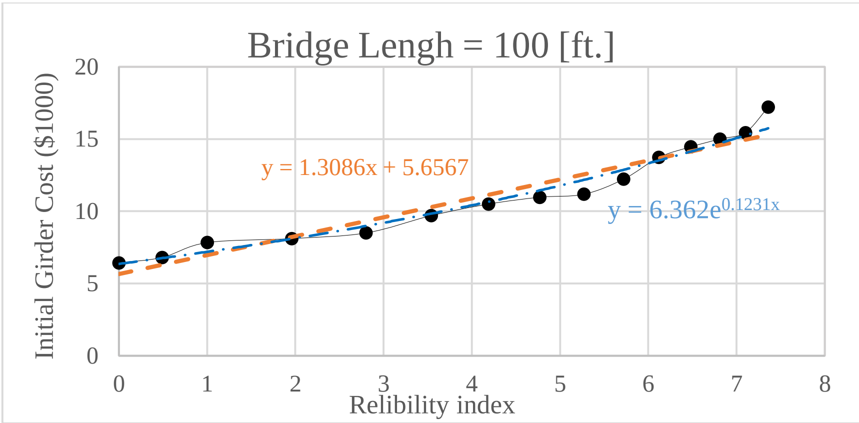

Figure 2.7. Initial Cost vs. Reliability Index, Bridge Length=100 [ft.]

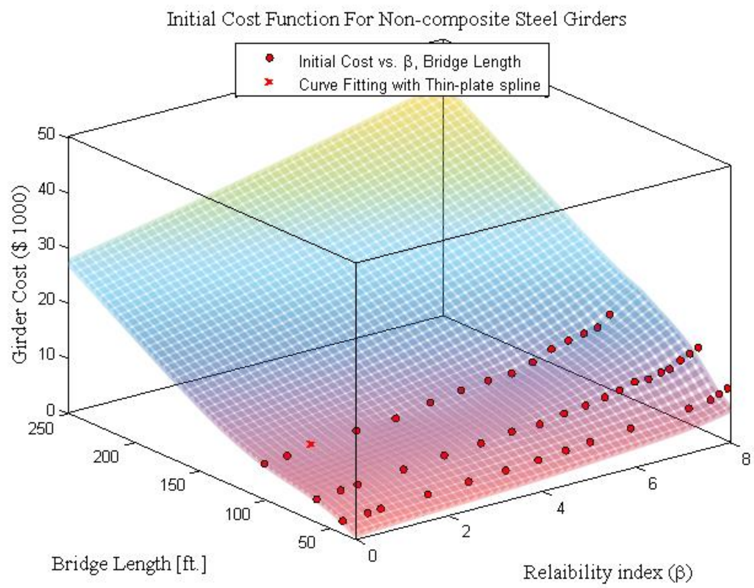

Figure 2.8. Initial Cost vs. Reliability Index with Respect to Different Bridge Lengths

To model the cost-reliability index relationship, linear approximations and exponential approximations are tabulated in Table 2.1. However, in order to avoid confusion from complexity, the linear approximation is more convenient. Also as discerned in Figures 2.5 to 2.8, in an expected range of the target reliability (3-4) for the aforementioned limit state (the ultimate limit of flexure), the cost and reliability index relation approximately behaves as a linear function.

Table 2.1. Initial Cost Model for a Steel Girder Bridge

Initial Cost	$[a + b\beta]$		$[a \times exp(b\beta)]$	
Girder length [ft.]	a	b	a	b
L=40	2.13	0.57	2.43	0.136
L=60	3.45	04	4.20	0.133
L=100	5.66	31	6.36	0.123

2.5.2. Probability of Failure with respect to the Lifecycle

The probability of failure/damage is a time-dependent function; in order to consider the variation of the probability of failure over time, it is essential to utilize stochastic analysis. Applying continuous stochastic analysis is a complicated and time-consuming effort; therefore, the discrete stochastic process is introduced by authors.

The statistical parameters of the load and resistance change with time. As long as live load is crossing the bridge over the time with respect to the particular Average Daily Truck Traffic (ADTT), the bias factor of the live load moment and its variation are changing. According to the study by Rakoczy (2011), the statistical parameters of the load were evaluated as follows, with consideration of ADTT equal to 5,000.

Table 2.2. Statistical Parameters of the Live Load Moment on a Bridge, ADTT=5,000, Rakoczy (2011)

ADTT=5,000 Bridge	Length =40 [ft.]		Length =60 [ft.]		Length =100 [ft.]	
	μ	V	μ	V	μ	V
1 Day	0.79	0.19	0.79	0.18	0.78	0.17
2 weeks	0.92	0.19	0.96	0.17	0.97	0.17
1 Month	0.98	0.16	0.99	0.17	1.04	0.17
2 Months	1.03	0.15	1.04	0.17	1.12	0.11
6 Months	1.06	0.15	1.07	0.17	1.14	0.10
1 Year	1.10	0.14	1.11	0.14	1.17	0.09
5 Years	1.19	0.12	1.18	0.12	1.20	0.08
50 Years	1.24	0.08	1.22	0.11	1.23	0.07
75 Years	1.25	0.07	1.24	0.10	1.25	0.07
100 Years	1.26	0.07	1.25	0.10	1.26	0.07

Resistance is also a time-dependent parameter. In this case study, because a steel girder is considered, the resistance of the steel girder is mainly decreased over time due to environmental influences (corrosion) and load variations (fatigue). In the current research, corrosion deterioration is investigated, however, fatigue deterioration and the combination of both are postponed to a future study.

In the present limit state (the ultimate limit state due to bending failure at the middle of the girder), if the girder performs in a linear region, the resistance can be shown as follows

$$R(t) = F_y(t)Z(t) \qquad (2.17)$$

where,

$R(t)$ = time-dependent resistance
$Z(t)$ = time-dependent plastic modulus of section
$F_y(t)$ = time-dependent yield stress, where here it is not considered

Corrosion of steel is an electro-chemical process that leads to the deterioration of the steel (Park 1999), and the steel deterioration affects the geometric properties of the steel cross-section. There are several environmental parameters that influence the intensity of the corrosion rate, among which the percentage of ambient humidity plays a pivotal role. Several models have been proposed to predict the corrosion behavior for steel girders; in 1984, Albrecht and Naeemi proposed a formula to consider the average corrosion penetration rate:

$$C = A.t^{B} \tag{2.18}$$

where,

$C =$ average corrosion penetration rate (micrometer)
$A\ and\ B =$ constant parameters determined by experimental analysis
$t =$ time variable (year)

Average values for corrosion parameters $A\ and\ B$, for carbon and weathering steel, are related to the location of the girders. Albrecht and Naeemi categorized the corrosion in three levels: 1- Rural, 2- Urban, 3- Marine locations. In 1999, Park classified the corrosion into high, medium and low levels as follows:

Table 2.3. Corrosion Penetration Based on the Park Model (1999)

Year	Low corrosion (in)	Medium corrosion (in)	High corrosion (in)
0	0	0	0
10	0	0.001	0.002
20	0.002	0.003	0.009
30	0.004	0.007	0.02
40	0.006	0.014	0.039
50	0.01	0.025	0.058
60	0.015	0.031	0.073
70	0.018	0.037	0.081
80	0.02	0.041	0.091
90	0.021	0.042	0.098
100	0.022	0.044	0.102

Figure 2.9 shows the penetration amount of corrosion in steel girders based on the Park (1999) categories; in this research the Park model is used to consider the corrosion effect on the resistance reduction.

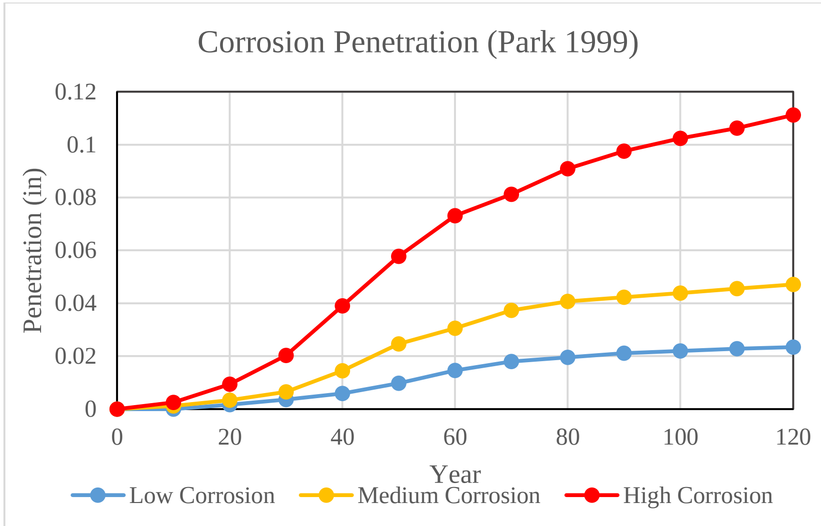

Figure 2.9. Steel Girder Corrosion Model Based on the Park Classification (1999)

According to the observations by Kayser and Nowak (1989), for the steel girder, at middle of the span, the bottom flange and one fourth of the lower part of the web are mainly exposed to the corrosion deterioration. Sharifi and Rahgozar in 2010 proposed the time-dependent plastic modulus $Z(t)$ based on the uniform and variant models; for the uniform model, they proposed the following formula:

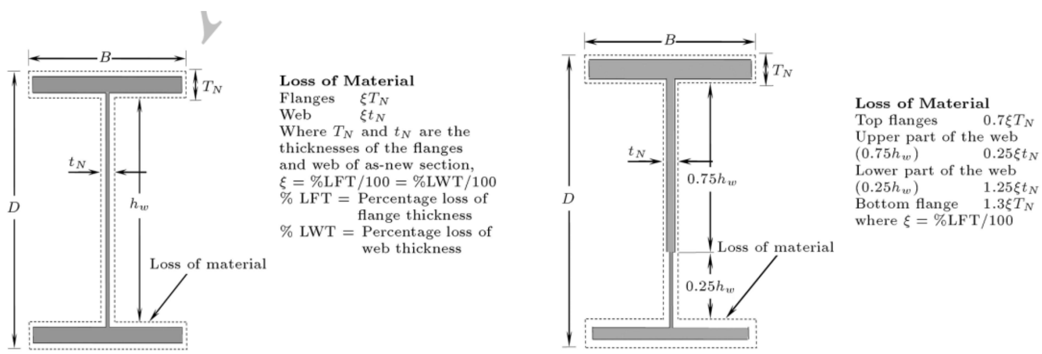

Figure 2.10. Corrosion model of the Steel Girder, Sharifi and Rahgozar (2010)

$$Z_{XC} \approx Z_{XN}(1-\xi), where\ \xi = \xi_f = 2\xi_w \quad (2.19)$$

where,

Z_{XC} = plastic modulus of the corroded I section
Z_{XN} = plastic modulus of the new I section
ξ = percentage loss of the thickness
ξ_w = percentage loss of the thickness of the web
ξ_f = percentage loss of the thickness of the flange

For the variant model, Sharifi and Rahgozar (2010) represented their result as follows:

$$Z_{XC} \approx Z_{XN} - \xi \left(Z_{XN} - T_N \frac{h^2{}_w}{8}\right) where\ \xi = \xi_f = \xi_w \tag{2.20}$$

where,

T_N = web thickness
h_w = web height

As seen, the loss rate of the plastic modulus depends on the girder section, and the girder section is related to the reliability index; therefore, probability of failure is also related to the assumed reliability index.

2.5.3. Target Reliability Analysis

In this section, target reliability is computed based on proposed objective function; furthermore, the structural importance factor (S_I) is considered with regard to the following equation.

$$\underset{t}{Max}\ \underset{\beta}{min} \Big\{[a + b\beta(t_o)] + \textstyle\sum_{i=1}^{n}(m[a + b\beta(t_m)](1 + i)^{t_m})_i\{P_d(\beta, t)\}_i + (S_I).n[a + b\beta(t_{end})](1 + i)^{t_{end}}\{(r_{\beta_{end}})P_f(\beta, initial)\}\Big\} \tag{2.21}$$

where,

t =time (year)
t_o = construction time (year)
t_m = maintenance time (year)
t_{end} = failure time (year)
i = inflation rate
$\beta(t)$ = time-dependent reliability
m & n = constant parameters
$(r_{\beta_{end}}) = \frac{\{P_f(\beta, initial)\}}{\{P_f(\beta, end)\}}$

As an example, for a non-composite steel girder bridge with a length of 100 ft. and economical inflation rate $i = 5\%$, the ratio of the maintenance cost to the initial cost can be determined. According to National Cooperative Highway Research Program (NCHRP) Report 483 for steel girder bridges (assuming the maintenance cost consisted of inspections and repaints of the bridges every five years in high corrosion conditions, 10 years in medium corrosion conditions and 15 years in low corrosion conditions), the maintenance cost was calculated.

Obtaining the failure cost is a very complicated task that requires tedious field study and cost estimation; because it requires numerous assumptions, the most important parameters are the location and strategic position of the bridge. As described, the failure cost is somehow related to the initial cost; therefore, in this research, the failure cost is considered as a proportion of the initial cost, n.

Several example bridges with lengths of 40, 60, and 100 ft are considered, subject to low, medium, and high corrosion conditions. First, the probability failure functions were evaluated, and then, the target reliability is computed for different lifecycle periods. Moreover, the relationship between the structural importance factor and the required target reliability was demonstrated.

Case Study

The bridge length is 100 ft., and the bridge is exposed to high, medium, and low corrosion conditions. Based on the initial cost analysis, the girder cost function is written as follows

$$C_I = [5.66 + 31\beta]$$

The probability of failure function is a time-dependent function at the specific lifecycle. Indeed, it could be possible to determine the failure probability of the structures with regards to the load and resistance. Since the resistance is a time-dependent parameter, first it is necessary to make a guess about the girder section properties. Based on the preliminary design for bridges with 100 ft. lengths and girder spacing of about 6 ft., the girder section was assumed as a W36x194. Now, with respect to the corrosion model, it is possible to compute the remaining capacity of the section, which tabulates in Table 2.4.

Table 2.4. Remaining Flexural Capacity of W36x194 and Statistical parameters of the Live Load for a bridge with a length of 100 [ft.]

Low corrosion based on the Park model										
Year	10	20	30	40	50	60	70	80	90	100
Remaining resistance %	1	0.999	0.997	0.996	0.993	0.990	0.988	0.987	0.986	0.985
Medium corrosion based on the Park model										
Year	10	20	30	40	50	60	70	80	90	100
Remaining resistance %	0.999	0.998	0.995	0.991	0.983	0.979	0.976	0.973	0.972	0.971
High corrosion based on the Park model										
Year	10	20	30	40	50	60	70	80	90	100
Remaining resistance %	0.998	0.994	0.987	0.974	0.962	0.952	0.946	0.940	0.935	0.932
Statistical parameters of live load										
Year	10	20	30	40	50	60	70	80	90	100
Bias	1.21	1.215	1.22	1.225	1.23	1.24	1.245	1.25	1.255	1.26
Variation	0.08	0.08	0.08	0.08	0.07	0.07	0.07	0.07	0.07	0.07

The probability of failures were calculated using Monte Carlo's simulation, and Table 2.5 tabulates the reliability index and Probability Distribution Functions (PDF) parameters of the probability of failure, for girders that were subjected to high corrosion conditions.

Table 2.5 Normal Distribution of the Probability of failure with High Corrosion for a bridge with a length of 100 [ft.]

Normal Distribution of the Probability of failure with High Corrosion			
year	**mean**	**Standard Variation**	**Reliability index**
0	16834	4725	3.563
10	15564	4696	3.314
20	15304	4625	3.309
30	14918	4602	3.242
40	14289	4566	3.130
50	13727	4451	3.084
60	13151	4429	2.969
70	12839	4412	2.910
80	12502	4380	2.855
90	12220	4367	2.798
100	12052	4349	2.771

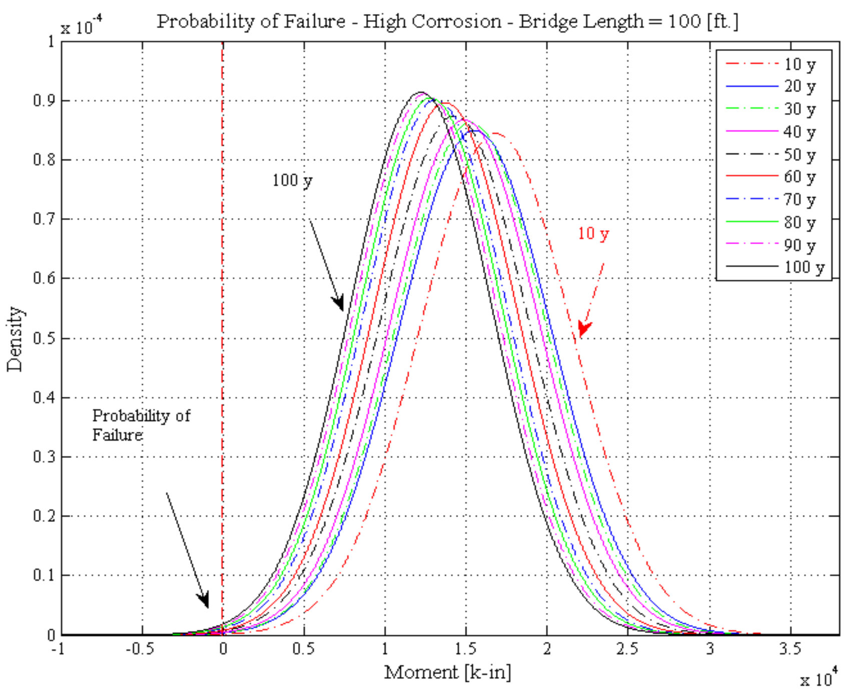

Figure 2.11. PDF of the Probability of Girder Failure due to the Strength Limit State being exposed to High Corrosion Conditions, Bridge Length =100 [ft.]

Figure 2.11 exhibits the probability of failure PDFs for the 100 ft girders that were exposed to high corrosion conditions. Now, it is time to establish the objective function; first, the objective function is broken into several time periods as follows:

$$if\ t_{end}100 \rightarrow \min_{\beta}\left\{[5.66+31\beta]\left((059+(S_I).n(1+0.015)^{100})\frac{1}{2}(r_\beta)exp\left(\left[\frac{-\beta}{1.9}\right]^2\right)\right)\right\}$$

$$if\ t_{end}=75 \rightarrow \min_{\beta}\left\{[5.66+31\beta]\left((0.876+(S_I).n(1+0.015)^{75})\frac{1}{2}(r_\beta)exp\left(\left[\frac{-\beta}{1.9}\right]^2\right)\right)\right\}$$

$$if\ t_{end}=50 \rightarrow \min_{\beta}\left\{[5.66+31\beta]\left((0.370+(S_I).n(1+0.015)^{50})\frac{1}{2}(r_\beta)exp\left(\left[\frac{-\beta}{1.9}\right]^2\right)\right)\right\}$$

$$if\ t_{end}=25 \rightarrow \min_{\beta}\left\{[5.66+31\beta]\left((0.653+(S_I).n(1+0.015)^{25})\frac{1}{2}(r_\beta)exp\left(\left[\frac{-\beta}{1.9}\right]^2\right)\right)\right\}$$

where,

$n = \frac{C_F}{C_I}$,

r_β = modification factor for the probability of failure.

The relationship between the probability of failure and the reliability index was proposed in Equation 2.14. As shown in Figure 2.11, the PDFs are not constant. In the equations above, the modification factor, r_β, is used to make a simplification for other time periods. The modification factor modifies the probability of failure at the failure time with respect to the probability of failure at the initial time. This factor is defined based on the ratio of the reliability index at the initial time to the reliability index at the failure time.

To determine the target reliability of the structure, it is necessary to establish the failure cost function. As mentioned, failure cost depends on several parameters. Therefore, applying failure cost analysis gives us an opportunity to make decisions on target reliability concerns.

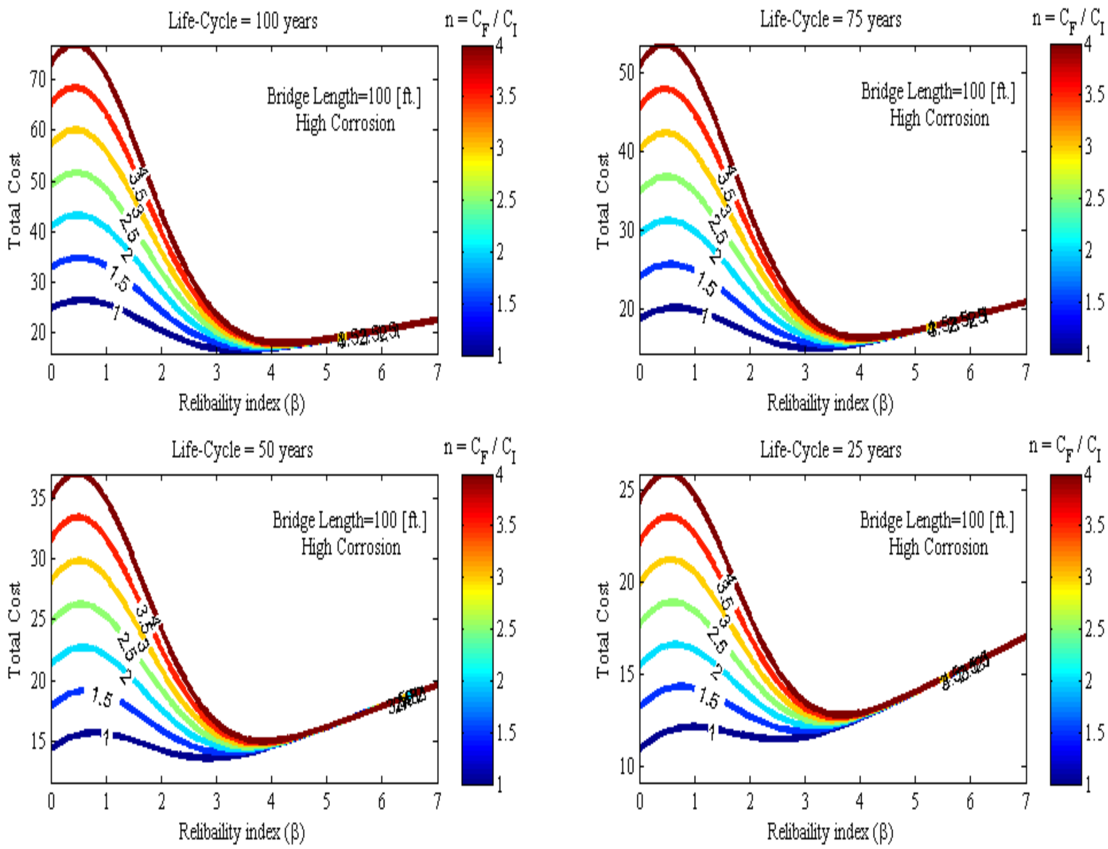

Figure 2.12. Target Reliability for a Bridge with a Length of 100 ft. Exposed to the High Corrosion Conditions Using the Contour of the Cost Ratio, n.

American Association of State Highway and Transportation Officials Load and Resistance Factor Design (AASHTO LRFD 2014) aims to design an ordinary highway bridge for a 75 year lifecycle. Regarding the contour analysis of target reliability, the failure cost is twice of the construction cost ($C_F = \mathbf{2}C_I$). Therefore, it is possible to state that, in AASHTO LRFD, the failure cost is considered as twice the construction cost at the failure time (after 75 years).

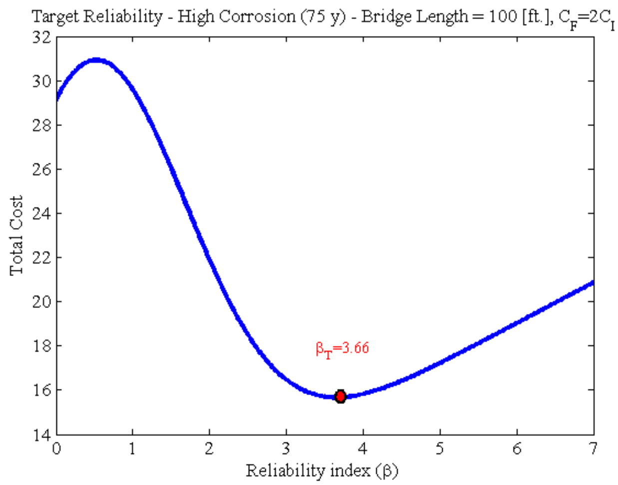

Figure 2.13. Target Reliability for a Bridge with a length of 100 ft., Exposed to High Corrosion Conditions, Assumed $C_F = 2C_I$

Also, with respect to the contour concept, the structural importance factor is determined. This research considers the structural importance factor, S_I, as a modification factor to the failure cost.

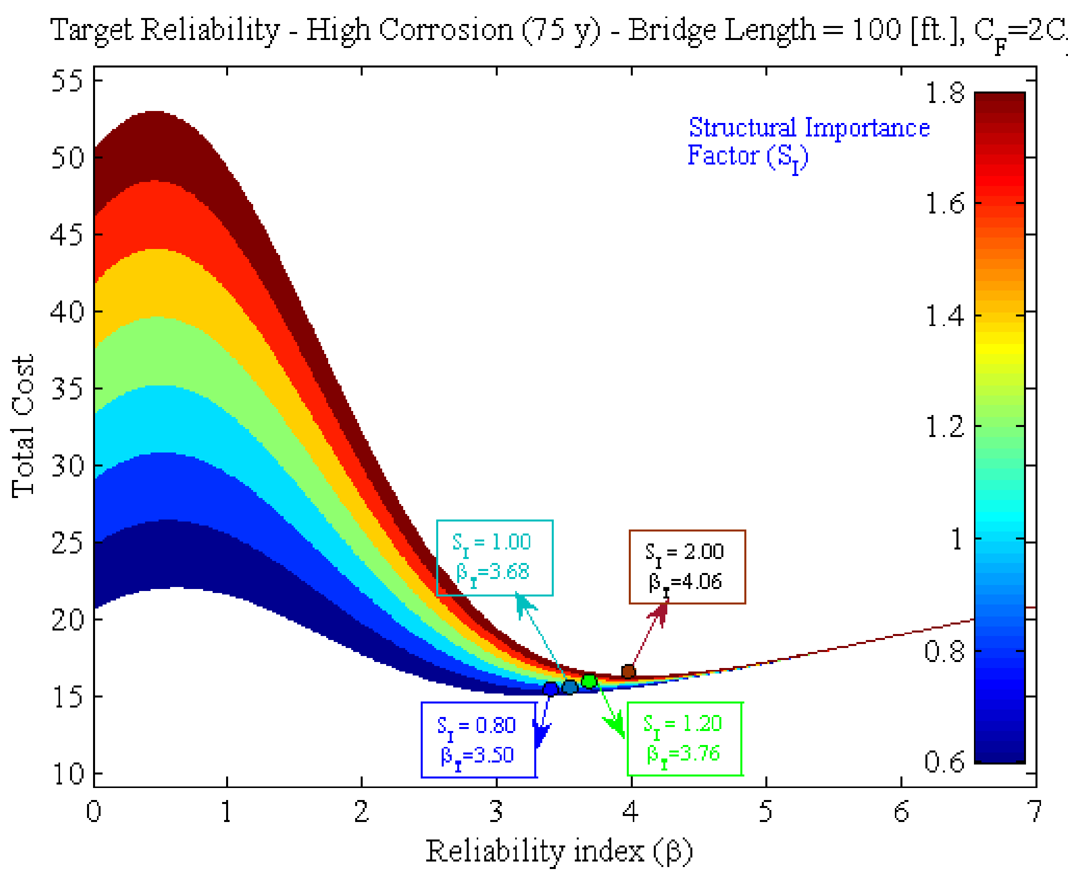

Figure 2.14. Target Reliability of a Bridge with a length of 100 ft., Exposed to High Corrosion Conditions, with Respect to Different Structural Importance Factors, S_I, $C_F = 2C_I$

Table 2.6. Required Target Reliability for Different Structural Importance Factors

Bridge with 100ft. Length, Exposed to High Corrosion Conditions, $C_F = 2C_I$				
Structural Importance Factor S_I	0.8	1.0	2.0	2.0
Target Reliability β_T	3.50	3.68	3.76	4.06

As shown in Figure 2.14 and Table 2.6, the greater structural importance factor requires the higher safety level. Alternatively, by load factor calibration, it would be possible to propose the new load factors for any desired structural importance factor.

The target reliability analyses for low, medium, and high corrosion conditions are presented in Appendix A. Finally, Figure 2.15 shows the discrepancies among the levels of corrosion for bridges with a length of 100 ft.

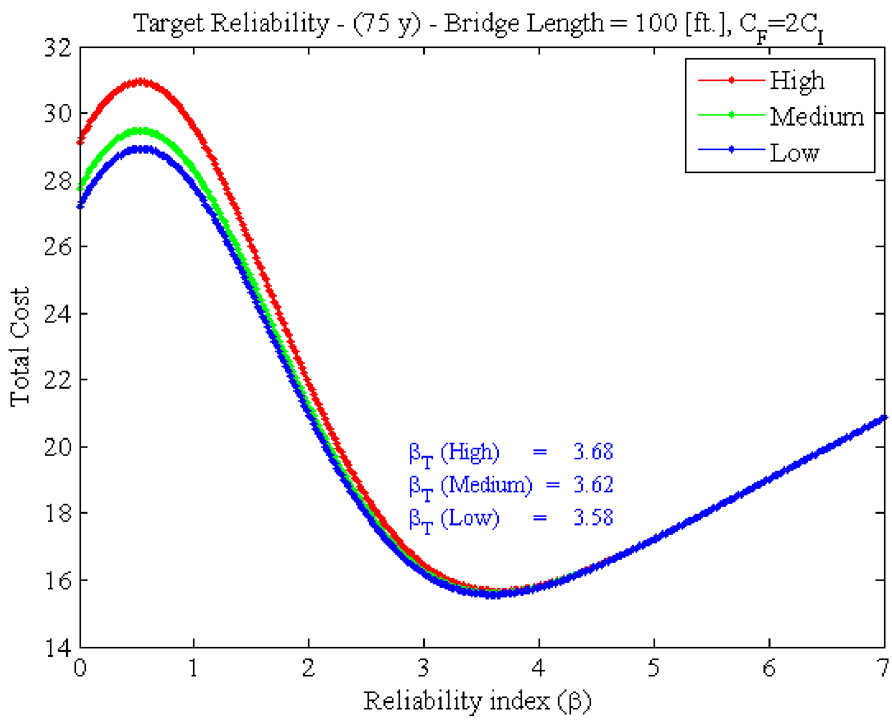

Figure 2.15. Target Reliability of a Bridge with a length of 100 ft., Exposed to high, Medium, and Low Corrosion Conditions, with Respect to Different Structural Importance Factors, S_I, $C_F = 2C_I$

Table 2.7. Required Target Reliability for Different Corrosion Conditions

Bridge with 100ft. Length, Exposed to Different Corrosion Conditions, $C_F = 2C_I$, $S_I = 1.0$			
Corrosion Level	Low	Medium	High
Required Target Reliability β_T	3.58	3.62	3.68

As can be seen, for severe corrosion conditions, it is necessary to consider the higher target reliability values.

2.5.4. Comparison the Target Reliability for Bridges with Different span Lengths

In this section, the result of the target reliability for different span lengths was investigated. Table 2.8 shows the required target reliability for different lengths with regard to the low, medium, and high corrosion conditions. As observed, by increasing the severity of the corrosion condition, a higher value of target reliability is required. It is necessary to mention that the tabulated values in Table 2.8 represent the target reliability for a 75 year lifecycle, and the failure cost is considered as twice as the construction cost.

Table 2.8. Required Target Reliability for Different Bridge Lengths, Regarding Corrosion Conditions

Bridge with 100ft. length, Exposed to the Different Corrosion Conditions, $C_F = \mathbf{2}C_I$, $S_I = 1.0$			
Corrosion Level	Low	Medium	High
Required Target Reliabilityβ_T	3.58	3.62	3.68
Bridge with 60ft. length, Exposed to the Different Corrosion Conditions, $C_F = \mathbf{2}C_I$, $S_I = 1.0$			
Corrosion Level	Low	Medium	High
Required Target Reliabilityβ_T	3.56	3.58	3.62
Bridge with 40ft. length, Exposed to the different Corrosion Conditions, $C_F = \mathbf{2}C_I$, $S_I = 1.0$			
Corrosion Level	Low	Medium	High
Required Target Reliabilityβ_T	3.64	3.66	3.74

Tables 9 to 11 demonstrate the relationship between the required target reliability (β_T) and the structural importance factor (S_I) for different bridge span lengths exposed to the different corrosion conditions.

Table 2.9. Required Target Reliability for Different Bridge Lengths, Regarding Structural Importance Factors, with Respect to High Corrosion Conditions

Bridge Length [ft.]	**Structural Importance Factor S_I**	**0.8**	**1.0**	**1.2**	**2.0**
100	Target Reliability, β_T	3.50	3.68	3.76	4.06
60	Target Reliability, β_T	3.50	3.62	3.76	4.04
40	Target Reliability, β_T	3.60	3.74	3.84	4.12

Table 2.10. Required Target Reliability for Different Bridge Lengths, Regarding Structural Importance Factors, with Respect to Medium Corrosion Conditions

Bridge Length [ft.]	**Structural Importance Factor S_I**	**0.8**	**1.0**	**1.2**	**2.0**
100	Target Reliability, β_T	3.46	3.60	3.72	4.02
60	Target Reliability, β_T	3.44	3.58	3.70	4.00
40	Target Reliability, β_T	3.52	3.66	3.76	4.06

Table 2.11. Required Target Reliability for Different Bridge Lengths, Regarding Structural Importance Factors, with Respect to Low Corrosion Conditions

Bridge Length [ft.]	**Structural Importance Factor S_I**	**0.8**	**1.0**	**1.2**	**2.0**
100	Target Reliability, β_T	3.44	3.58	3.70	4.00
60	Target Reliability, β_T	3.40	3.58	3.68	3.98
40	Target Reliability, β_T	3.48	3.62	3.74	4.04

As was expected, considering a higher value of the structural importance factor demands greater target reliability. It is worth mentioning that Tables 2.9 to 2.11 summarized the relationship between β_T and S_I with respect to the consideration of 75 years as a lifecycle performance of the bridge, while the failure cost is twice the construction cost at the failure occurrence. For example, in high corrosion condition, if S_I is considered about one, there is a bridge failure out of 10,000 ($\beta_T \approx 3.7$) bridges and if S_I is assumed about two, there is possibility of a bridge failure out of 32,500 ($\beta_T \approx 4.1$) bridges, which as economical point of the view this rate of the failure refers to the optimum (target) reliability.

2.6. Conclusions

The main objective of this research was to propose a methodology to determine the target reliability of structures. That methodology was accomplished based on the optimization approach with respect to the minimization of the structural cost at the certain lifecycle. Several new definitions and modifications were considered to create an applicable objective function.

1- A new approach was introduced to estimate the relationship between the initial cost and reliability index of the structure. In this approach, by changing the load factor, different levels of safety were obtained. The relationship between construction costs and the obtained reliability indices was demonstrated. For instance, in this research, for non-composite steel girder bridges, a linear relationship was observed.

2- This research proposed the failure cost based on the initial cost. The supporting idea behind this recommendation stems from the general economic failure cost, which is the same as the initial cost.

3- The relationship between probability of failure and reliability index was modified because the former proposed model would not be able to represent the accurate relationship between the probability of failure and reliability index.

4- New generation of the time-dependent objective function was proposed, which would be able to determine target reliability with regard to the maintenance cost of the structure at the desired lifecycle.

5- The time discretization approach was introduced to determine the target reliability from the proposed objective function at the lifecycle of the structure.

6- The structural importance factor, S_I, was defined as a modification to the failure cost, and, regarding the applied contour concept, the influence of the structural importance factor on the target reliability was illustrated. As expected, a higher value of S_I requires greater the target reliability. Hence, based on the required target reliability for a considered structural importance factor, the load factors can be calibrated.

7- In the case study, the corrosion model (Park 1999, Kayser and Nowak 1989) was examined to illustrate the deterioration of steel girders due to low, medium, and high corrosion conditions. Based on the obtained results, the high corrosion condition requires greater target reliability.

8- The target reliability of non-composite steel girder bridges was determined for bridges with various span lengths.

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3. Target Reliability Analysis for the Tunnels

Abstract III

The objective of this study is to provide background information for calibration of the design code for tunnels. The load and resistance factors are calculated using available statistical models and probability-based procedures.

This study describes the calibration procedure, i.e. calculation of load and resistance factors. The major steps include selection of representative structures, calculation of reliability for the selected structures, selection of the target reliability index and calculation of load factors and resistance factors. This paper also reviews load and resistance models. In particular, a statistical model is proposed for earth pressure (vertical and horizontal) and live load (weight of vehicles and passengers). Statistical models of resistance (load carrying capacity) are summarized for reinforced concrete in bending, shear and compression states.

The reliability indices are calculated for several segments of a selected circular tunnel designed according to the tunnel manual report FHWA-NHI-10-034 (2009) (Tunnel Manual). The resulting reliability indices were reviewed and the target reliability indices were selected for bending, shear and compression. Several sets of load factors and resistance factors were considered. All load and resistance factors are to be rounded to 0.05, therefore, the number of possible values was limited. The final recommendation was proposed based on the closeness to the target reliability index.
Keywords: *Load and Resistance Factor, Reliability Analysis, Target Reliability, Tunnel.*

3.1. Introduction

The paper includes seven Sections. After the introduction, Section 2 provides the description of the calibration procedure. The procedure is consistent with the calibration of the AASHTO LRFD Code for the design of bridges, as documented in NCHRP Report 368 (Nowak 1999). Section 3 covers the load models. For each load component, two parameters are considered: bias factor, λ, which is the ratio of mean-to-nominal and, V, coefficient of variation. The statistical parameters of the major load components were based on the available literature and previous research by the authors of this paper.

Resistance models are presented in Section 4. The load carrying capacity was determined for a circular tunnel provided by PB World. The analysis was performed for several sections and involved consideration of the ultimate capacity with regard to moment, shear and compression. For each limit state the statistical parameters also included λ and V.

The selected reliability analysis is described in Section 5. Resistance is considered as a lognormal random variable and total load effect as a normal random variable. A closed form formula is derived for calculation of the reliability index, β.

Reliability indices were calculated for the tunnel considered tunnel sections and the results are presented in Tables in Section 6. For each of the considered tunnel sections, the nominal load values were provided by the PB World. For each tunnel section, five segments were considered, and reliability indices were calculated for all of them. Nominal resistance was determined form the design formula (factored load has to be less than factored resistance). The obtained spectrum of reliability indices was reviewed to prepare a background for the selection of the target reliability index, β_T.

The selection of the target reliability index, β_T is considered in Section 7. The target reliability indices are considered separately for each limit state, i.e. bending, shear and compression. Then, the load and resistance factors are selected that result in reliability indices closest to the target value. The number of possible options is limited because load and resistance factors are rounded to 0.05. To confirm the validity of the recommended load and resistance factors, reliability indices were calculated and presented in Tables.

3.2.Calibration Procedure

Objective of calibration is to determine the load and resistance factors for tunnel design. The calibration procedure is consistent with the development of AASHTO LRFD Code (NCHRP Report 368). The procedure include the following steps:

Step 1. Review of the available literature and data.

The review will include the previous NCHRP projects, and other studies. The Research Team has a considerable experience in the reliability analysis of buried structures. An important part of the study is to collect and review previous research on statistical parameters of load and resistance parameters, in particular as related to tunnels.

Step 2. Select representative tunnel structures

Two types will be considered, a reinforced concrete box section and a reinforced concrete circular section. This step involves analysis of the technical drawings, dimensions, identification of structural types, materials, load components, type of soil, and so on. The design drawings were obtained from Parsons Brinckerhoff Inc., P&B, together with calculated values of load effects (bending moments and shear forces) at critical locations. The obtained designs are considered as representative for the tunnel structures covered by NCHRP 12-89.

For each of the considered components and cross section, the calculated load values include nominal (design) dead load, live load, earth pressure, water pressure and so on. The resistance (load carrying capacity) is calculated using the AASHTO provisions for reinforced concrete and prestressed concrete design. Load components and resistance (moment and shear) are calculated as unfactored nominal (design) values. The calculations are performed using the commercial program available to PB engineers. The information about this program will be provided by PB. Therefore, input data for calibration were provided by PB and it includes:

Technical drawings showing general view of the tunnel structure
Information about the materials (type, grade, and so on)

Calculation of nominal values of load components at critical locations (corners of wall plates, mid-span of roof slab, and so on)

Calculation of the nominal load carrying capacity (moment, shear and compression)
Information about the computer procedure used for calculation of design loads

Step 3. Formulation of limit state functions

Limit state function is a mathematical representation of the limit between acceptable and unacceptable performance of the considered structural component. A simple example of a limit state function is

$$g = R - D - E - L = 0 \tag{3.1}$$

where R = resistance (load carrying capacity), D = dead load, E = earth pressure and L = live load. If $g < 0$, is means that load is larger than load carrying capacity, which means the component fails. Otherwise, component is OK.

For tunnel components the limit state function can include more load components such as water pressure, horizontal and vertical earth pressure, surcharge and so on. The limit state function will be formulated for each considered design case. For the box section, the limit state functions will considered for the following design cases:

(a) moment at the center of vertical wall,
(b) moment at the center of horizontal roof slab
(c) moment at the center of horizontal bottom slab
(d) shear at the top of vertical wall
(e) shear at the end of bottom slab

(f) shear at the end of roof slab

and for a circular section, the circular structure is divided into several segments and the limit state functions will include moment, shear compression for all considered sections, including

(a) moment at each segment
(b) shear at the each segment
(c) compression at each segment

For each case, the load components will be identified and a mathematical equation will be written similar to Equation 3.1. These equations will be used in the reliability analysis.

Step 4. Nominal (design) values of load components and resistance

The nominal (design) values of load components were calculated by PB using a commercial program. These values represent moments, shear, and compression forces due to individual load components. We will use these values in further analysis.

Nominal (design resistance) will be calculated for two cases: for the actual tunnel design (as is) as provided by PB engineers. As required by the current AASHTO Specifications, i.e. using the following formula:

R_n = (factored load) $/\varphi =$ (sum of load components multiplied by load factors specified in the current AASHTO code) $/\varphi$,

where R_n = minimum nominal resistance required by the code, and φ = resistance factor.These two values of resistance (case (a) and (b)), will be used in the reliability analysis.

Step 5. Statistical parameters of load and resistance

The statistical parameters will be determined for each load component and resistance. For each load components, we will need to know the cumulative distribution function (CDF). In practice we will need at least two parameters: the mean value and standard deviation. It is convenient to actually use two non-dimensional parameters: the bias factor, λ, defined as the ratio of mean-to-nominal value and coefficient of variation, V, defined as the ratio of standard deviation and the mean. For dead load, live load and earth pressure related loads, the bias factors and coefficients of variation can be taken from previous studies (Nowak and Collins 2013).

In consideration of tunnel structures, the load components occur as a combination, or simultaneous occurrence. The probability of simultaneous occurrence of extreme load values is rather limited. To represent the actual situation, special load combination models were developed. These models take into account the fact that when considering load combination, some load components take average values. However, some of the load components can be correlated (this mean they are not independent of each other), for example a horizontal earth pressure on two sides of the tunnel can be almost the same (but opposite sign). These correlations require a special approach.

Step 6. Reliability analysis procedure

Reliability analysis procedure will be selected and adjusted for application to the considered tunnel structures. Reliability will be calculated in terms of the reliability index For example, for the limit state function represented by Figure 3.1, the reliability index, β, is (Nowak and Collins 2013).

$$\beta = \frac{(R_m - D_m - E_m - L_m)}{\sqrt{{\sigma_{R_m}}^2 + {\sigma_{D_m}}^2 + {\sigma_{E_m}}^2 + {\sigma_{L_m}}^2}} \tag{3.2}$$

where R_m, D_m, E_m, L_m are mean values, and σ_{R_m}, σ_{D_m}, σ_{E_m}, $\sigma_{L_m}$are standard deviations. Presence of correlated load components requires a special consideration. The approach was developed by the research team in the previous studies.

Step 7. Calculation of reliability indices

The reliability indices will be calculated for the selected representative tunnel structures provided by PB, for the considered design cases and limit states. The calculations will be performed for two sets of nominal resistance values as defined in Step 4 above. The resulting reliability indices will be treated as representative for the current design (before calibration).

The results will be presented in Tables and graphs. The results will serve as a basis for the calibration of the code for tunnels, i.e. selection of the target reliability index and then selection of the load and resistance factors.

Step 8. Selection of the target reliability index

The results of the reliability analysis will serve as a basis for the selection of the target reliability index, β_T. This Step will involve the review of calculation results in Step 7. It is expected that there will be a wide range of β values. Selection of the target depends on several considerations. The most important are consequences of failure. This means that if failure to satisfy the limit state function (i.e. have $g < 0$) is followed by serious consequences, then β_T should be high. For example, in the calibration of ACI 318, β_T for columns is 4.0, while for beams β_T is 3.5, because failure of columns is considered more serious than failure of beams. Another important consideration is the cost. If safety is cheap, we buy more of it, if it is prohibitively expensive, we accept a lower safety level.

In this study, the target reliability index will be consistent with slab design in AASHTO LRFD Code and ACI 318.

Step 9. Calculation of load and resistance factors

Calculation of load and resistance factors is the final step in the calibration procedure. For consistency of the code, the load factors that are not tunnel-specific (e.g. dead load and live load) will be assumed the same as in AASHTO LRFD Code. For tunnel-specific load components, the preliminary values of load factor, γ, will be determined from the formula

$$\gamma = \lambda(1 + nV) \tag{3.3}$$

where λ is the bias factor and V is coefficient of variation of the load component. Parameter n can be taken about 1.8-2.0 for the strength limit states (NCHRP Report 368).

The number of possible values of load factors is limited because they are rounded to the nearest 0.05. Therefore, for each load component, further calculations will be carried out for three possible values of load factor: one determined from Equation 3.3, rounded off to the nearest 0.05, and two other values larger and smaller by 0.05.

For correlated loads, load combination factors will be considered using the approach used in previous studies. For each considered set of load factors, the required nominal resistance will be calculated from the following equation,

R_n= (factored load) /φ = sum of load components multiplied by the corresponding set of load factors, (4)

where R_n = nominal resistance corresponding to the considered set of load factors, and φ = resistance factor.

Resistance factors for reinforced concrete wall and roof will be taken consistent with the AASHTO LRFD. For comparison, the reliability analysis will also be performed for φ factors higher and lower by 0.05 than AASHTO LRFD specified values.

Reliability analysis will be performed for a wide range of combinations of load factors. The results will be presented in Tables and graphs. The final recommendation as to load and resistance factors will be based on closeness to the target reliability index.

Step 10. Final check and presentation of results

The reliability indices will be calculated for the recommended set of load and resistance factors. The calibration procedure will be documented in the Calibration Report.

3.3.Load Models

The load components for the considered circular tunnel include dead load (self weight), vertical earth pressure, horizontal earth pressure, water pressure (horizontal and uplift), and live load (static and dynamic). Load models are developed using the available statistical data, surveys and other observations, and engineering judgment. Load components are treated as random variables. Their variation is described by the cumulative distribution function (CDF), mean value and coefficient of variation. The load components considered in this study are shown in Figure 3.1. The following notation is used:

DC = self weight of structural components (cast-in-place concrete),
DW = superimposed dead load
LL = live load,
I = impact load (due to the live load),
WA = hydrostatic pressure,
EV = vertical earth pressure (gravity force),
EH = horizontal earth pressure
ES-V = vertical building surcharge load,
ES-H = horizontal building surcharge load,
EHL = horizontal rock load (applied on the left side acting toward the right side),
HER = horizontal rock load (applied on the right side acting toward the left side).

The basic load combination for the tunnel structures evaluated is a simultaneous occurrence of dead load, earth and water pressure, and live load. It is assumed that the economic life time for newly designed structures is 75 years. Therefore, the extreme values of load components are extrapolated accordingly from the available data base. The statistical parameters of all load components correspond to 75 year time period.

Nominal values of load components were provided by PB World. The nominal values were determined according to the Tunnel Manual.

Dead Load

Dead load, DC, is the gravity load due to the self weight of the structural and non structural elements permanently connected to the structure. The statistical parameters of dad load are λ = 1.05 and V = 0.10 (Nowak and et.al 2001).

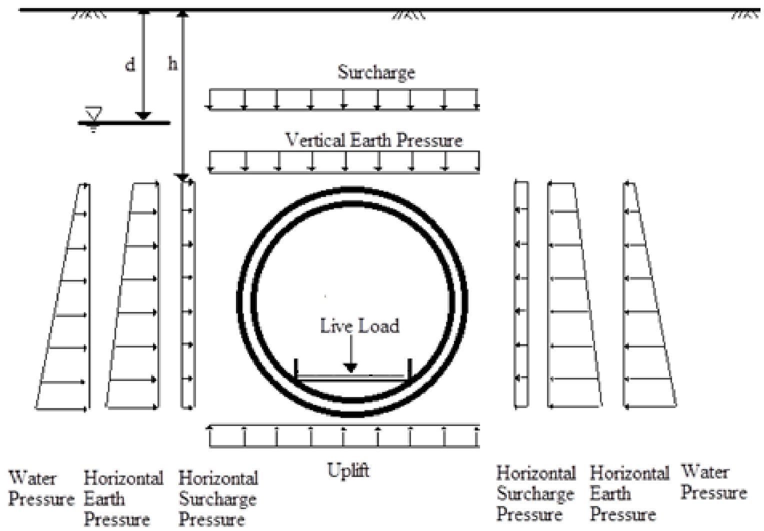

Figure 3.1. Load Components Considered in Calibration.

Components of DC are treated as normal random variables. The statistical parameters of dead load are taken as used in the previous bridge code calibration (NCHRP Report 368).

Superimposed Dead Load

Superimposed dead load is the weight of wearing surface and utilities. Utilities in tunnel can include drainage pipes, water supply lines, power lines, and etc. The DW behaves as a normal random variable. λ = 1.03 and the coefficient of variation V = 0.08 (Nowak and et.al 2001).

Vertical Earth Pressure

Buried structures are subjected to vertical and horizontal earth pressures. Furthermore, in many cases, earth pressure is the major load component (up to 90 percent of the total load effect).

Vertical earth pressure, EV, is caused by the self weight of earth placed on top of the structure. The actual load depends on the cover depth, h, density of material and compacting intensity. The statistical parameters of EV including bias factor (mean-to-nominal value) for the earth cover depth λ = 1.00 and the coefficient of variation V = 0.075 (Nowak and et.al 2001).

It is assumed that the design (nominal) earth density is determined by geotechnical engineers for each considered location (site-specific). Accordingly, the statistical parameters for the vertical earth pressure are λ = 1.0 and V = 0.14. Variation of soil cover does not include intentional alterations.

Vertical Surcharge

Surcharge, ES, represents the effect of building surcharge load over the buried structure specifies the statistical parameters of the surcharge load equal to: λ = 1.0 and V = 0.15 (Nowak and et.al 2001).

Horizontal At-Rest and Active Earth Pressure

Horizontal (lateral) earth pressure, EH, is applied to side walls. The actual value depends on construction method, compacting intensity, and water level. Lateral earth pressure is a resultant of at-rest pressure (normal conditions) or active pressure (during construction). For compacted earth fill or disturbed materials, Ko is lower than for at-rest condition, and it is reasonable to assume K_0 in range of 0.5 (EH1) to 1.0 (EH2). The statistical parameters of EH are assumed $\lambda = 0.95$ and V = 0.15. For active earth pressure the parameters are $\lambda = 0.80$ and V = 0.15. These values are based on the information provided in Nowak and et.al 2001.

Horizontal Surcharge

The horizontal pressure due to surcharge, ES-H, is modeled similar to horizontal earth pressure, ES-H, can occur on one side only. The actual value depends on the K factor, which is a subject to a considerable variation. Therefore, the parameters are $\lambda = 0.95$ and V = 0.15.

Water Pressure

Water pressure, WA, depends on the groundwater table in relation to the cover depth, h. The weight of water is 9.8 kN/m3. The major source of uncertainty in estimation of WA is the depth of the water table. The water table depth can vary in time. The statistical parameters of WA are $\lambda = 0.90$ and V = 0.15.

Live Load

The statistical parameters of live load, LL, depend on the span length, in this study they are assumed $\lambda = 1.25$ and V=0.18 (NCHRP Report 368).

Horizontal Rock Pressure

Horizontal rock pressure, ER can be applied to both sides of the structure (ER-R, which is applied on the right side acting toward the left side, and ER-L which is applied on the left side acting toward the right side). The bias factor of the horizontal rock pressure is, $\lambda = 1.0$, and the coefficient of variation of rock pressure can be considered as smaller than that of earth pressure, V = 0.12 (Nowak and et.al 2001).It is assumed that the loads acting on two opposite side are almost fully correlated. The coefficient of correlation is taken as 0.95.

Table 3.1. Statistical Parameters of Load Components

Load Component	Bias Factor	V
Dead load	1.05	0.10
Superimposed dead load	1.03	0.08
Live load	1.25	0.18
Hydrostatic pressure	0.90	0.15
Vertical earth pressure	1.00	0.14
Horizontal earth pressure	0.95	0.15
Vertical building surcharge load	1.00	0.15
Horizontal building surcharge load	0.95	0.15
Horizontal rock load	1.00	0.12

Load Combination

The total load effect, Q, is a combination of all components. Its model depends on load durations and probabilities of simultaneous occurrence. Dead load and earth pressure can be considered as time-invariant loads, whereas live lo*ad varies with time.

For tunnels, the load is dominated by earth and water pressure, with live load being within 0.05 of the total load effect. For the ultimate limit states (flexural capacity, shear, and compression capacity), the major load combination, Q, is

$$Q = DC + EV + WA + EH \tag{3.5}$$

where DC = resultant dead load, EV = resultant earth pressure, WA= resultant water pressure, and EH = resultant surcharge load.

3.4.Resistance Models

The structural capacity depends on the resistance of components and connections. The component resistance, R, is determined mostly by material strength and dimensions. R is a random variable and it can be considered as a product of the following parameters (Ellingwood et al. 1980):

$$R = M\,F\,P\,R_n \tag{3.6}$$

where M = material factor representing properties such as strength, modulus of elasticity, cracking stress, and chemical composition; F = fabrication factor including geometry, dimensions, and section modulus; P = analysis factor such as approximate method of analysis, idealized stress and strain distribution model.

The variation of resistance has been modeled by tests, simulations, observations of existing structures and by engineering judgment. The statistical parameters are developed for reinforced concrete slabs and beams (Nowak and Rakoczy (2012a)). Shear resistance is calculated using the modified compression field theory (Nowak and Rakoczy (2012b)).

Bias factors and coefficients of variation are determined for material factor, M, fabrication factor, F, and analysis factor, P. Factors M and F are combined. The parameters of R are calculated as follows:

$$\lambda_R = (\lambda_{FM})(\lambda_P) \tag{3.7}$$

where λ_R = bias factor of R; λ_{FM} = bias factor of FM; and λ_P = bias factor of P, and

$$V_R = \sqrt{V_{FM}^2 + V_P^2} \tag{3.8}$$

where V_R = coefficient of variation of R; V_{FM} = coefficient of variation of FM; and V_P = coefficient of variation of P.

Validity of the procedure was checked by comparison of parameters (material properties and dimensions), and analytical models, and it was concluded that the results are applicable to tunnel structures. Statistical data on material and dimensions used in previous report (NCHRP Report 368) was based on the available literature,

Recently it was observed that the quality of materials such as reinforcing steel and concrete has improved over the years. Therefore the material database has been updated, and so updated parameters were used (Nowak and Rakoczy (2012) as shown in Table 3.2.

Table 3.2. Statistical parameters of resistance based on the Nowak and Rakoczy (2012) for moment and shear carrying capacity. For axial load carrying capacity Monte Carlo simulation is used

Statistical Parameters of resistance	λ_R	V_R
Flexure	1.140	0.080
Minimum practical shear reinforcement (2#3 bars)		
f_c' = 20.7 MPa (3000 psi)	1.26	0.15
f_c' = 27.6 MPa (4000 psi)	1.24	0.145
f_c' = 34.5 MPa (5000 psi)	1.21	0.14
f_c' = 41.4 MPa (6000 psi)	1.19	0.135
Shear, average shear reinforcement		
f_c' = 20.7 MPa (3000 psi)	1.225	0.135
f_c' = 27.6 MPa (4000 psi)	1.225	0.13
f_c' = 34.5 MPa (5000 psi)	1.21	0.125
f_c' = 41.4 MPa (6000 psi)	1.19	0.135
Axial compressive load		
f_c' = 27.6 MPa (4000 psi)	1.22	0.14
f_c' = 34.5 MPa (5000 psi)	1.18	0.12
f_c' = 41.4 MPa (6000 psi)	1.15	0.11

3.5. Reliability Analysis

Limit states are the boundaries between safety and failure. In these structures failure can be defined as inability to carry traffic. Structures can fail in many ways, or modes of failure, by cracking, corrosion, excessive deformations, exceeding carrying capacity for shear or bending moment, local or overall buckling, and so on. Some members fail in a brittle manner, some are more ductile. In the traditional approach, each mode of failure is considered separately.

There are two types of limit states. Ultimate limit states (ULS) are mostly related to the bending capacity, shear capacity and stability. Serviceability limit states (SLS) are related to gradual deterioration, user's comfort or maintenance costs. The serviceability limit states include fatigue, cracking, deflection or vibration.

A traditional notion of the safety limit is associated with the ultimate limit states. For example, a beam fails if the moment due to loads exceeds the moment carrying capacity. Let R represent the resistance (moment carrying capacity) and Q represent the load effect (total moment applied to the considered beam). Then the corresponding limit state function, g, can be written, (see Nowak and Collins (2013)).

$$g = R - Q \tag{3.9}$$

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

$$P_F = Prob\,(R - Q < 0) = Prob\,(g < 0) \tag{3.10}$$

Let the probability density function (PDF) of R be f_R and PDF of Q be f_Q . Then let $Z = R - Q$. Z is also a random variable and it represents the safety margin, as shown in Fig. 3.2.

In general, limit state function can be a function of many variables (load components, influence factors, resistance parameters, material properties, dimensions, analysis factors). A direct calculation of P_F may be very difficult, if not impossible. Therefore, it is convenient to measure structural safety in terms of a reliability index.

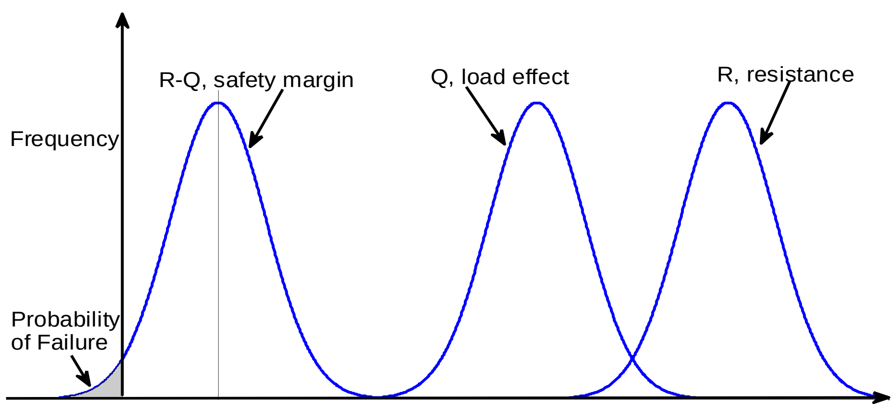

Figure 3.2. PDF's of Load, Resistance and Safety Reserve. Nowak and Collins (2013)

3.5.1. Reliability Index

The reliability index, β, is defined as a function of P_F, the calculation procedure of the reliability index was described by Nowak and Collins (2013). This research use Nowak and Collins (2013) formula to compute the reliability index, which is summarized as follows:

$$\beta = -\Phi^{-1}(P_f) \tag{3.11}$$

where Φ^{-1} = inverse standard normal distribution function. Examples of β's and corresponding P_f's are shown in Table 3.3.

There are various procedures available for calculation of β. These procedures vary with regard to accuracy, required input data and computing costs.

The simplest case involves a linear limit state function (Equation 3.8). If both R and Q are independent (in the statistical sense), normal random variables, then the reliability index is,

$$\beta = \frac{m_R - m_Q}{\sqrt{\sigma_R{}^2 + \sigma_Q{}^2}} \tag{3.12}$$

where m_R = mean of R, m_Q = mean of Q, σ_R = standard deviation of R and σ_Q = standard deviation of Q.

Table 3.3. Probability of Failure vs. $\boldsymbol{\beta}$.

Reliability index β	Reliability S (= 1 - P_f)	Probability of Failure
0.0	0.50	$0.500\times10^{+0}$
0.5	0.691	$0.309\times10^{+0}$
1.0	0.841	$0.159\times10^{+0}$
1.5	0.933 2	0.668×10^{-1}
2.0	0.977 2	0.228×10^{-1}
2.5	0.993 79	0.621×10^{-2}
3.0	0.998 65	0.135×10^{-2}
3.5	0.999 767	0.233×10^{-3}
4.0	0.999 968 3	0.317×10^{-4}
4.5	0.999 996 60	0.340×10^{-5}
5.0	0.999 999 713	0.287×10^{-6}
5.5	0.999 999 981 0	0.190×10^{-7}
6.0	0.999 999 999 013	0.987×10^{-9}
6.5	0.999 999 999 959 8	0.402×10^{-10}
7.0	0.999 999 999 998 72	0.128×10^{-11}
7.5	0.999 999 999 999 9681	0.319×10^{-13}
8.0	0.999 999 999 999 999 389	0.611×10^{-15}

If both R and Q are lognormal random variables, then β can be approximated by

$$\beta = \frac{ln\left(\frac{m_Q}{m_Q}\right)}{\sqrt{V_R{}^2 + V_Q{}^2}} \tag{3.13}$$

where V_R = coefficient of variation of R and V_Q = coefficient of variation of Q. A different formula is needed for larger coefficients of variation.

Equation 3.12 and 3.13 require the knowledge of only two parameters for each random variable, the mean and standard deviation (or coefficient of variation). Therefore, the formulas belong to the second moment methods. If the parameters R and Q are not both normal and lognormal, then the formulas give only an approximate value of β. In such a case, the reliability index can be calculated using the Rackwitz and Fiessler procedure, sampling techniques or by Monte Carlo simulations.

3.5.2. Reliability Methods used in Calibration

The statistical parameters of load and resistance are determined on the basis of the available data, simulations and engineering judgment. The techniques used in this study include Monte Carlo and the integration procedure developed by Nowak and et.al (1987).

The reliability is measured in terms of the reliability index. It is assumed that the total load, Q, is a normal random variable. The resistance is considered as a lognormal random variable.

For given nominal (design) value of resistance, R_n, the procedure used to calculate the reliability index, β, is outlined below (Nowak and Collins 2013).

1. Given:

resistance parameters: R_n, λ_R, V_R
load parameters: m_Q, σ_Q

2. Calculate the mean resistance, $m_R = \lambda_R R_n$.

3. Assume the design point is $R^* = m_R(1 - \mathrm{k}V_R)$, where k is unknown. Take $k = 2$ (initial guess), and calculate $R^* = m_R(1 - 2V_R)$.

4. Value of the cumulative distribution function of R (lognormal), and the probability density function of R, for $R^*$are,

$$F_R(R^*) = \Phi\left[\left(ln(R^*) - ln(m_R)\right)/V_R\right] \tag{3.14}$$

$$f_R(R^*) = \varphi\left[\left(ln(R^*) - ln(m_R)\right)/V_R\right]/(V_R R^*) \tag{3.15}$$

Calculate the argument of function Φ and φ,

$$a = \left(ln(R^*) - ln(m_R)\right)/V_R \tag{3.16}$$

5. Calculate the standard deviation and mean of the approximating normal distribution of R, at R^*,

$$\sigma_Q' = \frac{\varphi\{\Phi^{-1}(a)\}}{\left[\frac{\varphi(a)}{(V_R R^*)}\right]} = V_R R^* \tag{3.17}$$

$$m'_R = R^* - \sigma'_Q \Phi^{-1}[\Phi(a)] = R^* - a\sigma_{R'} \quad (3.18)$$

The load, Q, is normally distributed, therefore, the mean and standard deviation are m_Q and σ_Q.

6. Calculate the reliability index, β,

$$\beta = \frac{R^* - aV_R R^* - m_Q}{\sqrt{(V_R R^*)^2 + \sigma_Q{}^2}} \quad (3.19)$$

7. Calculate new design point,

$$R^* = \frac{m'_R - \beta(V_R R^*)^2}{\sqrt{(V_R R^*)^2 + \sigma_Q{}^2}} \quad (3.20)$$

8. Check if the new design point is different than what was assumed in step 3. If the same, the calculation of β is completed, otherwise go to step 4 and continue. In practice, the reliability index can be obtained in one cycle of iterations.

The formula for reliability index can be expressed in terms of the given data (R_n, λ_R, V_R, m_Q, σ_Q) and parameter k. By replacing R^* with $R_n\lambda_R(1 - kV_R)$, a with Equation 3.20, after some rearrangements, the formula can be presented as,

$$\beta = \frac{R_n\lambda_R(1-kV_R)[1-ln(1-kV_R)]-m_Q}{\sqrt{[R_n V_R \lambda_R(1-kV_R)]^2+\sigma_Q{}^2}} \quad (3.21)$$

3.6.Reliability Indices for Tunnels

The code calibration is based on calculations performed for a selected set of structures. The selection was based on structural type, dimensions and cover depth. The list of structures is provided in Table 3.4.

Table 3.4. Dimensions of Selected Structures

Station	**Radius (ft.)**	**Depth to crown (ft.)**
Charles Glass 36S	11	0.3
100 S. South Charles Street	11	60.3
Bromo Seltzer Tower	11	71.0
Eastern Ave 1401 Garage	11	54.0
Grudelsky	11	68.5
Holiday Inn - Inner Harbor Hotel	11	65.9
Market Center West Apts	11	65.9
Marriot Hotel	11	68.7

The basic design requirement according to the Tunnel Manual is given by formula in Equation 3.1. The reliability indices are calculated for reinforced concrete slabs and the limit states (moment, shear, and compression) described by the representative load components and resistance.

For the selected structures, moments, shears, and compression are calculated due to applied load. Nominal (design) values can be calculated using the current Tunnel Manual the mean maximum 75 year values of loads are obtained using the statistical parameters presented in Table 3.1. Resistance is calculated in terms of the moment carrying capacity, shear capacity, and axial load carrying capacity. For each case, the minimum required resistance, $Rmin$, is calculated as the minimum R which satisfies the design manual. For given loads, Q_i, the minimum required resistance, $Rmin$, according to design manual can be calculated as follows, (see Nowak and Collins (2013)).

$$R_{min} = \left(\sum \gamma_i Q_i\right) / \varphi \tag{3.21}$$

where γ_i are load factors. The load factors specified in the Tunnel Manual are listed in Table 3.5.

Table 3.5. Load Factors Specified in Tunnel Manual

Load Combination	DC		DW		EH EV		ES		EL	LL, IM	WA	TU*, CR**, SH***	
Min/max	Max	Min	Max	Min	Max	Min	Max	Min				Max	Min
Strength I	1.25	0.90	1.5	0.65	1.35	0.90	1.5	075	1.00	1.75	1.00	1.20	0.5

*TU= uniform temperature, **CR= creep, and ***SH= shrinkage

In Tunnel Manual resistance factors for moment is considered $\varphi = 0.90$ for shear recommends $\varphi = 0.85$ and for compression represented$\varphi = 0.75$, for concrete structure. The reliability indices are calculated for moment and shear. For each considered case, given are: mean total load, m_Q, standard deviation of total load, σ_Q, nominal (design) value of resistance, R_n, and the reliability index,β. Bias factor for resistance for various cases are listed Table 3.2.

In order to compute the reliability indices, the tunnels are divided into five segments (see Figure 3.3). It is assumed that each segment is designed to resist the maximum moment, shear, and axial load within the considered segment.

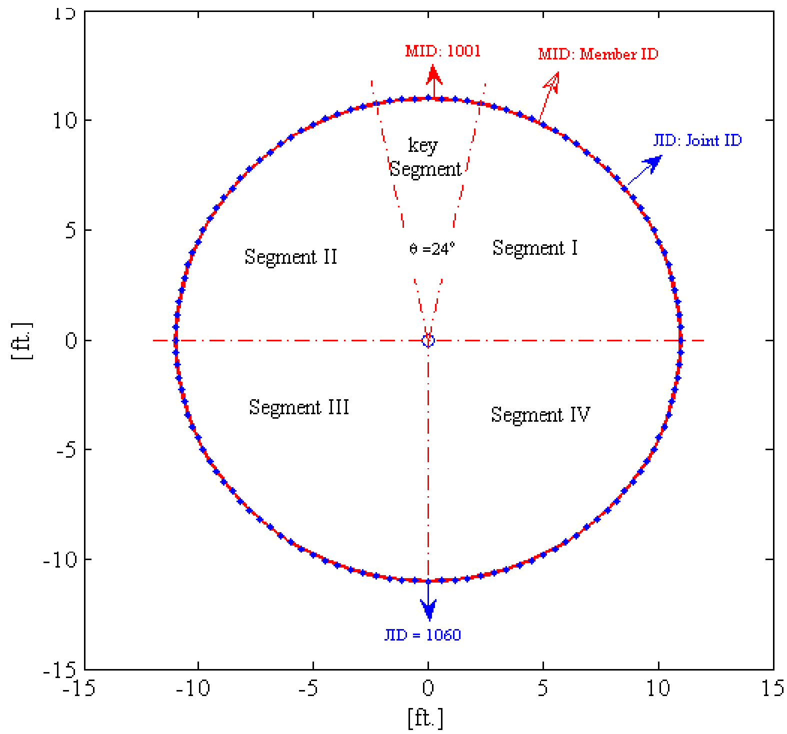

Figure 3.3. Considered Segments for Tunnel's Cross Section

The nominal resistance is determined using the factored loads, with load factors from the Tunnel Manual. Then, using the statistical parameters of load and resistance, the reliability indices were calculated for all the cross sections (eight stations shown in Table 3.4). For each case, the calculations were performed for three vales of resistance factor: one was a taken as specified in the Tunnel Manual, and two other values larger than smaller by 0.05. The calculation tables tabulates in appendix B.

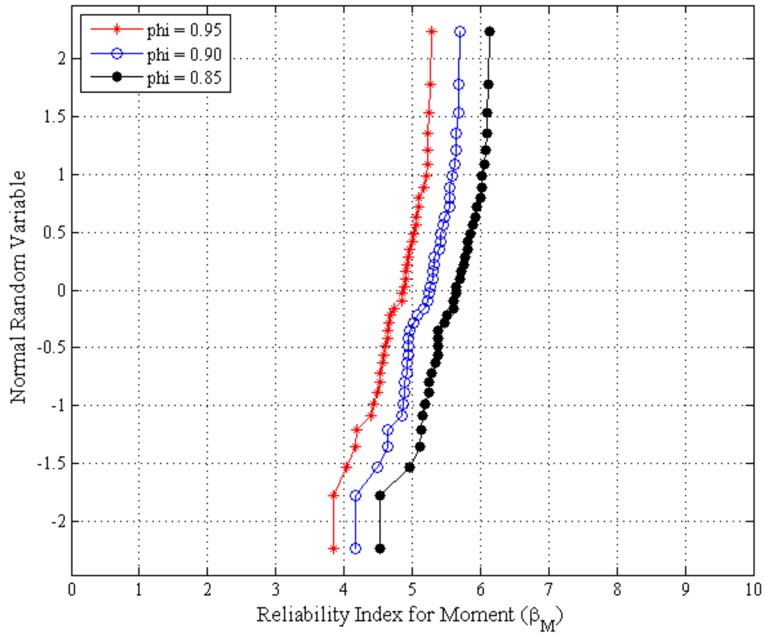

Figure 3.4. The Reliability Indices for Moment and Different Values of Resistance Factor, Using the Tunnel Manual (2009) Load Factors

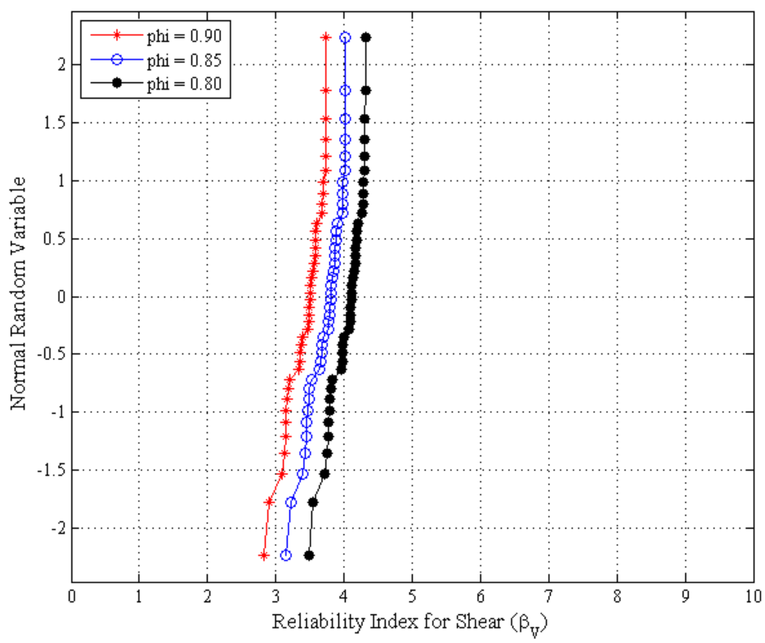

Figure 3.5. The Reliability Indices for Shear and Different Values of Resistance Factor, Using the Tunnel Manual (2009) Load Factors

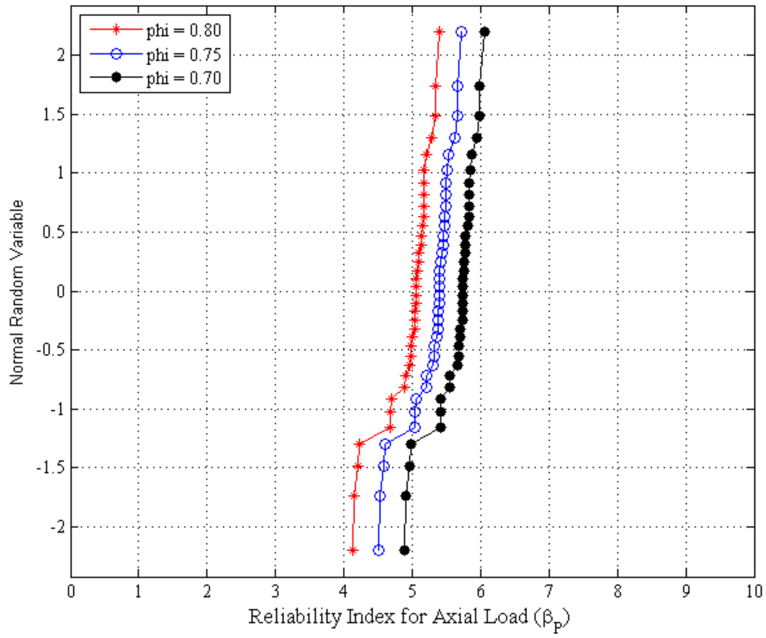

Figure 3.6. The Reliability Indices for Compression and Different Values of Resistance Factor, Using the Tunnel Manual (2009) Load Factors

Based on the review of the obtained reliability indices, the target reliability indices can be selected. Since the considered tunnel sections perform adequately, therefore, the proposed target reliability indices are as listed in Table 3.6.

Table 3.6. Proposed Target Reliability

	β_T
Moment	4.75
Shear	3.50
Compression	5.00

As can be seen in Figure 3.4 to 3.6, by decreasing the resistance factor, the reliability indices are increased. To obtain a more uniform spectrum of reliability indices, some adjustment of load factors were considered. The recommended set of load factors is shown in Table 3.7.

Table 3.7. Proposed New Load Factors

Load Combination	**DC***		**EHR**		**DW**		**EH EV**		**ES**		**LL, IM**	**WA**
Min/max	Max	Min	Max	Min	Max	Min	Max	Min	Max	Min		
Strength I	1.25	0.90	1.35	0.75	1.5	0.65	1.35	0.75	1.35	075	1.75	1.00

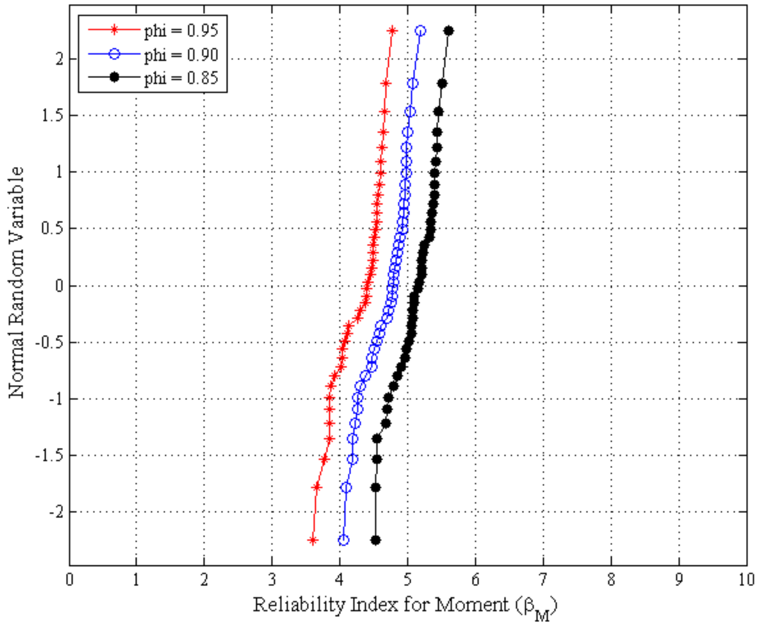

Figure 3.7. The Reliability Indices for Moment and Different Values of Resistance Factor, Using the Proposed Load Factors

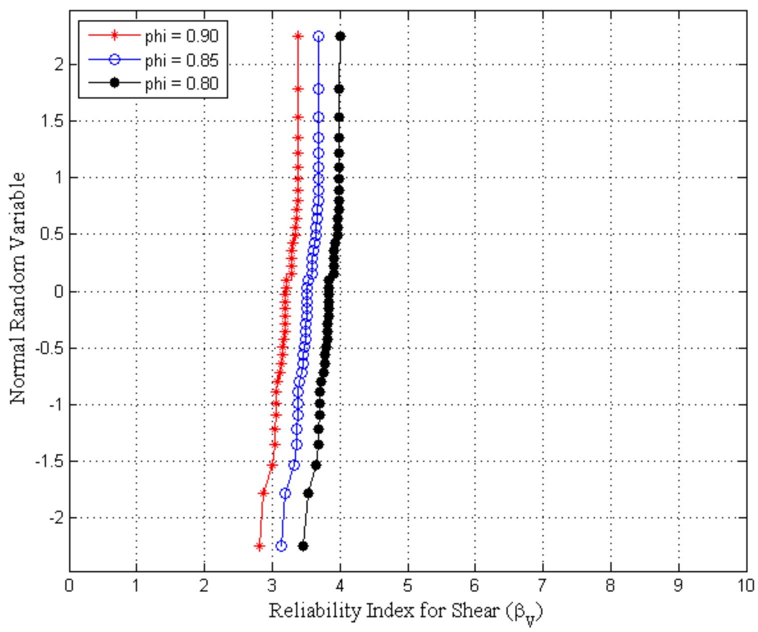

Figure 3.8. The Reliability Indices for Shear and Different Values of Resistance Factor, Using the Proposed Load Factors

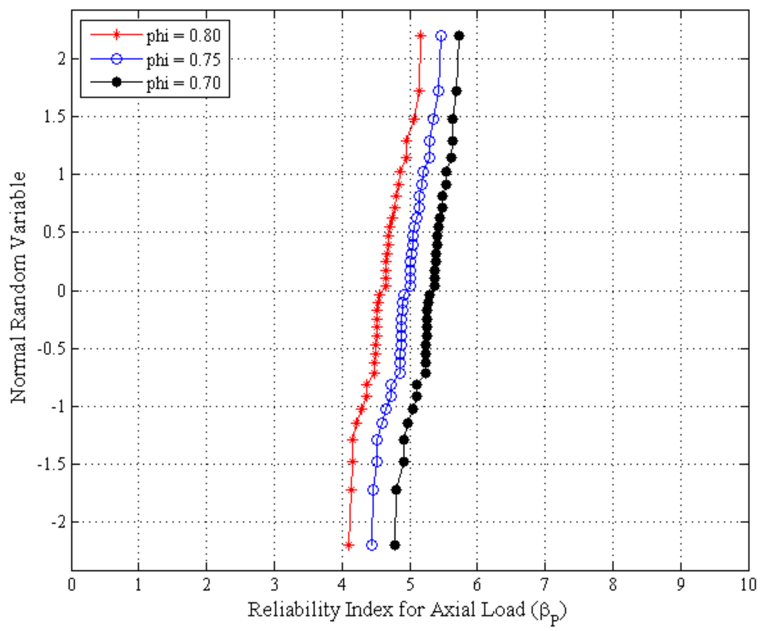

Figure 3.9. The reliability indices for compression and different values of resistance factor, using the proposed load factors

A comparison of the reliability indices obtained for the design using the load and resistance factors specified in the Tunnel Manual and the proposed load factors is shown in Figure 3.10 to 3.12 for moment, shear and compression, respectively.

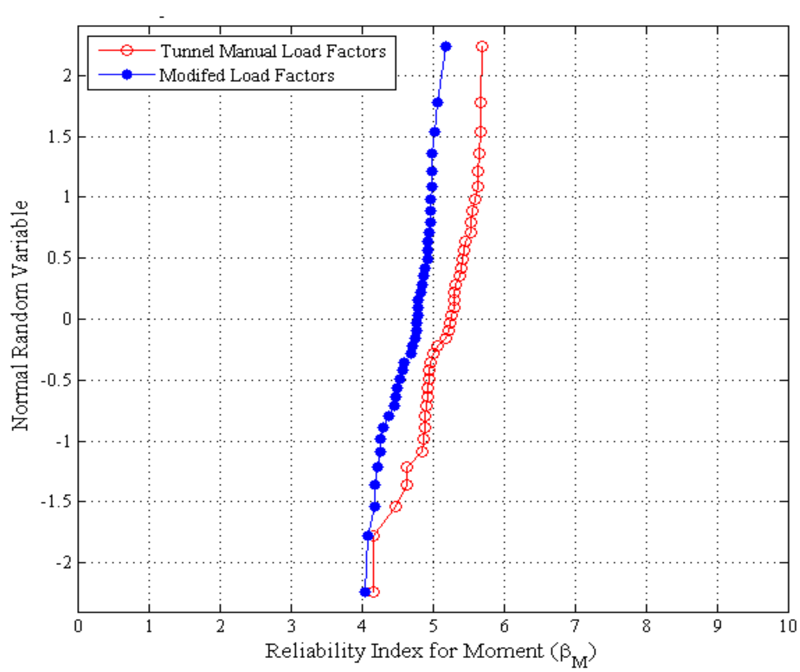

Figure 3.10. Comparison between the Load Factors in Tunnel Manual and New Proposed Load Factors (Moment Carrying Capacity)

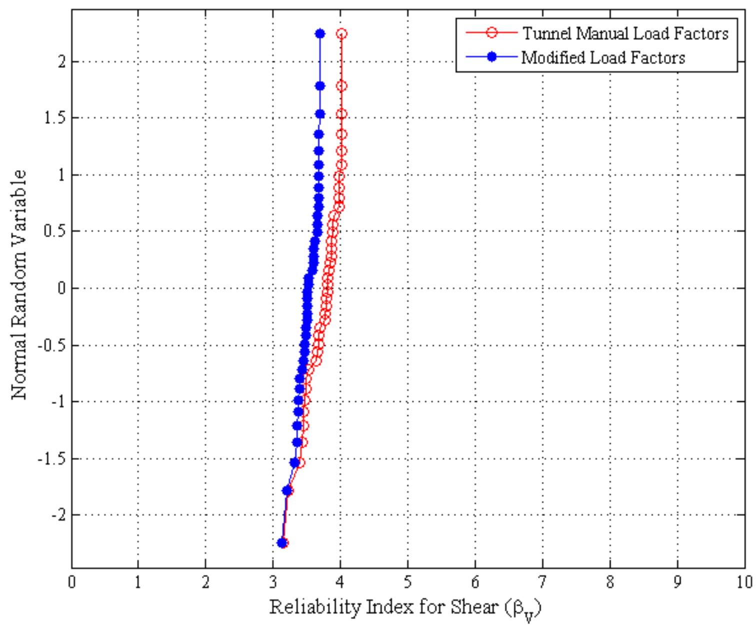

Figure 3.11. Comparison between the Load Factors in Tunnel Manual and New Proposed Load Factors (shear Carrying Capacity)

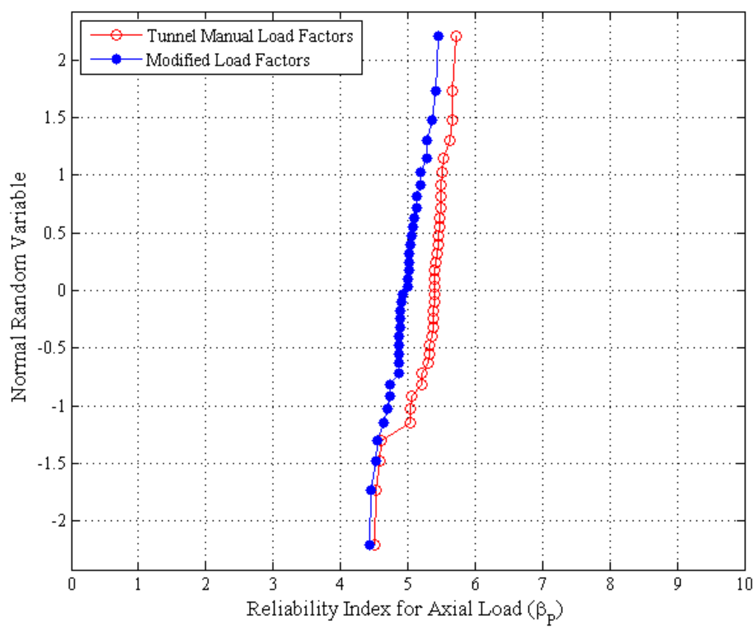

Figure 3.12. Comparison between the Load Factors in Tunnel Manual and new Proposed Load Factors (Compression carrying Capacity)

By comparison between variation of the reliability indices based on the Tunnel Manual load factors and proposed load factors, using the new load factors represents noticeably more constant reliability indices.

3.7.Conclusion

The reliability-based calibration resulted in selected target reliability indices and corresponding load and resistance factors. It is assumed that the resistance factors will be the same as specified in the Tunnel Manual. Target reliability indices and corresponding φ factors are listed in Table 3.8.

Table 3.8. Selected Target Reliability Indices

	φ	β_T
Moment	0.90	4.75
Shear	0.85	3.50
Compression	0.75	5.00

The major proposed change is adjustment of the load factors. The recommended load factors are as follows (for each load components two load factors are provided, one for maximum value and the other for minimum value):

Dead load	1.25/0.90
Horizontal rock pressure	1.35/0.75
Superimposed dead load	1.50/0.65
Horizontal earth pressure	1.35/0.75
Vertical earth pressure	1.35/0.75
Horizontal surcharge pressure	1.35/0.75
Vertical surcharge pressure	1.35/0.75
Live load and dynamic load	1.75/0.00
Water pressure	1.00/0.00

Reliability indices were calculated using these proposed load factors and φ factors from the Tunnel Manual. For comparison, the average values of φ were also calculated for the load factors from the Tunnel Manual. In addition, the calculation were also performed for φ +0.05 and φ –0.05. The results are shown in Table 3.9.

Table 3.9. Average target reliability of the tunnels resulting from Tunnel Manual load factors and proposed load factors

	φ	β	
		Old	**Proposed**
Moment	0.85	5.59	5.11
	0.90	5.17	4.69
	0.95	4.78	4.29
Shear	0.80	4.05	3.85
	0.85	3.75	3.52
	0.90	3.45	3.22
Compression	0.70	5.66	5.31
	0.75	5.31	4.95
	0.80	4.97	4.60

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4. Target Reliability for Serviceability Limit State in Bridges for Vehicular Deflection

Abstract IV

Design of the structures based on the reliability analysis requires to select a proper target reliability. In order to perform the reliability analysis for structures, it is necessary to determine the statistical parameters of the load and resistance. Then, based on the obtained statically parameters the reliability index can be computed.

In structural engineering, Serviceability Limit State, SLS, refers to the conditions which subjected to the durability of components and the structural stability. Using reliability analysis, this study intends to evaluate the safety level of SLS of bridges due to vehicular deflections. This research used the available Weigh In Motion, WIM, data to determine the statistical parameters of the deflection. For doing so, three different methods are considered. Eventually, using the obtained statistical parameters, the target reliability of the bridges in SLS due to the vehicular deflection is determined.

Keywords: *Serviceability Limit State, Target Reliability, Deflection of the Bridge.*

4.1.Introduction

The live load deflection limit is a constraint that controls intolerable vibrations, which do not necessarily cause damage to structures. However, they induce bothersome vibrations to bridges, which transfers anxiety to passengers or drivers. In 1905, the American Railroad Association, AREA, was one of the first design codes that made provisions for live load deflection; this design code restricted the deflection to the ratio of the span length (center-to-center bearing or the length between contraflexure points) to the total depth of the superstructure (including the concrete deck, haunch, and depth of the steel section). After that, the American Association of State Highway Official (AASHO) specified a certain limit on the live load deflection. Roeder et al. (2002) tabulated the considered limit of the live load deflection from an earlier year of the AREA and AASHO publications in National Cooperative Research Program, NCHRP, web document 41.

Table 4.1. Deflection Limit with Respect to the Span-to-Depth, L/D , Ratios by Roeder et al. (2002)

Year	Trusses	Plate girders	Rolled Beam
AREA			
1905	1/10	1/10	1/12
1907,1911,1915	1/10	1/12	1/12
1919, 1921, 1950, 1953	1/10	1/12	1/15
AASHO			
1913,1924	1/10	1/12	1/20
1931	1/10	1/15	1/20
1935, 1941, 1949, 1953	1/10	1/25	1/25

The Bureau of Public Roads conducted research to examine the unpleasant vibration limit on a bridge based on human mentality, and they decided on a new limitation equal to the length of the bridge divided by a certain value. Based on the reports of Oehler (1957), Wright and Walker (1971), and Fountain and Thunman in 1987, they agreed on a limitation of approximately $L/800$.

Later, in 1970, Oehler conducted a survey regarding the human response of passengers in vehicles or pedestrians who experienced a reaction to bridge vibration. Of 41 states, only 14 were informed of unpleasant vibrations; however, for those bridges, there was no structural deficiency. Then, based on the collected data, the bridges were classified with respect to the required deflection in three classes:

1- Bridges that are meant only for vehicular traffic can be designed based merely on the stress criteria, regardless of the deflection (ref. Oehler 1970).

2- Bridges that are designed in urban zones and subjected to pedestrians and parking application, should have a stiffness of at least approximately 200 kips/in. (ref. Oehler 1970)

3- Bridges with fishing benches, etc. should satisfy a minimum stiffness of approximately 200 kips/in with consideration of 7.5 percent critical damage (ref. Oehler 1970).

Wright and Walker continued their study and, in 1971, based on their investigations regarding human response and vibration damage to bridges, revealed that vibrations could not exert considerable effect on the structural performance and that they believed the deflection control could not be a reliable criterion to measure human comfort levels.

Since then, human reaction has played the most pivotal role in deflection control. Nowak and Groumi (1988a) posited that human satisfaction can be a proper parameters to establish the bridge deflection limit. Later, Nowak and Kim (1998b) and Nowak et al. (1988c, 2000) developed a guideline for evaluation of bridge deflection based on human reactions; the characteristic parameters that influence the human reaction stem from the deflection, acceleration, and frequency of the response. In 1996, Nowak and Saraf developed their guideline with regard to the load test on bridges. Recently, in 2011, Barker et al. developed a deflection criterion with regard to the natural frequency.

However, it worth considering the effect of the resonance phenomenon on bridges, which is related to the natural frequency of the bridge and the excitation frequency. If the main frequency of the excitation approaches to the main frequency of the bridges, the more severe the resonance effect would be expected. As a topic for future study considering the external expiation of vehicles, traffic or a human march, it may be possible to determine the critical excitation, (Ashtari and Ghasemi (2013), Ghasemi and Ashtari (2014)). And then, by combining the critical excitation with the reliability analysis (Ghasemi et al. (2013)), it would be feasible to determine the critical response with respect to the probability of the occurrence. Eventually, by measuring the human dissatisfaction level resulting from the vehicular vibration, the deflection criteria can be established with regard to the desired reliability consideration.

One of the main concerns in serviceability of bridges is the deflection control of the vehicular load. This design control was established to satisfy the human reaction because of the vehicle deflection. AASHTO LRFD 2014 presents four limitations for this type of deflection control, with regard to the application of the bridge (vehicular and pedestrian), material (steel, aluminum, and/or concrete), and boundary conditions (general, cantilever), as follows:

1. Vehicular load, general

$$\frac{L}{800} < \Delta \tag{4.1.1}$$

2. Vehicular and pedestrian loads

$$\frac{L}{1000} < \Delta \tag{4.1.2}$$

3. Vehicular load on cantilever arms

$$\frac{L}{300} < \Delta \tag{4.1.3}$$

4. Vehicular and pedestrian loads on cantilever arms

$$\frac{L}{375} < \Delta \tag{4.1.4}$$

where,

$\Delta =$ Deflection of the bridge subjected to the vehicular load

Because the AASHO LRFD is designed based on the reliability analysis, there is a need to determine the target reliability of the deflection. Therefore, it is necessary to compute the statistical parameters of the deflection resulting from the truck's load. This research generated MATLAB code to filter the trucks from the other vehicles. Then with regard to another generated sub-code the static deflection of bridges is calculated.

In this research, first the statistical parameters of deflection are obtained, and then the target reliability of bridges due to the serviceability limit state is determined. It is worth mentioning that in order

to illustrate the actual deflection, the dynamics effect should be applied. The next section deals with the considerations of the dynamic effect of the moving load.

4.1.1. Dynamics Effect

If the applied load on a system is a function of time, it can cause dynamic excitations. Because the moving load is a time-dependent excitation, the dynamic responses of the system can be calculated based on solving the partial equation of the wave motion. The wave equation was discovered by Jean-Baptiste le Rond d'Alembert in the eighteenth century and which is generally expressed as follows:

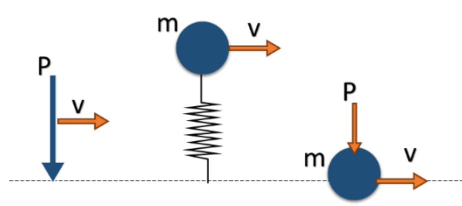
$$\frac{\partial^2 y}{\partial x^2} = \frac{1}{c^2}\frac{\partial^2 y}{\partial t^2} \tag{4.2}$$

where,

x = first space dimension, alongside of the wave propagation
y = second space dimension, time dependent displacement
t = time dimension
∂ = differential operator
c = constant parameters of the wave propagation speed.

In this study, the dynamic behavior of bridges when subjected to the moving load is considered. The moving load can be considered in three shapes (Dyniewicz and Bajer in 2012).

I. A simple massless force
II. An oscillator
III. An inertial force



(I) (II) (III)

Figure 4.1. Three Different Models for Moving Loads by Dyniewicz and Bajer in 2012

There are extensive studies in to compute the dynamic response of the moving load. For instance, Ingis (1934) attempted to create a proper vibration model of the moving load on railway bridges. Based on Ingis idea, In 1999, Fryba developed massless load in motion. Recently, in 2013, Bajer and Dyniewicz (2012) modeled the inertial load effects using numerical approaches. They also developed an analytical and finite element (FE) formulation in order to solve the equation of motion with respect to the moving load.

As a simple condition, let us model the moving load based on the constant P moving with constant velocity v on a simply supported string of length l, cross section A, mass density ρ, and tension force N. Smith (1964) used the following motion equation to deliberate the displacement of a beam.

$$-N\frac{\partial^2 \Delta(x,t)}{\partial x^2} + \rho A\frac{\partial^2 \Delta(x,t)}{\partial t^2} = \delta(x - vt)P \tag{4.3}$$

where,

$\Delta(x,t)$ = displacement of any point (x) at any time (t)
$\delta(.)$ = delta Dirac's function

Equation 4.3 is a partial differential equation that is established based on the wave equation subjected to the external excitation. The external excitation is a moving load that is assumed to be a constant force applied at position x at time t; therefore, the location of the load is a time-dependent parameter, which is determined by multiplication of magnitude of the load into the delta Dirac's function in the shape of $\delta(x - vt)P$. This equation is analytically solved by any proper mathematical approach, for instance, a Fourier's series concludes to the following responses, (Dyniewicz and Bajer (2012)):

$$\Delta(x,t) = \frac{2P}{\rho Al}\sum_{j=1}^{\infty}\frac{1}{\omega_{(j)}^2-\omega^2}\left(sin(\omega t) - \frac{\omega}{\omega_{(j)}}sin\big(\omega_{(j)}t\big)\right) \tag{4.4}$$

where,

$\omega = \frac{j\pi v}{l}\, j^{th}$, frequency of the moving load

$\omega_{(j)} = \sqrt{\frac{j^2\pi^2}{l^2}\frac{N}{\rho A}}$, j^{th} natural frequency of the strings

The second model of the moving load class (oscillator mass) demands the information of the dynamic system of the vehicle. Consider that the inertial moving load (case III) only causes the change into the excitation terms of the wave equation. Therefore, the moving load excitation consists of constant load and an influence of the inertia of the mass. Finally, the equation of motion is written as follows, which is mentioned by Dyniewicz and Bajer (2012).

$$-N\frac{\partial^2\Delta(x,t)}{\partial x^2} + \rho A\frac{\partial^2\Delta(x,t)}{\partial t^2} = \delta(x - vt)P - \delta(x - vt)m\frac{d^2\Delta(x,t)}{dt^2} \tag{4.5}$$

where,

$m =$ mass of the moving vehicle

$\frac{d^2\omega(vt,t)}{dt^2} =$ acceleration of the moving load

A comparison between Equations 4.3 and 4.5 shows that the external excitation of Equation 4.5 is significantly complicated. However, the semi-analytical solution was provided, and, for more complex cases, the finite element method (FEM) is used to transfer partial differential equation (PDE) into a summation of the ordinary differential equations (ODEs), which are much simpler to solve. It is worth mentioning that the FEM has a time stepping issue, and it cannot provide a comprehensive result for a system. The basic idea of the FEM is to discretize a system into several systems that are connected together with nodes and elements, and all external loads should be applied on the nodes; therefore, time stepping should cause time-intervals such that the moving load would be applied on the nodes, not on the elements. As a matter of fact, ignorance of this crucial issue may lead to the unreliable results. Considering the aforementioned FEM deficiency is a difficult task; therefore, for simple geometry cases, analytical solution still is a truthful option.

Finally, it would be possible to develop the wave equation for a bridge example, where the bending stiffness of the bridge is a considerable parameter (see Dyniewicz and Bajer (2012)).

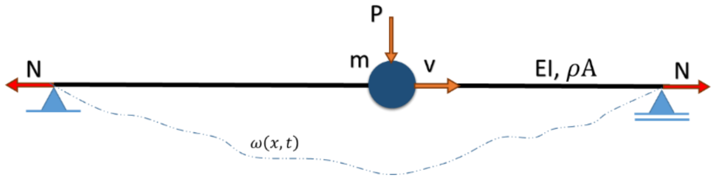


Figure 4.2. Deflection for a Simple Massless Force, (Dyniewicz and Bajer (2013))

$$EI\frac{\partial^4\Delta(x,t)}{\partial x^4} - N\frac{\partial^2\Delta(x,t)}{\partial x^2} + \rho A\frac{\partial^2\Delta(x,t)}{\partial t^2} = \delta(x - vt)P - \delta(x - vt)m\frac{d^2\Delta(x,t)}{dt^2} \tag{4.6}$$

where,

EI = bending stiffness

The general solution of the above equation is possible using different PDE approaches. The boundary conditions for a simply supported beam with a length of l can be defined as follows:

$$\omega(0,t) = 0,\ \omega(l,t) = 0,\ \frac{\partial^2\Delta(x,t)}{\partial x^2}\bigg|_{x=0} = 0,\ \frac{\partial^2\Delta(x,t)}{\partial x^2}\bigg|_{x=l} = 0 \tag{4.7}$$

Additionally, for initial conditions, it may be possible to assume the bridge was at the rest condition before applying a load

$$\omega(x,0) = 0,\ \frac{\partial\Delta(x,l)}{\partial t}\bigg|_{t=0} = 0 \tag{4.8}$$

Considering a moving-simple-massless-force (MSM) on a tensionless simply supported bridge (Figure below), the PDE of motion can be summarized as follows (Reference Dyniewicz and Bajer, 2012):

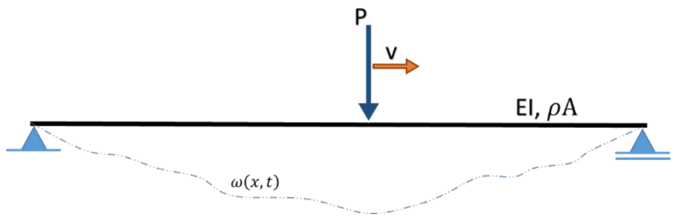


Figure 4.3. Defection for a simple massless force (Dyniewicz and Bajer (2012))

$$EI\frac{\partial^4\Delta(x,t)}{\partial x^4} + \rho A\frac{\partial^2\Delta(x,t)}{\partial t^2} = \delta(x - vt)P \tag{4.9}$$

$$\boldsymbol{M}\begin{bmatrix}\ddot{V}(1,t)\\ \ddot{V}(2,t)\\ \vdots\\ \ddot{V}(n,t)\end{bmatrix}[\ddot{V}] + \boldsymbol{K}\begin{bmatrix}V(1,t)\\ V(2,t)\\ \vdots\\ V(n,t)\end{bmatrix} = \boldsymbol{P} \tag{4.10}$$

In short form

$$\boldsymbol{M\ddot{V}} + \boldsymbol{KV} = \boldsymbol{P} \tag{4.11}$$

$$\boldsymbol{M}\begin{bmatrix}\ddot{V}(1,t)\\ \ddot{V}(2,t)\\ \vdots\\ \ddot{V}(n,t)\end{bmatrix}[\ddot{V}]+\boldsymbol{K}\begin{bmatrix}V(1,t)\\ V(2,t)\\ \vdots\\ V(n,t)\end{bmatrix}=\boldsymbol{P} \tag{4.12}$$

Regarding Equation 4.9, it is possible to expect that the dynamic responses of the bridges can be related to the natural frequency of the bridges and the vehicle speed; however, in addition to the dynamic behavior of the bridge, it is necessary to consider both the dynamic behavior of the moving load and the surface roughness; this analysis has been performed by Nowak and Hong (1991). Finally, based on the Nowak and Hong's results and dynamic analysis in our research, the dynamic impact factor is considered according to the AASHTO LRFD 2014 recommendation.

4.2.Computation of Deflection

To evaluate the bridge deflection (Δ), AASHTO LRFD presents four load combinations in Table 3.4.1-1. If the vertical displacement experienced is to be computed, regardless of the wind effect on the bridge or live load, Service II is the dominant load combination. However, the dynamic load effect should be properly considered; as clearly specified in AASHTO LRFD 2014, Article 2.5.2.6, in order to calculate the bridge deflection under the condition of load combination of service II, the dynamic load allowance, IM, shall amplify the static deflection by 33 percent (according to Table 3.62.1-1, the dynamic load allowance = 33%).

It is worth mentioning that if the bridge is made of wood or has particular conditions, AASHTO LRFD 2014 suggested several more criteria.

In this research, the vehicular load deflection is considered; however, this research methodology can be developed to address other types of the limit state function.

Deflection of the bridge can be calculated based on the deflection of its girders. Once the bridge is subjected to vertical loads, it can be assumed that the girders perform as a parallel system. Therefore, if the slab system properly distributes vertical load to the girders, and, compared to the girder stiffness, slab stiffness is negligible, girders deflection represent the total deflection of the bridge.

Regarding the structural analysis, the flexural deflection of a bridge is represented based on the flexural rigidity (EI), where E is the modulus of elasticity and I is the second moment of the area. AASHTO LRFD 2014 recommends evaluating the vehicular deflection according to the HL-93. HL-93 load consists of the distributed lane load and truck load. Lane deflection for simply supported beams can be computed easily based on the following formula (AASHTO LRFD 2014):

$$\Delta_{lane}=\left(\frac{5(\omega_l)L^4}{384EI}\right) \tag{4.13}$$

where,

$\omega_l = 0.64\ kip/ft$ is the distributed lane load

However, in order to determine the maximum deflection of the truck, it is necessary to place a truck on the bridge and calculate the mid-span deflection with regard to the position of the axle loads of the truck, and, by using the superposition law. The total deflection of the bridge subjected to the truck loads is formulated as follows

$$\Delta_{truck} = Max\left\{\frac{P_1 b_1}{48EI}\left(3L^3 - 4{b_1}^2\right) + \frac{P_2 b_2}{48EI}\left(3L^3 - 4{b_2}^2\right) + \frac{P_3 b_3}{48EI}\left(3L^3 - 4{b_3}^2\right)\right\} \quad (4.14)$$

where,

$P_1 = 8\ kip$, $P_2 = P_3 = 32\ kip$

Moreover, because the AASHTO code was designed based on the Load and Resistance Factor (LRFD), in order to suggest the design formula, AASHTO LRFD considers the load factors, which, for live load in a service II load combination, is

$$\gamma_l = 1.3, \text{ Live load factor} \quad (4.15)$$

Article 3.6.1.3.2 in AASHTO LRFD 2014 specifies two criteria to evaluate the bridge deflection, which is subjected to the HL-93 vehicular load, as follows

i. deflection resulting from the design truck alone, or
ii. deflection resulting from 25 percent of the design truck taken together with the design lane load.

where the larger value of those criteria should be used for deflection control.

Therefore, considering the vehicular load in general, the design formula can be written as Equation 4.16.

$$\frac{EIL}{800} - \gamma_L\left(max\left\{\sum_{i=1}^{3}\frac{P_i b_i}{48}\left(3L^3 - 4{b_i}^2\right)\right\}\right) > 0 \quad (4.16a)$$

$$\frac{EIL}{800} - \gamma_L\left(\frac{5(\omega_l)L^4}{384} + 0.25 * max\left\{\sum_{i=1}^{3}\frac{P_i b_i}{48}\left(3L^3 - 4{b_i}^2\right)\right\}\right) > 0 \quad (4.16b)$$

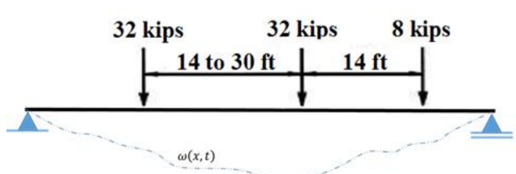


Figure 4.4A. Design Criteria for Deflection (i), (ref. AASHTO LRFD 2014)

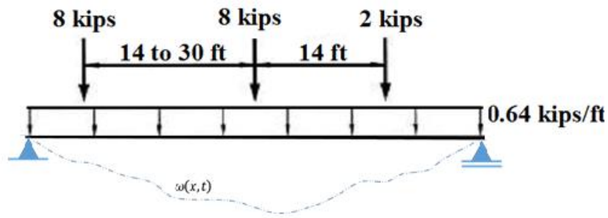


Figure 4.4B. Design Criteria for Deflection (ii), (ref. AASHTO LRFD 2014)

Table 4.2. Bridges Deflection resulting from the Design Criteria, AASHTO LRFD 2014

bridge length (ft.)	100	110	120	130	140	150
deflection resulting from truck $(k(lb)ft^3EI)$	-1,424,462	-1,913,112	-2,500,762	-3,196,412	-4,009,062	-4,947,712
deflection resulting from traffic $(k(lb)ft^3EI)$	-833,333	-1,220,100	-1,728,000	-2,380,100	-3,201,300	-4218,750
1/4truck+lane	-1,189,449	-1,698,378	-2353,191	-3,179,203	-4,203,566	-5455,678
design deflection	-1,424,462	-1,913,112	-2,500,762	-3,196,412	-4,203,566	-5455,678
Ratio: truck/HL-93	1.71	1.57	1.45	1.34	1.25	1.17
bridge length (ft.)	160	170	180	190	200	300
deflection resulting from truck $(k(lb)ft^3EI)$	-6,021,362	-7,239,012	-8,609,662	-10,142,312	-11,845,962	-40,267,462
deflection resulting from traffic $(k(lb)ft^3EI)$	-5,461,300	-6,960,100	-8,748,000	-10,860,000	-13,333,333	-67,500,000
1/4truck+lane	-6,966,641	-8,769,853	-10,,900,416	-13,395,578	-16,294,824	-77,566,866
design deflection	-6,966,641	-8,769,853	-10,900,416	-13,395,578	-16,294,824	-77,566,866
Ratio: truck/total	1.10	1.04	0.98	0.94	0.89	0.60

As can be seen, if the boundary conditions of the bridge are assumed as simply supported conditions, for bridges approximately or longer than 140 ft., the combination of the lane load and 25 percent of the truck load is the design criteria; however, for bridges shorter than 140 ft., the HL-93 truck load is the dominant case.

Accordingly, in order to specify the reliability index, the limit state function can be determined based on the following formula

$$g = \frac{L}{800} - \bar{\Delta} > 0 \tag{4.17}$$

where,

$\bar{\Delta}$= mean value of deflection

4.3.Maximum Deflection for Different Time Periods

To ascertain the reliability index of the structure, the statistical parameters (bias and standard deviation) of the load and resistance should be obtained; these statistical parameters are time-dependent variables. In this section the statistical parameters of bridge deflection subjected to the design load criteria is determined.

To determine the bias factor and standard deviation of the deflection, the databases of the Weigh-In-Motion (WIM) on the way for several sites in the United States were considered, as is shown in Table 4.3 in next section.

Table 4.3. Geographical Positions and Coordinates of the Studied Stations

State	Highway
Arizona1	US-93 North at M.P. 52.62
Arizona2	I-10 East at M.P. 108.6
California	SR-99 at M.P. 32.5
Colorado	I-76 East at M.P. 39.7
Delaware	US-113 Southbound North of SR 579
Illinois	I-57 at M.P. 225.6
Indiana	US-31 North at M.P. 216.9
Kansas	I-70 West at M.P. 287.48
Louisiana	US-171 at M.P. 8.4
Minnesota	US-2 at M.P. 91.8
New Mexico1	I-25 North at M.P. 36.1
New Mexico2	I-10 East at M.P. 50.2
Pennsylvania	I -80 at M.P. 158.2
Tennessee	I-40 West at M.P. 91.67
Virginia	US-29 bypass at M.P. 12.8
Wisconsin	US-29 at M.P. 189.8

The procedure to select the trucks was explained by Rakoczy (2011). In this research, the MATLAB code is generated by the Ghasemi and Nowak to compute the deflection of a bridge because of the trucks' loads. The collected WIM data refers to the specific time period, which, in most cases, is approximately 12 months; nevertheless, the number of vehicles is also variable because of the different average daily truck traffic (ADTT). Thus, the maximum deflection only illustrates the maximum deflection in that time period.

4.3.1. Maximum Deflection for Different Time Periods Based on the Previous Methods

The reliability analysis demands the statistical parameters at the desired time periods. Nowak in 1999 proposed a method to determine the statistical parameters of the Cumulative Distribution Function, CDF, for different time periods. Nowak plotted the CDF of the data on a probability paper. He considered the vertical coordinate of the probability paper, Z_{max}, which is the standard normal variable and refers to the specific number of vehicles N, as a maximum mean value for that specific number (also used by Rakoczy in 2011).

$$Z_{max} = -\Phi^{-1}\left(\frac{1}{N}\right) \tag{4.18}$$

where,

Φ^{-1} = inverse standard normal distribution function

Based on the reliability analysis, there is a certain safety level behind the design formula. In fact, in order to calculate the reliability index of the bridge resulting from the deflection of the vehicular load, it is necessary to obtain the statistical parameters of the deflection. Based on the dataset which was provided in Table 4.2, and by considering ADTT=5,000, this research intends to obtain the bias and standard deviation of the deflection.

The deflection ratio is defined as the ratio of the bridge deflection resulting from truck load to the deflection of the bridge resulting from the design criteria. As an example, Figure 4.5 illustrates the CDF of the deflection ratio for bridges with length about 100 ft. subjected to ADTT=5,000. Following the figures, the statistical parameters (maximum mean values and their standard deviation) of deflection are summarized in Table 4.4.

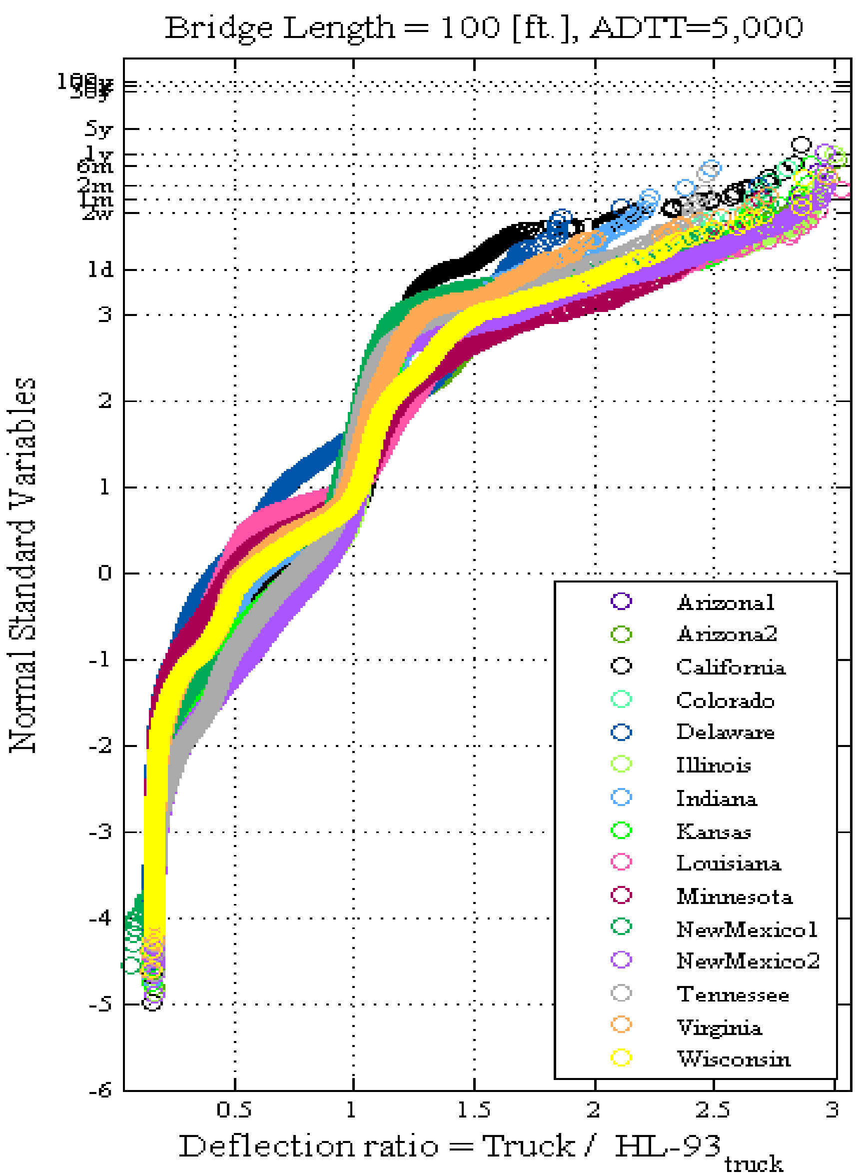


Figure 4.5. CDF of Deflection Ratio of WIM Deflection to the Design Truck Deflection, Span Length =100 [ft.]

Table 4.4 Statistical Parameters of the Bridge Deflection, Span Length = 100 ft., ADTT=5,000

Bridge Length = 100 [ft.], ADTT = 5,000, Truck / $(HL93)_{truck}$										
Site	**1d**	**2w**	**1m**	**2m**	**6m**	**1y**	**5y**	**50y**	**75y**	**100y**
Arizona1	2.076	2.821	2.893	2.948	2.996	2.996	3.027	3.075	3.088	3.1
Arizona2	1.431	2.14	2.488	2.631	2.821	2.837	2.893	2.895	2.901	2.916
California	2.092	2.615	2.663	2.71	2.829	2.853	2.89	3.075	3.091	3.107
Colorado	1.648	1.95	2.314	2.679	2.774	2.84	2.86	2.885	2.893	2.948
Delaware	2.219	2.901	2.932	2.94	2.98	2.996	3.035	3.043	3.051	3.15
Illinois	1.775	2.164	2.227	2.393	2.504	2.536	2.615	2.726	2.742	2.758
Indiana	2.14	2.647	2.837	2.837	2.901	2.916	2.933	2.964	2.916	2.975
Kansas	2.282	2.893	2.98	3.015	3.019	3.122	3.059	3.067	3.072	3.17
Louisiana	2.33	2.647	2.695	2.71	2.742	2.758	2.79	2.821	2.837	2.853
Minnesota	2.045	2.695	2.71	2.726	2.734	2.75	2.75	2.79	2.869	2.88
New Mexico 1	2.251	2.695	2.71	2.805	2.948	2.893	2.98	3.027	3.035	3.043
New Mexico 2	1.934	2.393	2.425	2.441	2.457	2.454	2.481	2.536	2.552	2.56
Tennessee	1.759	2.552	2.718	2.853	2.964	3.075	3.077	3.083	3.09	3.1
Virginia	2.108	2.695	2.861	2.869	2.877	2.901	2.885	2.92	2.924	2.932
Wisconsin	2.108	2.79	2.837	2.885	2.959	2.98	3.0	3.075	3.078	3.08
Statistical parameters										
Bias	2.01	2.57	2.69	2.76	2.83	2.86	2.89	2.93	2.94	2.97
Standard deviation	0.25	0.28	0.22	0.17	0.16	0.18	0.16	0.15	0.15	0.16

4.3.2. New Method to Evaluate the Maximum Mean Deflection Ratio for Different Time Periods

The obtained mean maximum value by the previous method does not refer to the exact mean maximum value in a considered time period because the time period was determined based on the assumption of the ADTT. However, today, the WIM data include the date information. Hence, the statistical parameters can be evaluated more precisely. For example, if the data are collected over one year, it is possible to determine the maximum daily value; then, by plotting the CDF of the maximum daily values on a probability paper, the statistical parameters are determined. This procedure is applied to the weekly or even monthly daily truck traffic. As an example, following Figure shows the daily, weekly and monthly extreme deflection ratio for the bridge with length of approximately 100 ft. for a station which is located in California (2012). The results for other sites are presented in Appendix C.

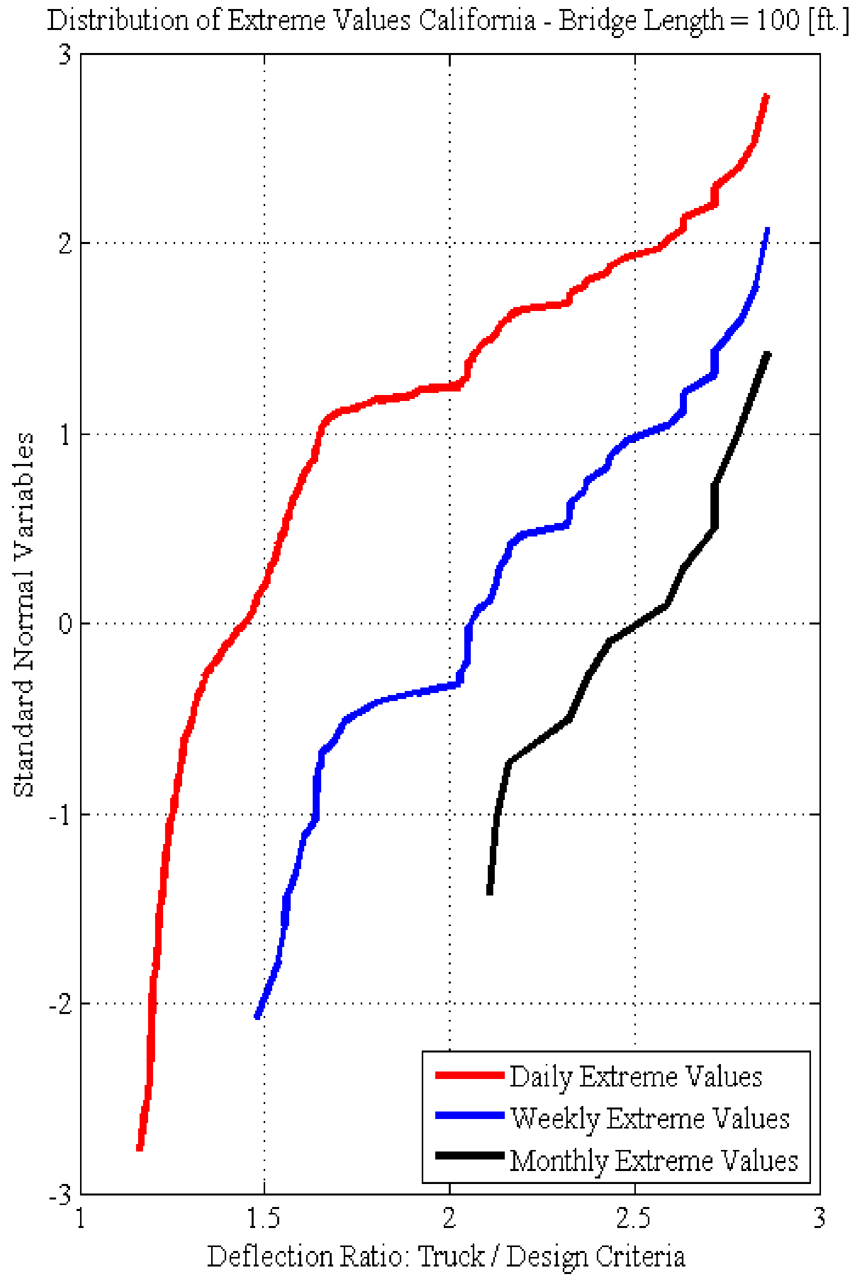


Figure 4.6. CDF of Deflection Ratio of WIM Deflection to the Design Truck Deflection, Span Length =100 [ft.]

4.3.3. Introducing the Probability Space to Determine the Statistical Parameters of the Bridges Deflection for the Different Time Periods With Respect to Its ADTT

As observed, according to the Nowak's method, determining the statistical parameters of deflection of bridges resulting from the WIM for different time periods was a tedious effort; moreover, it needs many justifications and roughly extrapolations to determine the maximum mean value, which brings two ambiguities: 1- the exact number of trucks, which corresponded to the maximum mean value and 2- the method of extrapolating. This research introduces the 3D probability space to compute the statistical parameters of deflection for the different time period.

The concept of the probability space is similar to the concept of the probability paper, except that it has one more coordinate to specify the volume of the truck traffic. The procedure to create the 3D probability plot for the daily deflection ratio is generated using the following steps:

1. Determine the daily truck traffic (DTT)
2. Compute the maximum daily deflection
3. Calculate the deflection ratio
4. Create a matrix with two columns, 1- Deflection ratio and 2- Daily truck traffic
5. Sort the matrix with respect to the deflection ratio
6. Find the probability value p_i based on the number of the day, N, in the investigated period

$$p_i = i/(N+1) \tag{4.19}$$

7. Create the standard normal variables based on the inverse function of the standard normal distribution

$$z_i = -\Phi^{-1}(p_i) \tag{4.20}$$

8. Add a z_i column to the matrix from step 5, and plot that matrix

The aforementioned procedure can be developed weekly, monthly or for any desired time period. After plotting the data on the 3D probability paper, it is time to fit a surface over the data to observe the relationship between the maximum mean value and DTT based on the actual scenarios.

Surface fitting depends on many factors, in this case, the linear pricewise interpolation would be one of the best and most exact approaches; however, from an engineering point of view, the simplest function would be the best answer. One of the simplest formulations to fit a surface on 3d data is polynomial fitting. Therefore, using MATLAB, this research considers polynomial functions. As an example the procedure applied to the bridge with length of 100 ft. for daily, weekly, and monthly traffic, which the result exhibits in Figure 4.7.

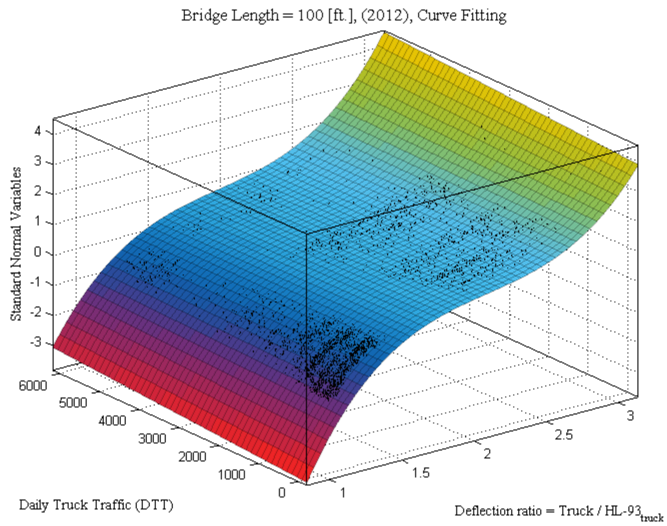


Figure 4.7A. 3D Probability Space Based on Maximum Daily Deflection, Bridge Length=100 [ft.]

$$f(x,y) = p_{00} + p_{10}x + p_{01}y + p_{20}x^2 + p_{11}xy + p_{30}x^3 + p_{21}x^2y$$

where,

$p00 = -17.64 \; (-17.86, -17.41)$

$p10 = 24.88 \; (24.49, 25.27)$

$p01 = 0.0003492 \; (0.000295, 0.0004035)$

$p20 = -11.46 \; (-11.68, -11.25)$

$p11 = -0.0003674 \; (-0.0004277, -0.0003072)$

$p30 = 1.823 \; (1.784, 1.863)$

$p21 = 9.073e-05 \; (7.488e-05, 0.0001066)$

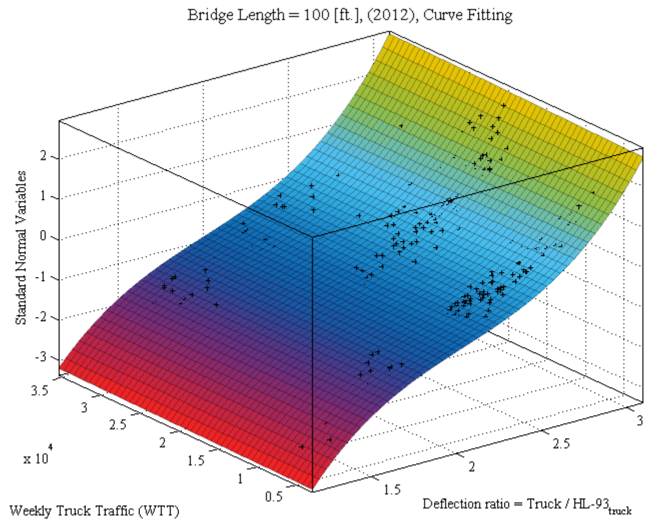


Figure 4.7B. 3D Probability Space Based on Maximum Weekly Deflection, Bridge Length=100 [ft.]

$$f(x,y) = p_{00} + p_{10}x + p_{01}y + p_{20}x^2 + p_{11}xy + p_{30}x^3 + p_{21}x^2y$$

where,

$p00 = -17.55\ (-19, -16.1)$

$p10 = 21.48\ (19.4, 23.56)$

$p01 = 2.49e-05\ (-8.84e-06, 5.864e-05)$

$p20 = -9.753\ (-10.73, -8.776)$

$p11 = -2.612e-05\ (-5.79e-05, 5.669e-06)$

$p30 = 1.592\ (1.441, 1.744)$

$p21 = 6.385e-06\ (-9.294e-07, 1.37e-05)$

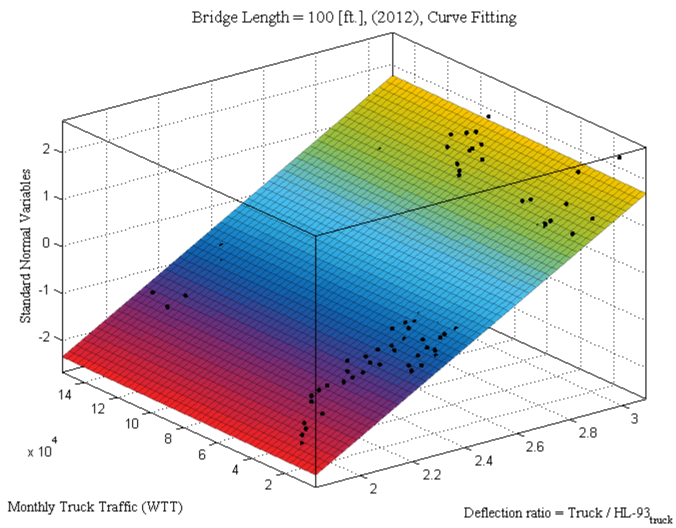


Figure 4.7C. 3D Probability Space Based on Maximum Monthly Deflection, Bridge Length=100 [ft.]

$$f(x,y) = p_{00} + p_{10}x + p_{01}y$$

where,

$p00 = -8.396\ (-8.691, -8.102)$

$p10 = 3.267\ (3.15, 3.383)$

$p01 = 4.339e-07\ (-4.915e-07, 1.359e-06)$

The formulations of the surfaces were obtained based on the linear polynomial surface, where Poly11 refers to the following parametric equation

$$f(x,y) = p_{00} + p_{10}x + p_{01}y \tag{4.21}$$

where,

$y =$ (daily, weekly, or monthly) truck traffic

$x =$ defelction ratio (truck deflection over deflection based on the Current ASSHTO criteria)

$f(x,y) =$ standard normal variables

$pij =$ surface parameters

where, Poly31 is

$$f(x,y) = p_{00} + p_{10}x + p_{01}y + p_{20}x^2 + p_{11}xy + p_{30}x^3 + p_{21}x^2y \tag{4.22}$$

Finally the result of the probability space can be summarized as follows

1- By increasing the number of trucks for each specific time period, the fitted surface has a positive inclination, which means that if the truck traffic is increased, the maximum deflection ratio is increased as well.

2- The general shape of the fitted surface tends to be a linear function, by considering the greater time periods.

3- Ability to determine the maximum deflection ratio (which is applicable for moment, shear, etc.) based on the actual volume of truck traffic for specific time period.

4- Determine the maximum deflection ratio for different time periods, based on curve fitting, instead of gigantic tables. And, generate a closed-form formula, which is more appropriate for engineering applications.

4.4.Target Reliability for Current Deflection Criteria

The target reliability is a design constraint that guarantees the required safety level of structures. In Load and Resistance Factor Design (LRFD), according to the desired safety level the load and resistance factors are computed. One of the safety measurement is the reliability index, which was defined by Basler (1961) and Cornell (1969) as:

$$\beta = -\Phi^{-1}(P_f) \tag{4.23}$$

where,

$\beta =$ reliability index
$\Phi^{-1} =$ inverse of the cumulative distribution of the standard normal function

To perform the reliability analysis of the structures, instead of using the deterministic capacity of the structure and applied load, it is required to utilize the statistical parameters of the load and/or resistance. The methods to determine the reliability index was explicitly described by Nowak and Collins (2013).

In this research, based on the reliability analysis, using the statistical parameters obtained from previous section and consideration of the current design criteria in AASHTO LRFD, the reliability indices of deflections was computed and tabulated as follows

Table 4.5. Target reliability for SLS due to the Vehicular Deflection

Bridge Length [ft.]	**Bias**	**Standard deviation**	**Deflection limit L/800 (in)**	**Reliability index**
60	1.58	0.39	0.9	0.24
100	1.71	0.40	1.5	0.03
150	1.65	0.37	2.25	0.13

Table 5 demonstrated the serviceability target reliability for bridges. As can be seen the obtained target reliability of the bridges for SLS is about zero.

4.5.Conclusion

In this research the statistical parameters of deflection of bridges were determined based on three different methods. The first method was introduced by Nowak in 1999 and also applied in this research. The second method, however, determined the statistical parameters based on daily, weekly, and monthly time periods, for any desired time. For the third method, the CDF of the deflection of the WIM data was plotted on the probability space, which discerned the CDF of the maximum daily, weekly, and monthly deflection on the probability space with respect to their volume truck traffics. By fitting a curve over the obtained results, it would be feasible to formulate the behavior of the statistical parameters of the deflection. Therefore, instead of the rough extrapolation and tedious effort to predict the statistical parameters based on the previous methods, the probability space can formulate the statistical parameter in more convenient way.

Another objective of this research was to determine the target reliability of bridges with consideration of serviceability limit state due to the vehicular deflection. To do so, based on the obtained statistical parameters of deflection, the target reliability index of the bridges with for SLS was demonstrated.

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Appendix A

Table A1. Normal Distribution of Probability of failure Medium Corrosion for bridge with 100 [ft.] length

Normal Distribution of Probability of failure Medium Corrosion			
year	**mean**	**Standard Variation**	**Reliability index**
0	16834	4725	3.563
10	15583	4663	3.342
20	15480	4653	3.327
30	15259	4628	3.297
40	15034	4640	3.240
50	14583	4552	3.204
60	14313	4523	3.165
70	14127	4533	3.116
80	13910	4513	3.082
90	13797	4505	3.063
100	13708	4496	3.049

Table A2. Normal Distribution of Probability of failure Low Corrosion for bridge with 100 [ft.] length

Normal Distribution of Probability of failure Low Corrosion			
year	**mean**	**Standard Variation**	**Reliability index**
0	16834	4725	3.563
0	16834	4725	3.563
20	15495	4660	3.325
30	15339	4657	3.294
40	15250	4642	3.285
50	15074	4608	3.271
60	14790	4553	3.248
70	14617	4568	3.200
80	14529	4573	3.177
90	14389	4548	3.164
100	14377	4571	3.146

High Corrosion, Bridge Length=100 [ft.]

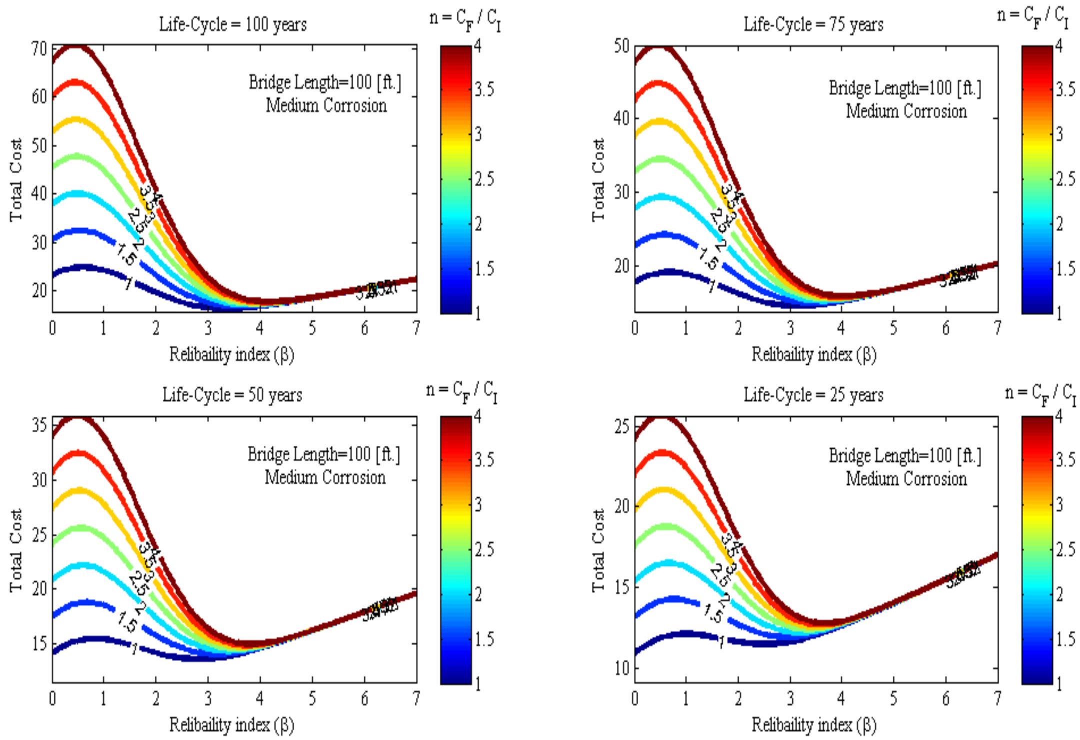

Figure A1. Target Reliability for Bridge with 100ft. length, Exposed to the Medium Corrosion Condition Using Contour of the Cost Ratio, n.

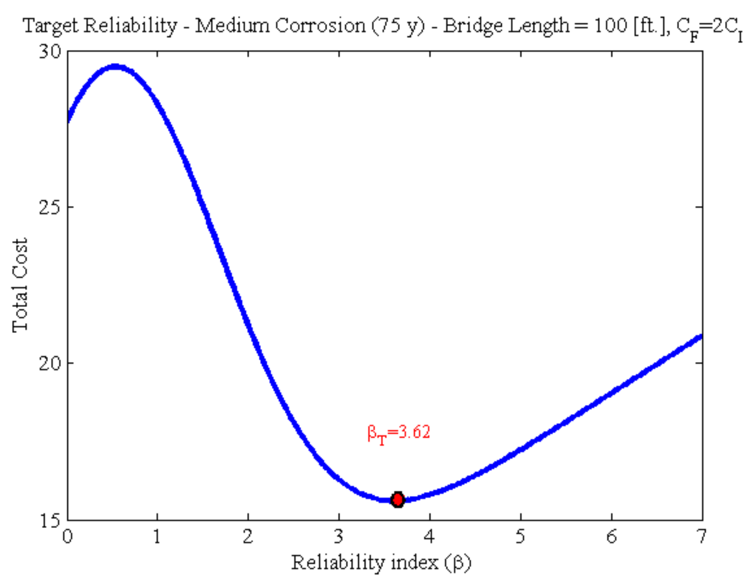

Figure A2. Target Reliability for Bridge with 100ft. length, Exposed to the Medium Corrosion Condition, Assumed $C_F = 2C_I$

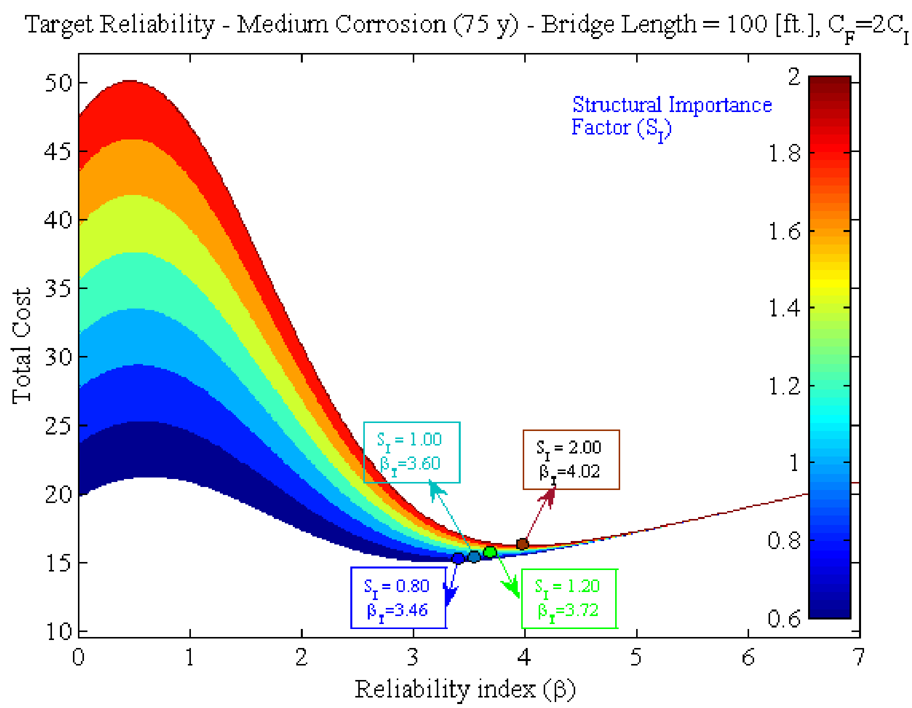

Figure A3. Target Reliability of Bridge with 100ft. Length, Exposed to the Medium Corrosion Condition, with Respect to the Different Structural Importance Factors, S_I, $C_F = 2C_I$

Table A3. Required Target Reliability for Different Structural Importance Factors

Bridge with 100ft. length, Exposed to the Medium Corrosion Condition, $C_F = 2C_I$				
Structural Importance Factor S_I	0.8	1.0	1.2	2.0
Target Reliability β_T	3.46	3.60	3.72	4.02

Low Corrosion, Bridge Length=100 [ft.]

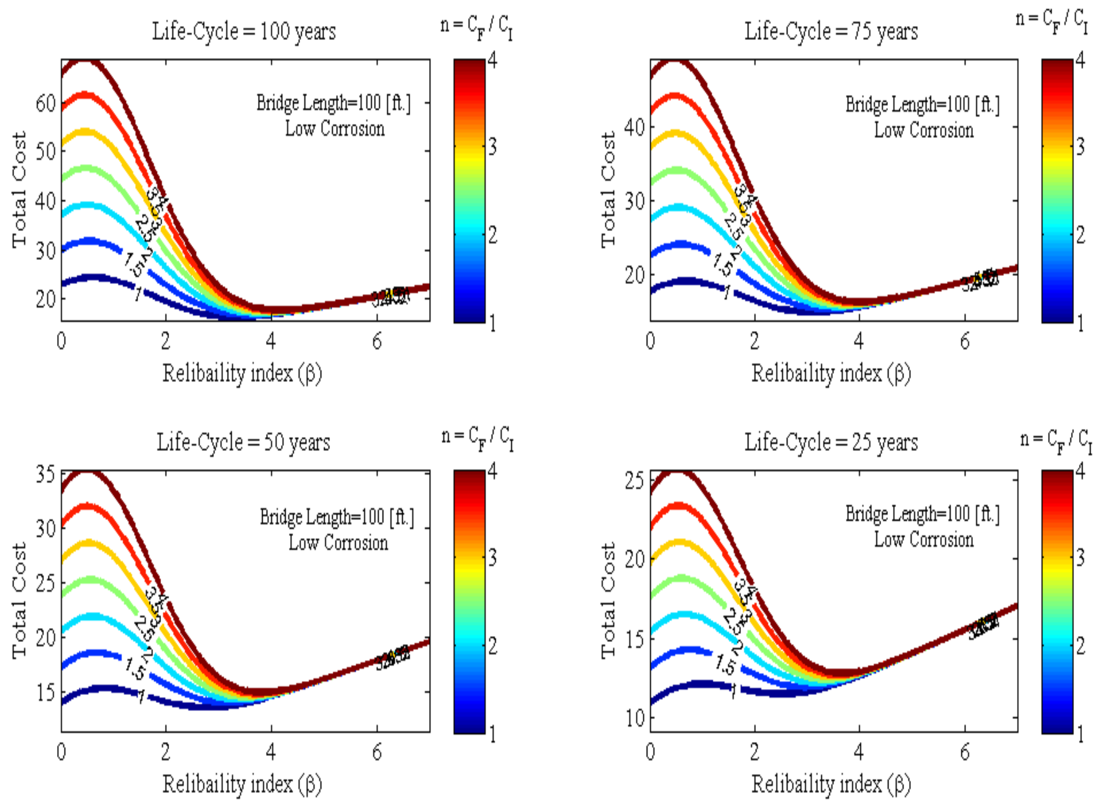

Figure A4. Target Reliability for Bridge with 100ft. length, Exposed to the Low Corrosion Condition Using Contour of the Cost Ratio, n.

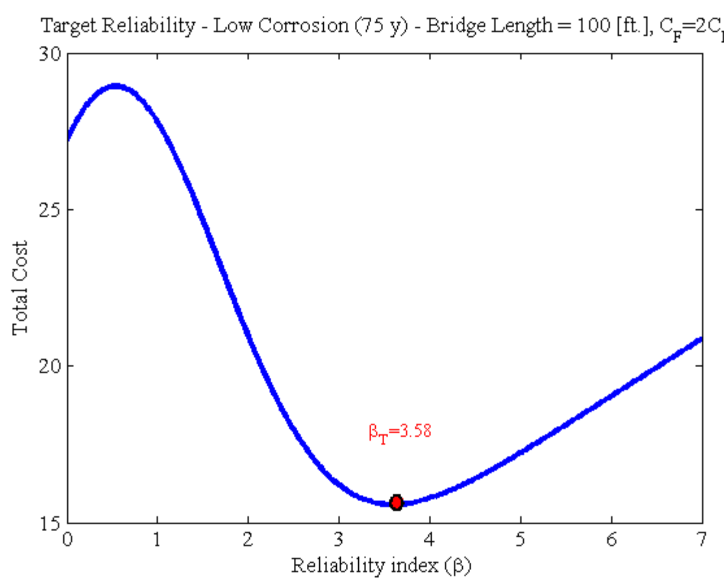

Figure A5. Target Reliability for Bridge with 100ft. length, Exposed to the Low Corrosion Condition, Assumed $C_F = 2C_I$

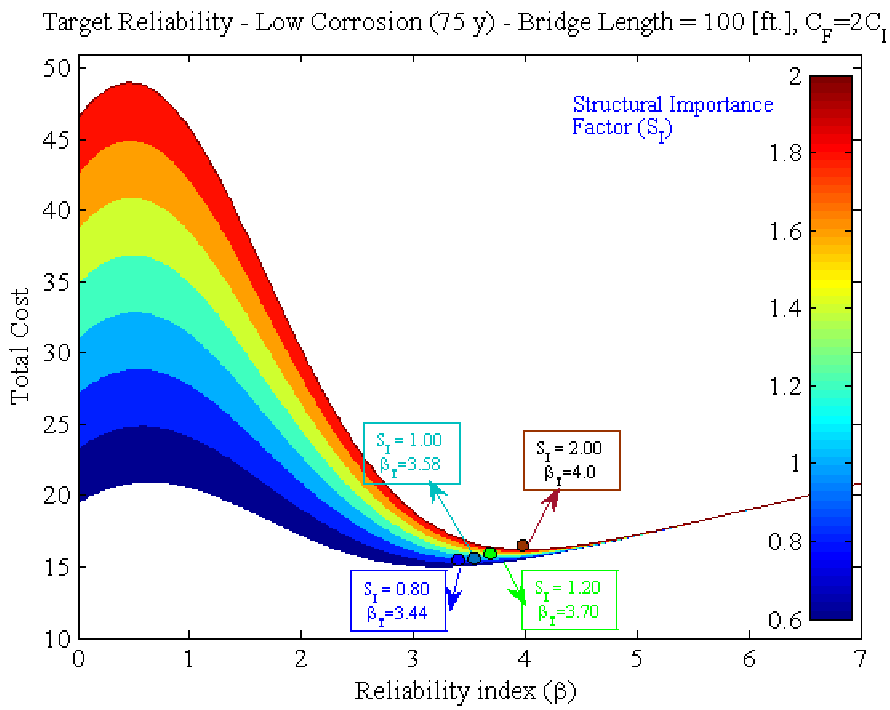

Figure A6. Target Reliability of Bridge with 100ft. Length, Exposed to the Low Corrosion Condition, with Respect to the Different Structural Importance Factors, S_I, $C_F = 2C_I$

Table A4. Required Target Reliability for Different Structural Importance Factors

Bridge with 100ft. Length, Exposed to the Low Corrosion Condition, $C_F = 2C_I$				
Structural Importance Factor S_I	0.8	1.0	1.2	2.0
Target Reliability β_T	3.44	3.58	3.70	4.00

Bridges length=60 ft.
Section = W27x146-fy=52

Table A5. Remaining the Flexural Capacity of W27x146, and Statistical parameters of the Live Load for bridge with 60 [ft.] length

Low corrosion based on Park model										
Year	10	20	30	40	50	60	70	80	90	100
Remaining resistance %	1	0.998	0.996	0.995	0.991	0.987	0.984	0.982	0.981	0.980
Medium corrosion based on Park model										
Year	10	20	30	40	50	60	70	80	90	100
Remaining resistance %	0.998	0.998	0.994	0.987	0.978	0.972	0.967	0.963	0.962	0.960
High corrosion based on Park model										
Year	10	20	30	40	50	60	70	80	90	100
Remaining resistance %	0.997	0.992	0.982	0.965	0.948	0.934	0.927	0.918	0.912	0.908
Statistical parameters of live load										
Year	10	20	30	40	50	60	70	80	90	100
Bias	1.22	1.225	1.23	1.235	1.24	1.242	1.245	1.25	1.255	1.26
Variation	0.11	0.11	0.11	0.11	0.10	0.10	0.10	0.10	0.10	0.10

Table A6. Normal Distribution of Probability of failure High Corrosion for bridge with 60 [ft.] length

Normal Distribution of Probability of failure High Corrosion			
year	**Mean**	**Standard Variation**	**Reliability index**
0	12160	3274	3.42
10	9980	3036	3.29
20	9784	3039	3.21
30	9476	3004	3.15
40	8962	2962	3.03
50	8465	2897	2.92
60	8077	2856	2.83
70	7860	2837	2.77
80	7571	2810	2.69
90	7376	2810	2.62
100	7218	2800	2.57

Table A7. Normal Distribution of Probability of failure Medium Corrosion for bridge with 60 [ft.] length

Normal Distribution of Probability of failure Medium Corrosion			
year	**Mean**	**Standard Variation**	**Reliability index**
0	12160	3274	3.42
10	10000	3050	3.27
20	9947	3044	3.26
30	9549	3051	3.22
40	9776	3029	3.15
50	9285	2957	3.13
60	9113	2935	3.10
70	8918	2944	3.03
80	8782	2920	3.01
90	8718	2915	2.99
100	8621	2921	2.95

Table A8. Normal Distribution of Probability of failure Low Corrosion for bridge with 60 [ft.] length

Normal Distribution of Probability of failure Low Corrosion			
year	**Mean**	**Standard Variation**	**Reliability index**
0	12160	3274	3.42
10	10184	3038	3.31
20	9946	3038	3.27
30	9856	3051	3.23
40	9776	3036	3.22
50	9628	2991	3.22
60	9514	2971	3.20
70	9386	2975	3.15
80	9303	2968	3.13
90	9224	2969	3.10
100	9154	2972	3.08

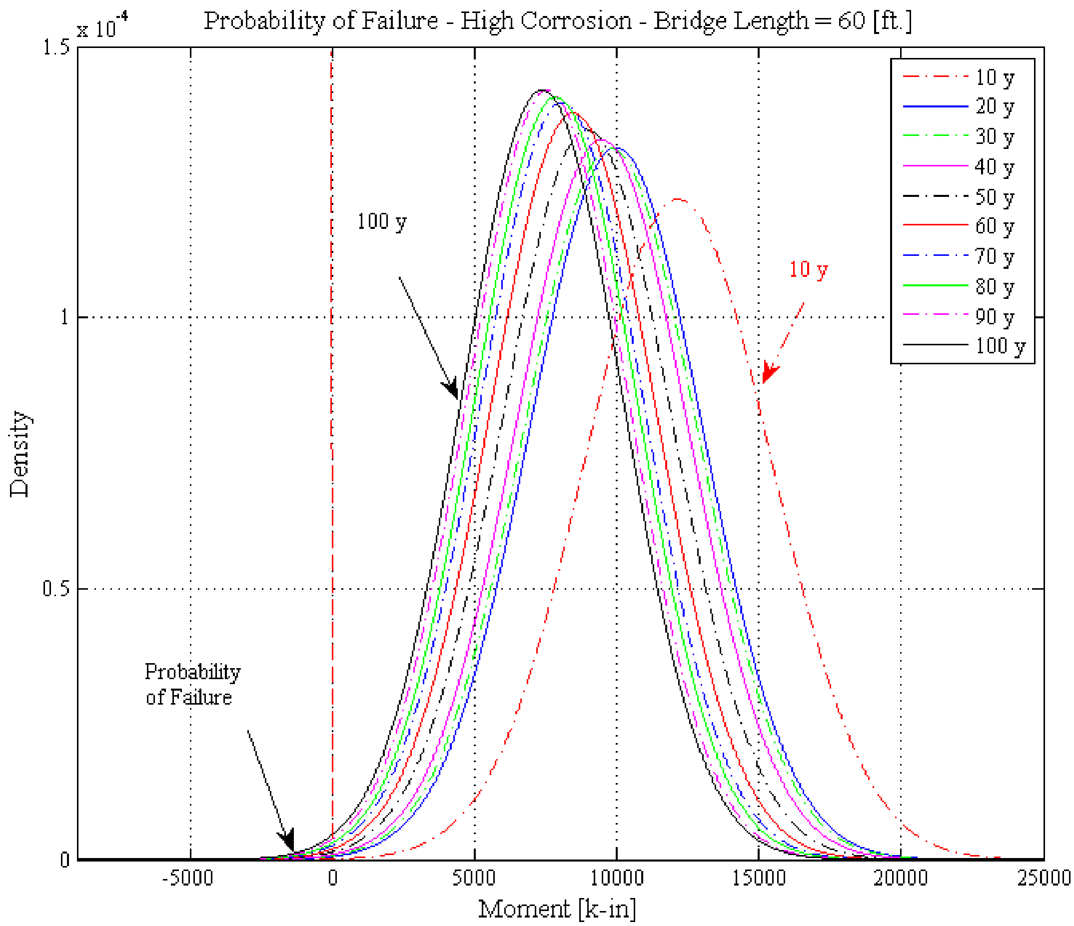

Figure A7. PDF of Probability of Girder Failure due to the Strength Limit State exposed to the High Corrosion Condition, Bridge Length =40 [ft.]

High Corrosion, Bridge Length=60 [ft.]

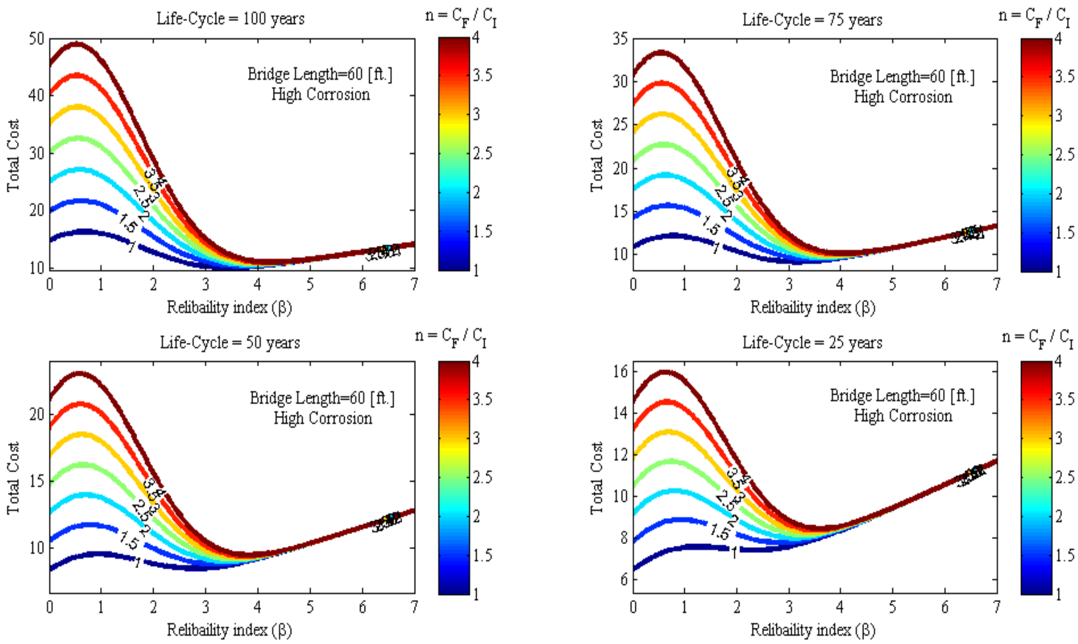

Figure A8. Target Reliability for Bridge with 60ft. length, Exposed to the High Corrosion Condition Using Contour of the Cost Ratio, n.

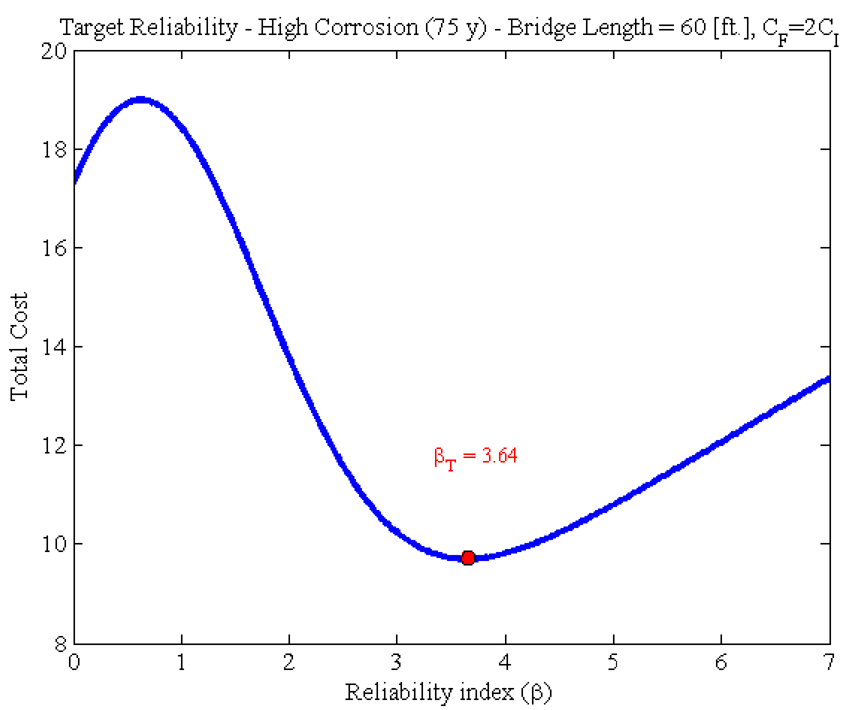

Figure A9. Target Reliability for Bridge with 60ft. length, Exposed to the High Corrosion Condition, Assumed $C_F = 2C_I$

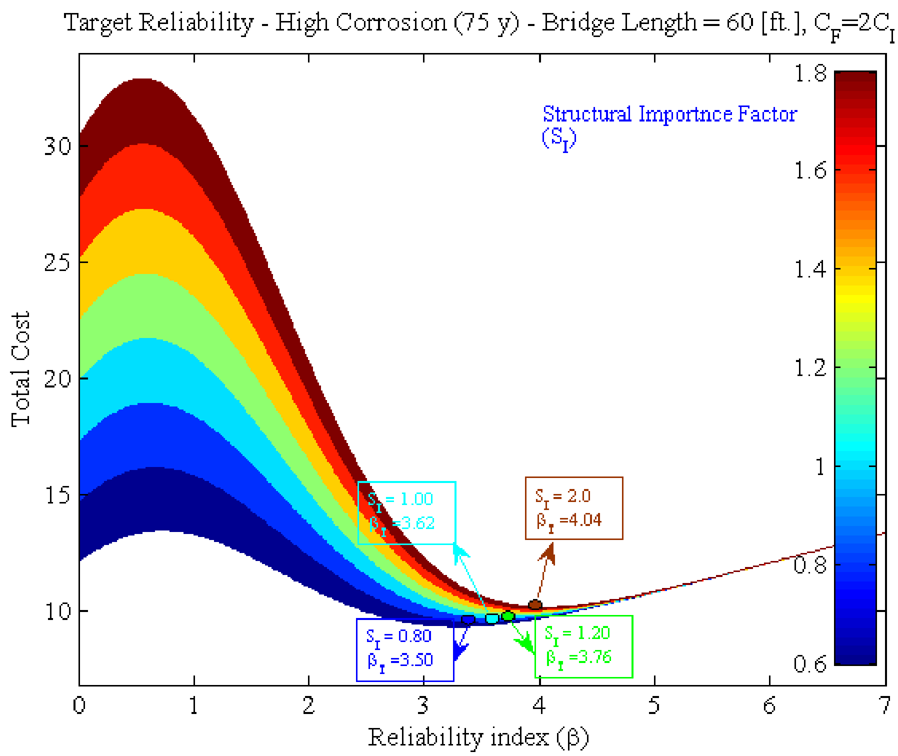

Figure A10. Target Reliability of Bridge with 60ft. Length, Exposed to the High Corrosion Condition, with Respect to the Different Structural Importance Factors, S_I, $C_F = 2C_I$

Table A9. Required Target Reliability for Different Structural Importance Factors

Bridge with 60ft. Length, Exposed to the High Corrosion Condition, $C_F = 2C_I$				
Structural Importance Factor S_I	0.8	1.0	1.2	2.0
Target Reliability β_T	3.50	3.62	3.76	4.04

Medium Corrosion, Bridge Length=60 [ft.]

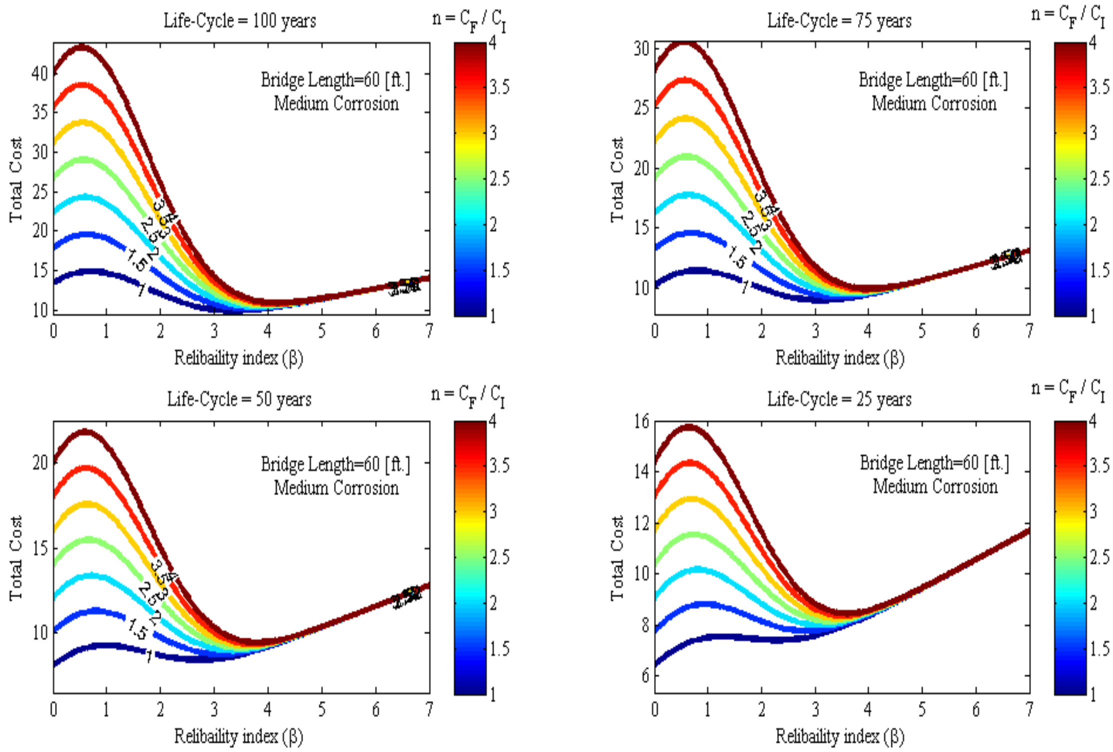

Figure A11. Target Reliability for Bridge with 60ft. length, Exposed to the Medium Corrosion Condition Using Contour of the Cost Ratio, n.

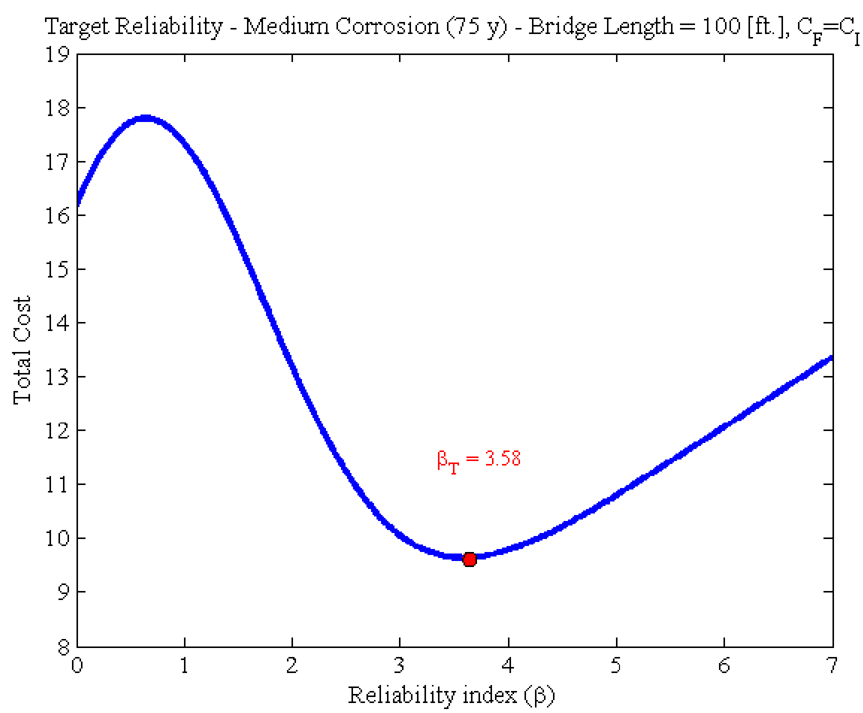

Figure A12. Target Reliability for Bridge with 60ft. length, Exposed to the Medium Corrosion Condition, Assumed $C_F = 2C_I$

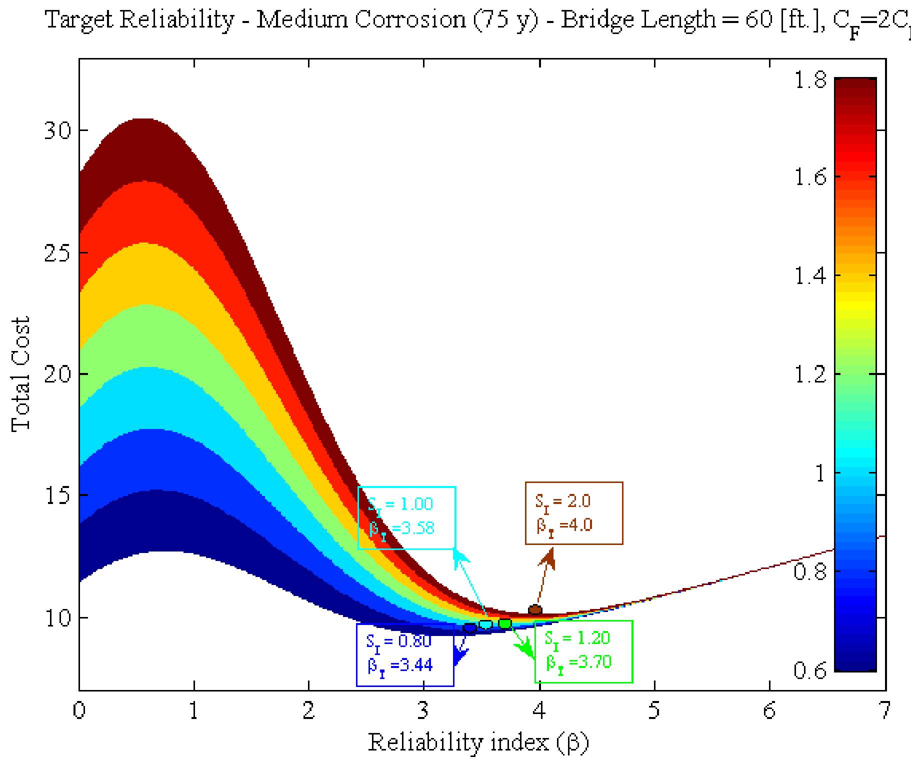

Figure A13. Target Reliability of Bridge with 60ft. Length, Exposed to the Medium Corrosion Condition, with Respect to the Different Structural Importance Factors, S_I, $C_F = 2C_I$

Table A10. Required Target Reliability for Different Structural Importance Factors

Bridge with 60ft. Length, Exposed to the Medium Corrosion Condition, $C_F = 2C_I$				
Structural Importance Factor S_I	0.8	1.0	1.2	2.0
Target Reliability β_T	3.44	3.58	3.70	4.00

Low Corrosion, Bridge Length=60 [ft.]

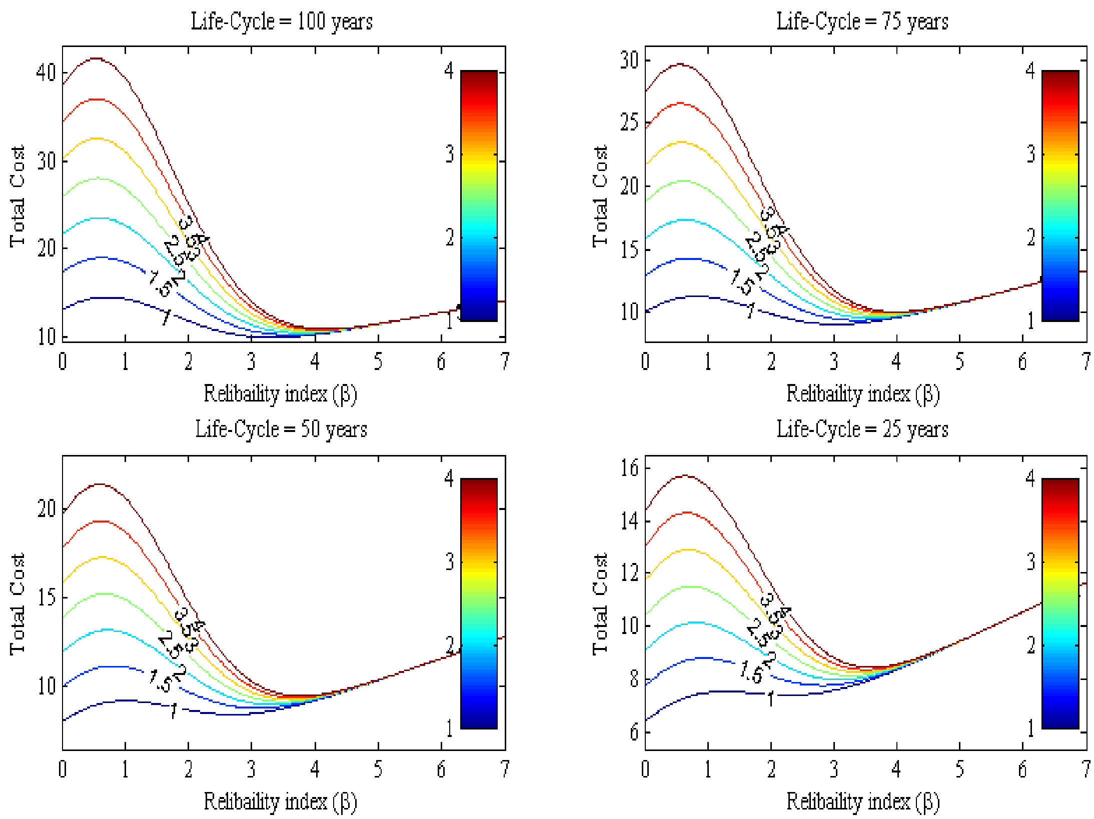

Figure A14. Target Reliability for Bridge with 60ft. length, Exposed to the Low Corrosion Condition Using Contour of the Cost Ratio, n.

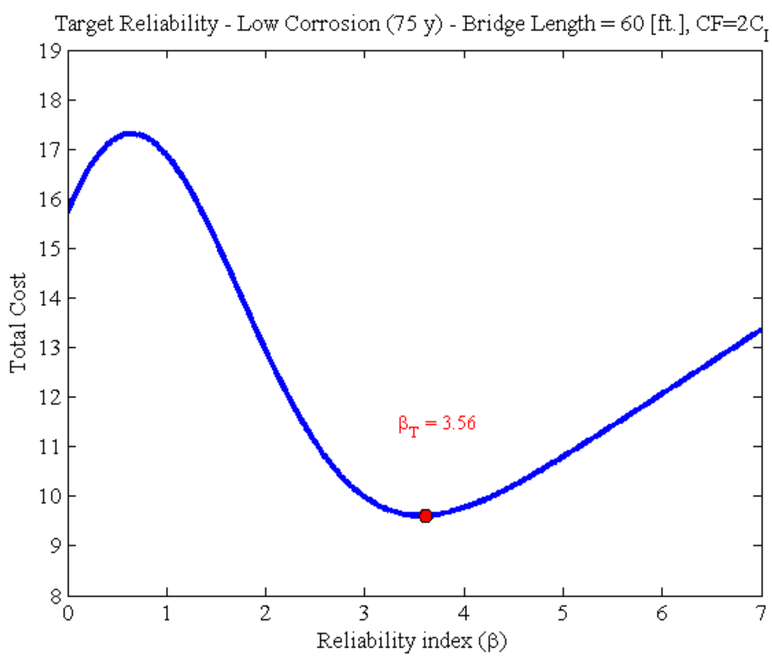

Figure A15. Target Reliability for Bridge with 60ft. length, Exposed to the Low Corrosion Condition, Assumed $C_F = 2C_I$

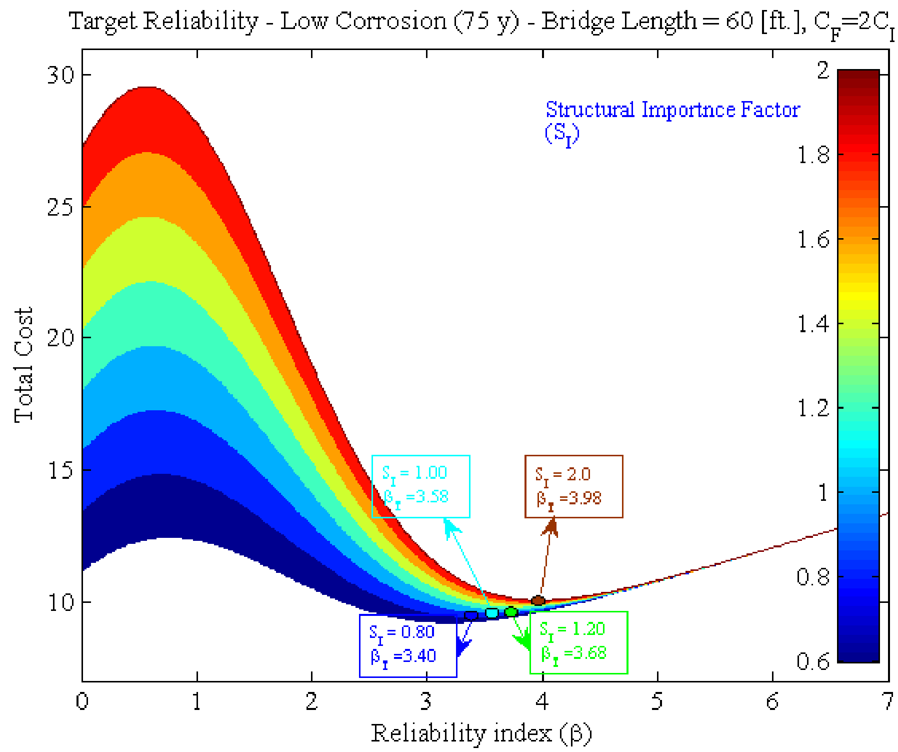

Figure A16. Target Reliability of Bridge with 60ft. Length, Exposed to the High Corrosion Condition, with Respect to the Different Structural Importance Factors, S_I, $C_F = 2C_I$

Table A11. Required Target Reliability for Different Structural Importance Factors

Bridge with 60ft. Length, Exposed to the Low Corrosion Condition, $C_F = 2C_I$				
Structural Importance Factor S_I	0.8	1.0	1.2	2.0
Target Reliability β_T	3.40	3.58	3.68	3.98

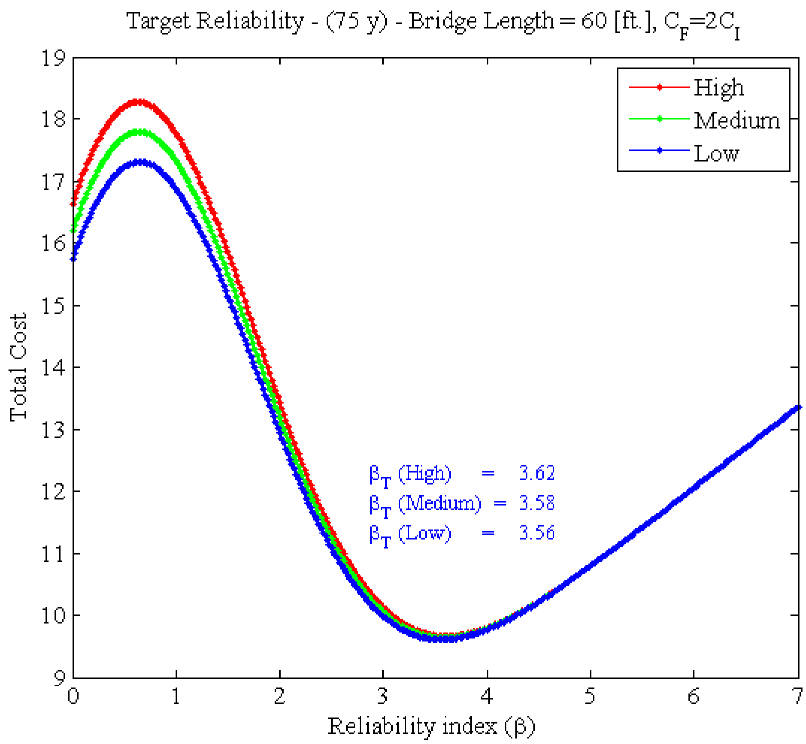

Figure A17. Target Reliability of Bridge with 60ft. Length, Exposed to the high, Medium, and Low Corrosion Conditions, with Respect to the Different Structural Importance Factors, S_I, $C_F = 2C_I$

Table A12. Required Target Reliability for Different Corrosion Conditions

Bridge with 60ft. length, Exposed to the Different Corrosion Conditions, $C_F = 2C_I$, $S_I = 1.0$			
Corrosion Level	Low	Medium	High
Required Target Reliability β_T	3.56	3.58	3.62

Bridges length=40 ft.
Section = W21x101, Fu=50, bias=1.07, v=0.15

Table A13. Remaining the Flexural Capacity of W21x101, and Statistical parameters of the Live Load for bridge with 40 [ft.] length

Low corrosion based on Park model										
Year	10	20	30	40	50	60	70	80	90	100
Remaining resistance %	1	0.998	0.996	0.993	0.989	0.983	0.980	0.978	0.977	0.976
Medium corrosion based on Park model										
Year	10	20	30	40	50	60	70	80	90	100
Remaining resistance %	0.998	0.997	0.991	0.984	0.972	0.966	0.959	0.955	0.953	0.952
High corrosion based on Park model										
Year	10	20	30	40	50	60	70	80	90	100
Remaining resistance %	0.996	0.990	0.978	0.957	0.936	0.919	0.910	0.899	0.891	0.888
Statistical parameters of live load										
Year	10	20	30	40	50	60	70	80	90	100
Bias	1.18	1.19	1.20	1.21	1.22	1.225	1.23	1.233	1.235	1.24
Variation	0.12	0.12	0.11	0.11	0.10	0.10	0.10	0.10	0.10	0.10

Table A14. Normal Distribution of Probability of failure High Corrosion for bridge with 40 [ft.] length

Normal Distribution of Probability of failure High Corrosion			
year	**Mean**	**Standard Variation**	**Reliability index**
0	6030	1700	3.55
10	5408	1637	3.30
20	5046	1591	3.17
30	5003	1598	3.13
40	4699	1565	3.00
50	4351	1517	2.86
60	4079	1496	2.73
70	3925	1487	2.63
80	3755	1475	2.54
90	3635	1468	2.48
100	3564	1460	2.44

Table A15. Normal Distribution of Probability of failure Medium Corrosion for bridge with 40 [ft.] length

Normal Distribution of Probability of failure Medium Corrosion			
year	**Mean**	**Standard Variation**	**Reliability index**
0	6030	1700	3.55
10	5433	1640	3.31
20	5362	1638	3.27
30	5230	1604	3.26
40	5182	1615	3.21
50	4864	1559	3.12
60	4752	1558	3.04
70	4618	1544	2.99
80	4544	1540	2.95
90	4517	1546	2.92
100	4457	1543	2.88

Table A16. Normal Distribution of Probability of failure Low Corrosion for bridge with 40 [ft.] length

Normal Distribution of Probability of failure Low Corrosion			
year	**mean**	**Standard Variation**	**Reliability index**
0	6030	1700	3.55
10	5455	1644	3.32
20	5389	1641	3.28
30	5301	1616	3.27
40	5207	1621	3.23
50	5098	1581	3.22
60	5011	1579	3.17
70	4921	1577	3.12
80	4878	1573	3.10
90	4850	1574	3.08
100	4811	1575	3.05

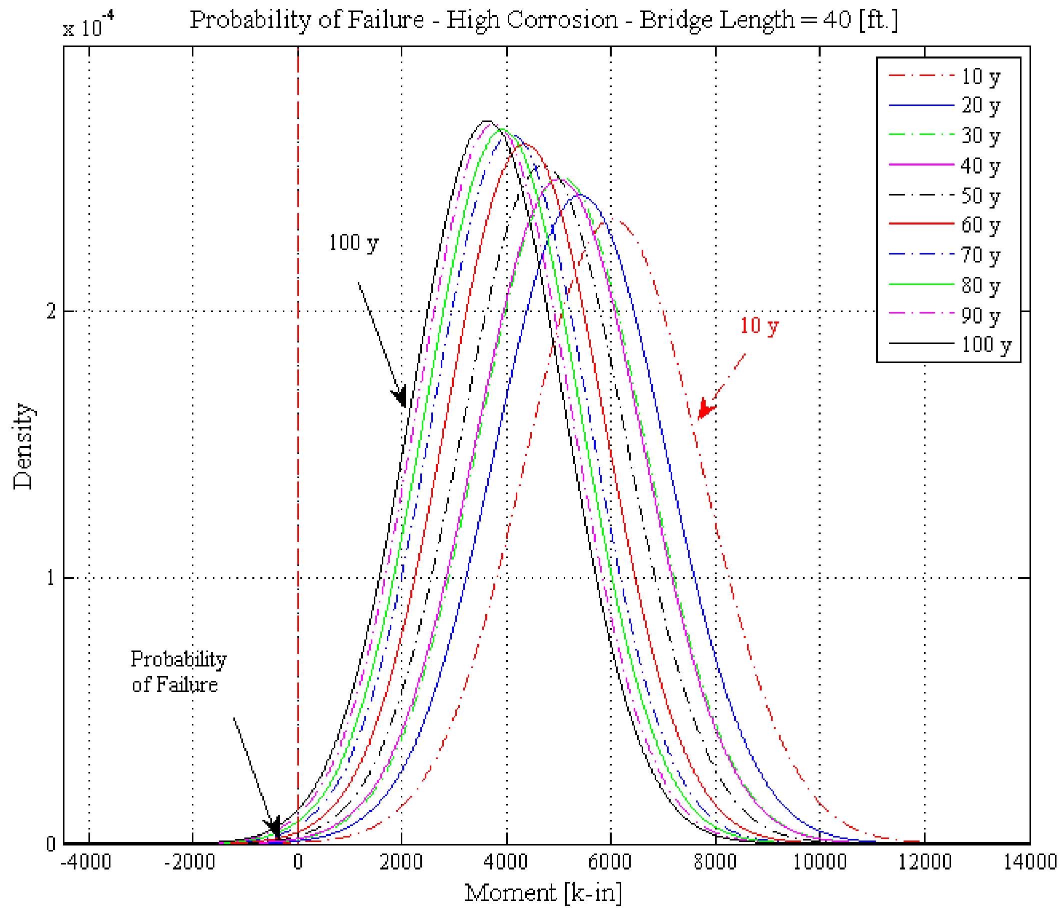

Figure A18. PDF of Probability of Girder Failure due to the Strength Limit State exposed to the High Corrosion Condition, Bridge Length =40 [ft.]

High Corrosion, Bridge Length=40 [ft.]

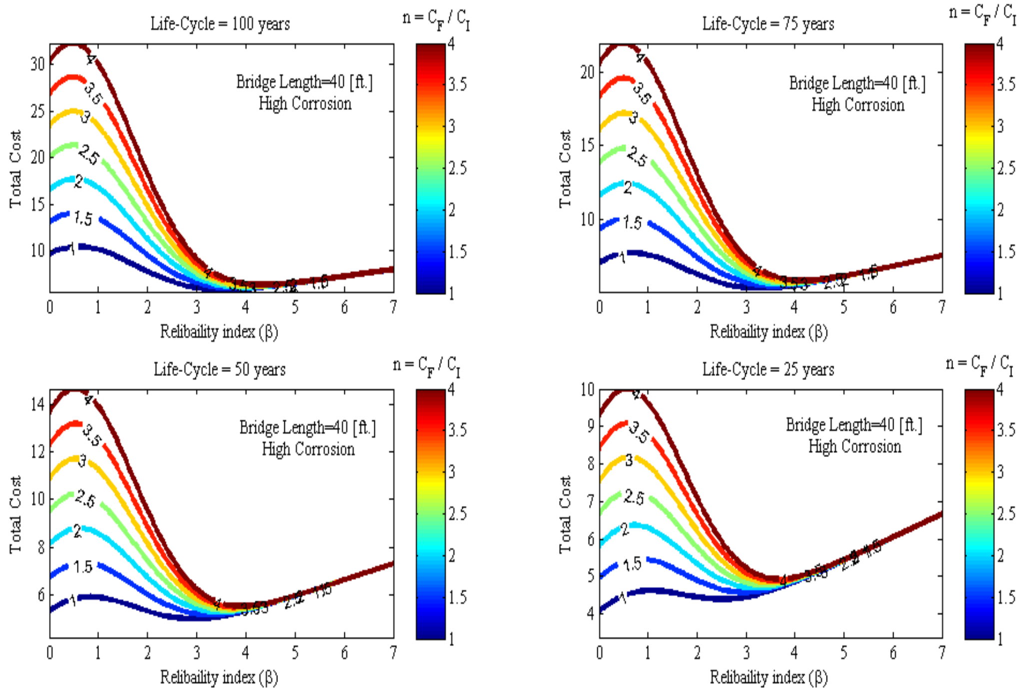

Figure A19. Target Reliability for Bridge with 40ft. length, Exposed to the High Corrosion Condition Using Contour of the Cost Ratio, n.

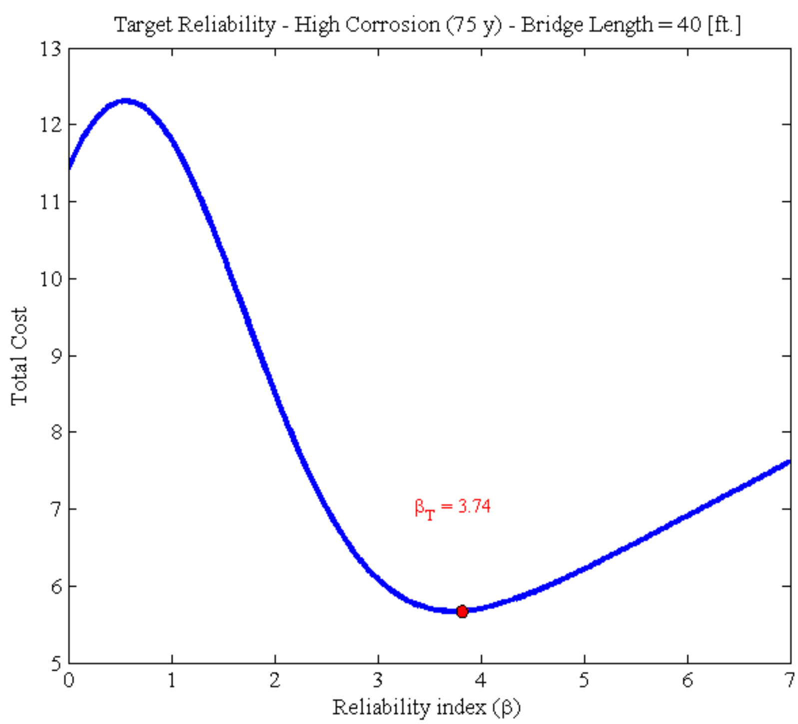

Figure A20. Target Reliability for Bridge with 100ft. length, Exposed to the High Corrosion Condition, Assumed $C_F = 2C_I$

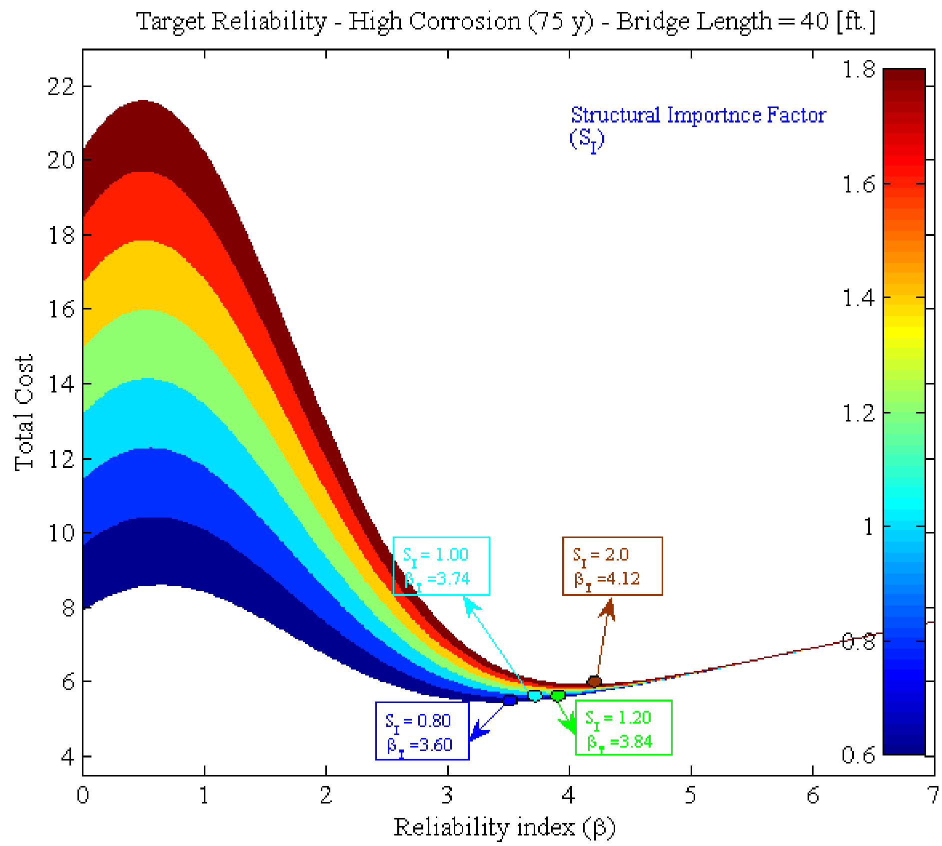

Figure A21. Target Reliability of Bridge with 40ft. Length, Exposed to the High Corrosion Condition, with Respect to the Different Structural Importance Factors, S_I, $C_F = 2C_I$

Table A17. Required Target Reliability for Different Structural Importance Factors

Bridge with 40ft. Length, Exposed to the High Corrosion Condition, $C_F = 2C_I$				
Structural Importance Factor S_I	0.8	1.0	1.2	2.0
Target Reliability β_T	3.60	3.74	3.84	4.12

Medium Corrosion, Bridge Length=40 [ft.]

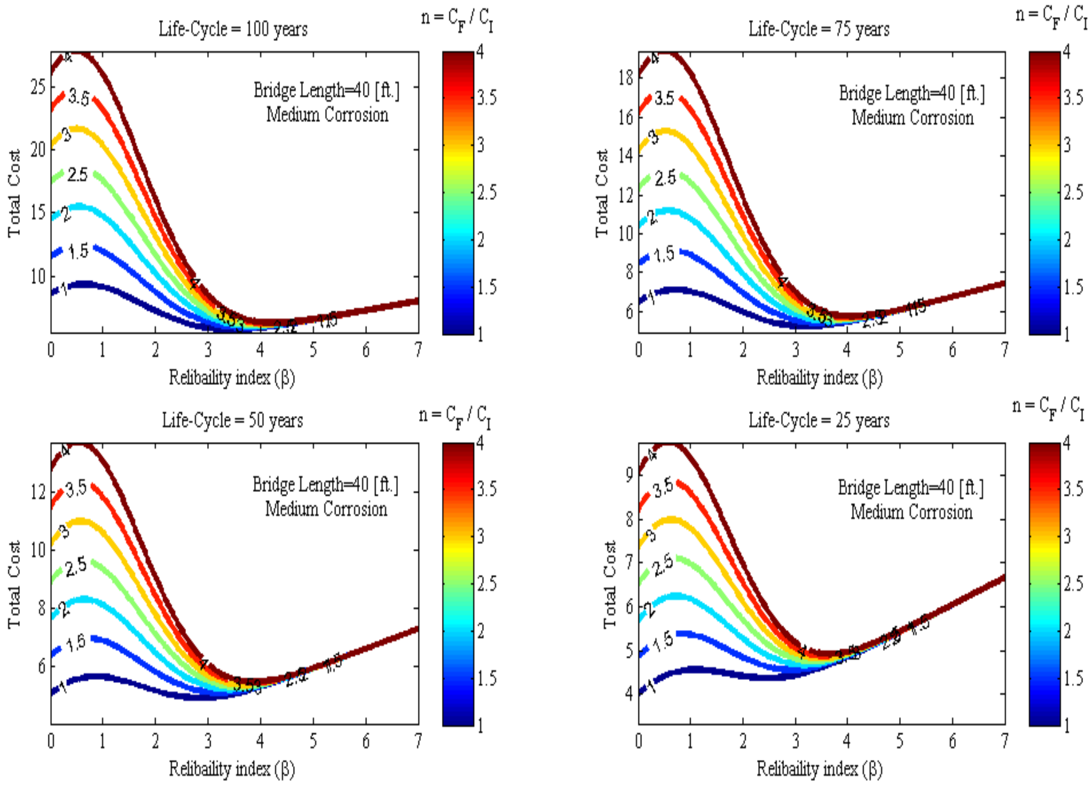

Figure A22. Target Reliability for Bridge with 40ft. Length, Exposed to the Medium Corrosion Condition Using Contour of the Cost Ratio, n.

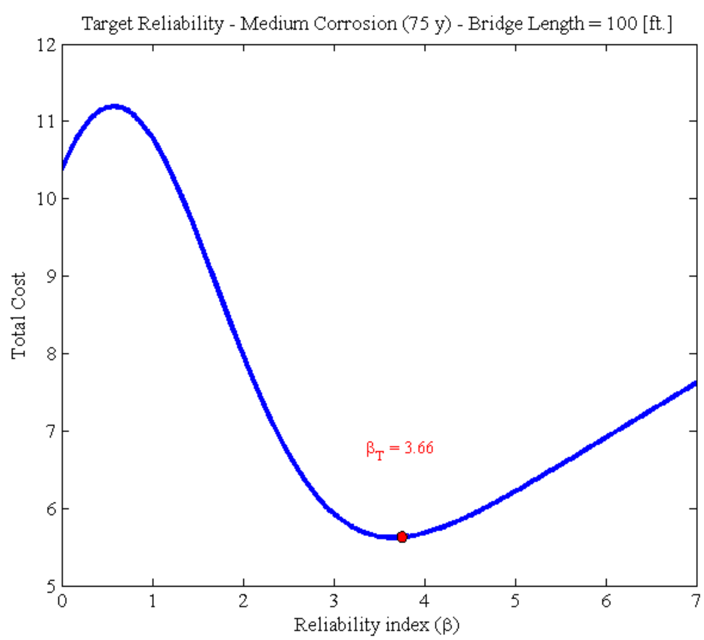

Figure A23. Target Reliability for Bridge with 40ft. length, Exposed to the Low Corrosion Condition, Assumed $C_F = 2C_I$

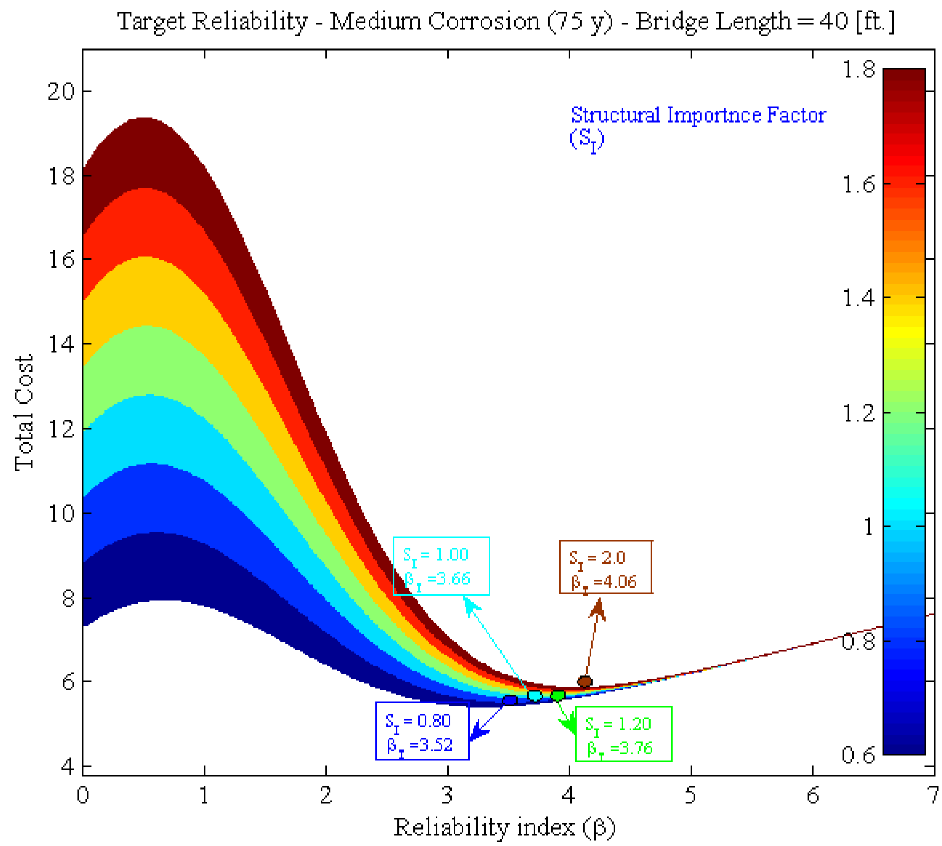

Figure A24. Target Reliability of Bridge with 40ft. Length, Exposed to the Medium Corrosion Condition, with Respect to the Different Structural Importance Factors, S_I, $C_F = 2C_I$

Table A18. Required Target Reliability for Different Structural Importance Factors

Bridge with 40ft. Length, Exposed to the Medium Corrosion Condition, $C_F = 2C_I$				
Structural Importance Factor S_I	0.8	1.0	1.2	2.0
Target Reliability β_T	3.52	3.66	3.76	4.06

Low Corrosion, Bridge Length=40 [ft.]

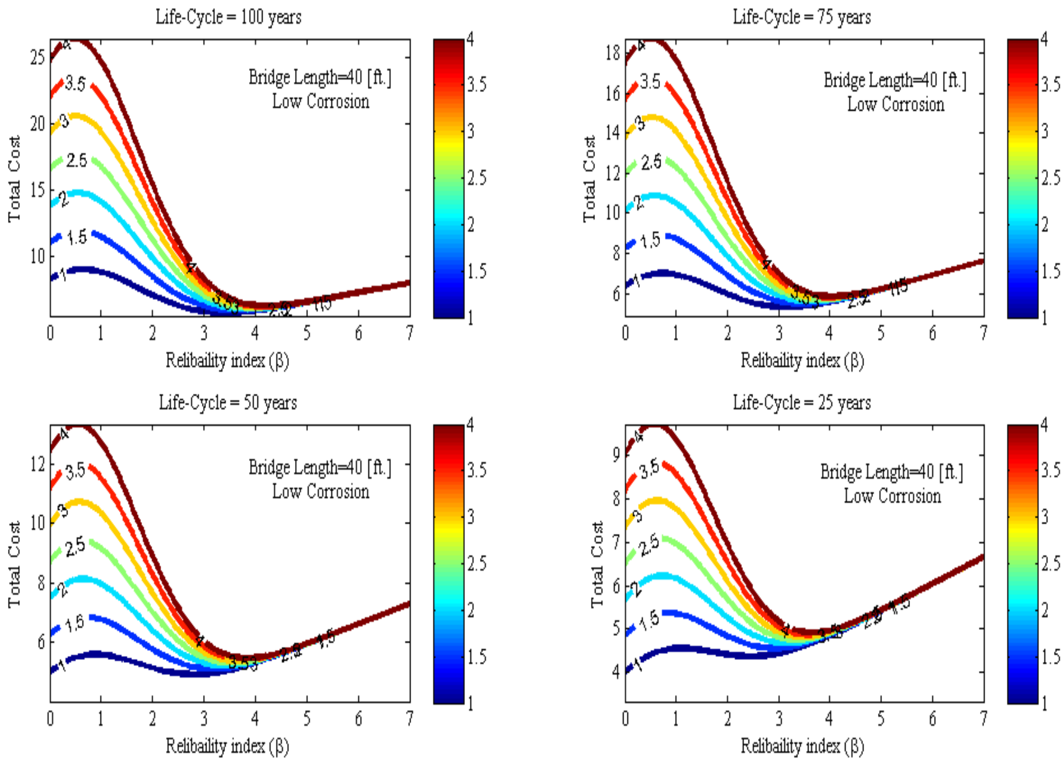

Figure A25. Target Reliability for Bridge with 40ft. Length, Exposed to the High Corrosion Condition Using Contour of the Cost Ratio, n.

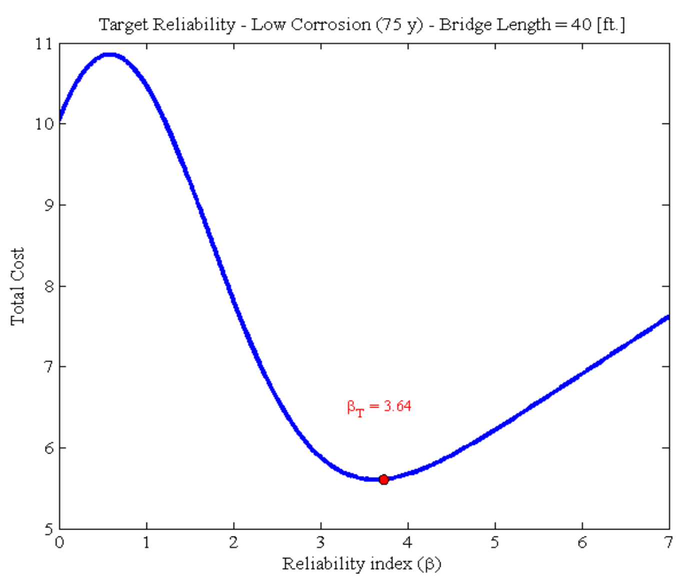

Figure A26. Target Reliability for Bridge with 100ft. length, Exposed to the Low Corrosion Condition, Assumed $C_F = 2C_I$

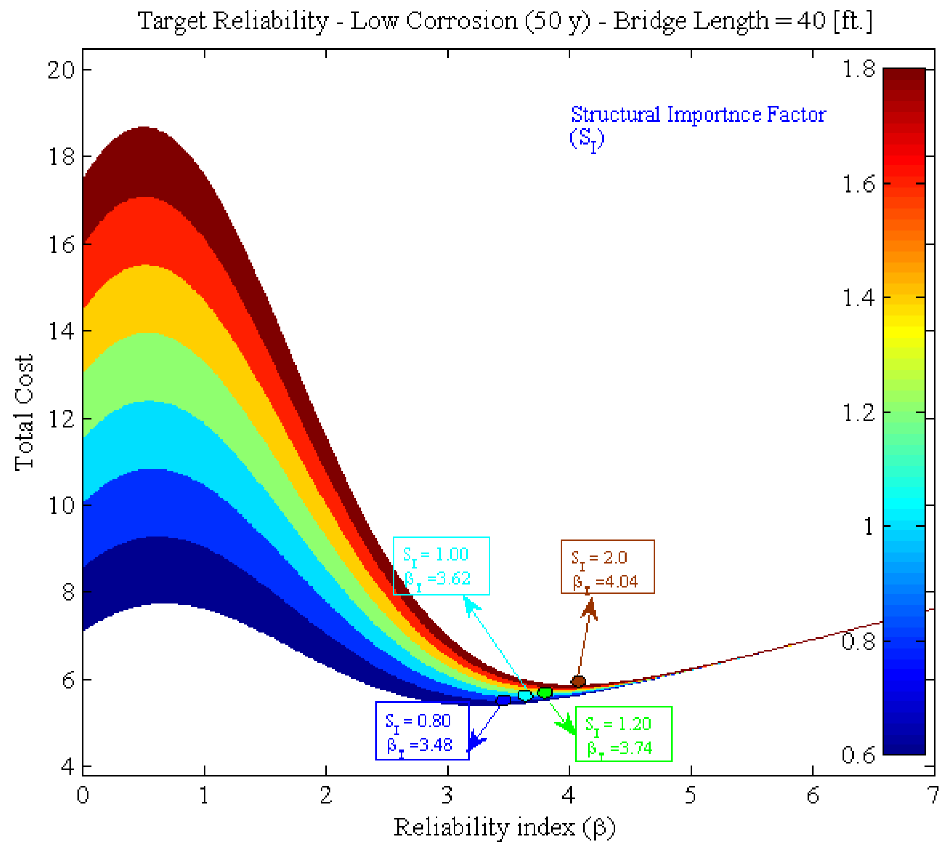

Figure A27. Target Reliability of Bridge with 40ft. Length, Exposed to the Low Corrosion Condition, with Respect to the Different Structural Importance Factors, S_I, $C_F = 2C_I$

Table A19 Required Target Reliability for Different Structural Importance Factors

Bridge with 40ft. Length, Exposed to the Low Corrosion Condition, $C_F = 2C_I$				
Structural Importance Factor S_I	0.8	1.0	1.2	2.0
Target Reliability β_T	3.48	3.62	3.74	4.04

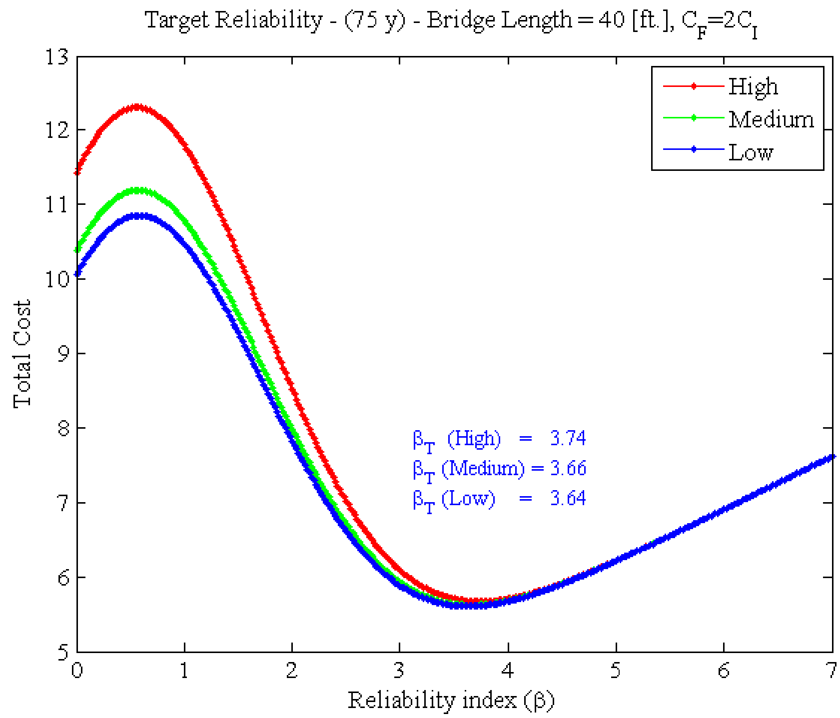

Figure A28. Target Reliability of Bridge with 40ft. Length, Exposed to the high, Medium, and Low Corrosion Conditions, with Respect to the Different Structural Importance Factors, S_I, $C_F = 2C_I$

Table A20. Required Target Reliability for Different Corrosion Conditions

Bridge with 40ft. length, Exposed to the Different Corrosion Conditions, $C_F = 2C_I$, $S_I = 1.0$			
Corrosion Level	Low	Medium	High
Required Target Reliability β_T	3.64	3.66	3.74

Appendix B

Table B1. Reliability indices for the moment capacity based on the Tunnel Manual, $\varphi = 0.95$

	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L
Load Factor	1.25	1.5	1.75	1.75	1	1.35	1.35	1.35	1.5	1.5	1.35	1.35
	0.9	0.65	1.75	1.75	1	0.9	0.9	0.9	0.75	0.75	0.9	0.9
λ_Q	1.05	1.03	1.25	1.25	0.9	1	0.95	0.95	1	0.95	1	1
V_Q	0.1	0.08	0.18	0.18	0.15	0.14	0.15	0.15	0.15	0.15	0.12	0.12

λ_R	V_R	ϕ
1.140	0.080	0.95

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Segment	member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1002	-0.297	-4E-04	0.001	4E-04	1.32	-10	-1.3	1.22	11.9	5.92	0.3	0.36	10.13	2.43	22.37	**5.05**
I	1021	-0.061	-0.008	0.006	0.002	1.09	0.263	0.58	1.47	8.3	4.15	0.48	0.77	16	1.41	24.98	**5.09**
II	1101	-0.061	4E-04	0.006	0.002	1.09	0.263	-0.59	1.63	8.29	4.15	0.77	0.48	16.14	1.41	25.19	**5.10**
III	1090	-0.011	0.006	0.054	0.018	0.4	-0.1	-0.04	0.67	4.58	2.29	-0.24	0.02	7.729	0.77	12.05	**4.84**
IV	1032	-0.011	-0.008	0.054	0.018	0.4	-0.1	0	0.08	4.6	2.29	0.02	-0.24	7.175	0.77	11.23	**4.73**

100 S. South Charles Street

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1002	-0.3	0	0	0	1.2	-11.6	-2.5	1.4	11.6	5.7	0.2	0.3	8.01	2.53	20.01	**4.9**
I	1007	-0.1	0	0	0	1	-4.4	-1.9	1.1	6.6	3.3	0.1	0.6	7.875	1.28	14.98	**5.23**
II	1103	-0.3	0	0	0	1.2	-8.1	-1.3	0.6	12.5	6.2	0.2	0.2	12.03	2.37	24.25	**5.06**
III	1089	0	0	0	0	0.6	0.1	0	0.1	4.2	2.1	0	0	6.93	0.7	10.86	**4.85**
IV	1031	0	0	0	0	0.4	0.1	0	1.1	5.2	2.6	0	-0.1	9.075	0.88	14.35	**5.01**

Bromo Seltzer Tower

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	0.3	0	0	0	-1.1	18.7	2.3	-1	38.1	-9.7	-0.2	-0.2	45.96	6.44	76.98	**4.65**
I	1006	0.2	0	0	0	-0.9	11.1	0.9	-0.8	35.2	-7	-0.1	-0.3	38.29	5.6	64.01	**4.53**
II	1116	0.2	0	0	0	-0.9	11.1	1.8	-0.7	35.3	-7	-0.3	-0.1	38.49	5.61	64.26	**4.53**
III	1089	0	0	0	0	0.6	0.4	0	1.2	22.7	4.7	0.1	0	29.34	3.48	46.31	**4.58**
IV	1031	0	0	0	0	0.4	0.2	0	0	24.2	5.2	0	-0.1	29.7	3.71	47.03	**4.49**

Eastern Ave 1401 Garage

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1006	0.812	0.316	1.655	0.546	-0.92	-34.1	-0.04	1.18	26.4	13.1	0	0	8.421	6.48	36.39	**4.61**
I	1016	0.013	0.037	0.737	0.243	-0.49	14.85	1.93	-0.72	10.6	5.3	0	0	30.43	2.73	46.95	**4.95**
II	1090	0.335	0.388	2.48	0.818	-2.26	-17.5	-2.34	4.23	44.7	22.3	0	0	54.44	7.85	99.98	**5.29**
III	1087	0.02	0.481	2.984	0.985	-2.53	-8.6	-1.75	3.16	41.9	20.8	0	0	58.28	7.09	100.8	**5.25**
IV	1035	0.441	0.612	2.984	0.985	-2.53	-8.6	0.18	1.33	41.9	20.8	0	0	57.16	7.08	99.02	**5.22**

Grudelsky

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	-0.225	6E-04	0.003	0.001	0.7	-7.99	-0.58	1.18	5.36	2.68	0.43	0.43	2.256	1.44	8.562	**4.65**
I	1014	-0.174	0	0.004	0.001	0.27	-6.38	-0.48	0.82	4.48	2.24	0.37	0.08	1.501	1.17	6.499	**4.57**
II	1108	-0.174	9E-04	0.004	0.001	0.27	-6.38	-0.17	1.08	4.48	2.24	0.08	0.37	1.751	1.17	6.874	**4.65**
III	1089	-0.007	0.006	0.054	0.018	0.24	-0.15	-0.03	0.14	0.27	0.13	-0.14	-0	0.665	0.06	0.948	**4.17**
IV	1031	-0.009	-0.004	0.054	0.018	0.23	-0.12	-0.01	0	0.68	0.34	-0	-0.2	1.151	0.12	1.672	**4.18**

Holiday Inn - Inner Harbor Hotel

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	0.9	0	0	0	0.1	42.3	1.8	-0.3	-15	-7.6	0	0	20.83	6.43	43.27	**3.85**
I	1016	0.3	0	0	0	1.1	18.2	0.5	0.5	6.1	3	0	0	28.93	2.75	42.49	**4.43**
II	1105	0.3	0	0	0	1.2	21.6	1.1	-0.2	3.7	1.8	0	0	28.22	3.09	40.85	**4.03**
III	1076	-0.6	0	-0.1	0	0	-14.1	-0.5	-0.2	27.8	13.9	-0.6	0	25.99	5.02	50.97	**4.93**
IV	1032	-1	0	0	0	1.7	-21.6	-1.2	1.1	29.6	14.8	0	-0.2	23.59	5.78	51.86	**4.95**

Market Center West Apts

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	0.9	0	0	0	0.1	42.3	1.3	-0.6	-15	-7.6	0	0	20.55	6.43	42.99	**3.85**
I	1016	0.3	0	0	0	1.1	18.2	1	-0.3	6.1	3	0	0	28.17	2.75	41.5	**4.40**
II	1093	-0.9	0	0	0	0.5	-26.7	-2.1	1.7	32.4	16.2	-0.3	0	22.21	6.56	53.25	**4.88**
III	1076	-0.6	0	-0.1	0	0	-14.1	-0.9	1.6	27.8	13.9	-0.6	0	27.7	5.03	53.44	**5.00**
IV	1032	-1	0	0	0	1.7	-21.6	-0.5	-0.2	29.6	14.8	0	-0.2	22.35	5.78	50.11	**4.90**

Marriot Hotel

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1005	-0.819	0.339	1.941	0.64	-1.69	-34.8	0	-0.14	29.3	14.6	0	0.45	9.249	6.89	39.62	**4.69**
I	1029	-0.914	0.522	2.054	0.678	-2.78	-27.6	0	0.28	45.3	22.5	0	0.59	39.79	8.46	84.31	**5.17**
II	1091	-0.827	0.359	2.422	0.799	-3.08	-23.5	0	0.61	48	23.9	0	-0.02	47.71	8.63	94.51	**5.20**
III	1087	-0.549	0.479	3.007	0.992	-3.28	-13.7	0	1.42	44.7	22.2	0	-0	54.42	7.68	98.82	**5.26**
IV	1034	-0.626	0.595	2.884	0.952	-3.29	-16.2	0	0.3	46.6	23.2	0	0.02	53.53	8.08	99.19	**5.23**

Table B2. Reliability indices for the moment capacity based on the Tunnel Manual, $\varphi = 0.90$

	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L
Load Factor	1.25	1.5	1.75	1.75	1	1.35	1.35	1.35	1.5	1.5	1.35	1.35
	0.9	0.65	1.75	1.75	1	0.9	0.9	0.9	0.75	0.75	0.9	0.9
λ_Q	1.05	1.03	1.25	1.25	0.9	1	0.95	0.95	1	0.95	1	1
V_Q	0.1	0.08	0.18	0.18	0.15	0.14	0.15	0.15	0.15	0.15	0.12	0.12

λ_R	V_R	ϕ
1.140	0.080	0.90

36 S

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1002	-0.297	-4E-04	0.001	4E-04	1.32	-10	-1.3	1.22	11.9	5.92	0.3	0.36	10.13	2.43	23.62	**5.42**
I	1021	-0.061	-0.008	0.006	0.002	1.09	0.263	0.58	1.47	8.3	4.15	0.48	0.77	16	1.41	26.37	**5.55**
II	1101	-0.061	4E-04	0.006	0.002	1.09	0.263	-0.59	1.63	8.29	4.15	0.77	0.48	16.14	1.41	26.59	**5.56**
III	1090	-0.011	0.006	0.054	0.018	0.4	-0.1	-0.04	0.67	4.58	2.29	-0.24	0.02	7.729	0.77	12.72	**5.29**
IV	1032	-0.011	-0.008	0.054	0.018	0.4	-0.1	0	0.08	4.6	2.29	0.02	-0.24	7.175	0.77	11.85	**5.18**

100 S. South Charles Street

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1002	-0.3	0	0	0	1.2	-11.6	-2.5	1.4	11.6	5.7	0.2	0.3	8.01	2.53	21.12	**5.24**
I	1007	-0.1	0	0	0	1	-4.4	-1.9	1.1	6.6	3.3	0.1	0.6	7.875	1.28	15.81	**5.63**
II	1103	-0.3	0	0	0	1.2	-8.1	-1.3	0.6	12.5	6.2	0.2	0.2	12.03	2.37	25.6	**5.45**
III	1089	0	0	0	0	0.6	0.1	0	0.1	4.2	2.1	0	0	6.93	0.7	11.47	**5.30**
IV	1031	0	0	0	0	0.4	0.1	0	1.1	5.2	2.6	0	-0.1	9.075	0.88	15.14	**5.46**

Bromo Seltzer Tower

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	0.3	0	0	0	-1.1	18.7	2.3	-1	38.1	-9.7	-0.2	-0.2	45.96	6.44	81.26	**5.07**
I	1006	0.2	0	0	0	-0.9	11.1	0.9	-0.8	35.2	-7	-0.1	-0.3	38.29	5.6	67.56	**4.94**
II	1116	0.2	0	0	0	-0.9	11.1	1.8	-0.7	35.3	-7	-0.3	-0.1	38.49	5.61	67.83	**4.94**
III	1089	0	0	0	0	0.6	0.4	0	1.2	22.7	4.7	0.1	0	29.34	3.48	48.88	**5.02**
IV	1031	0	0	0	0	0.4	0.2	0	0	24.2	5.2	0	-0.1	29.7	3.71	49.64	**4.92**

Eastern Ave 1401 Garage

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1006	0.812	0.316	1.655	0.546	-0.92	-34.1	-0.04	1.18	26.4	13.1	0	0	8.421	6.48	38.41	**4.88**
I	1016	0.013	0.037	0.737	0.243	-0.49	14.85	1.93	-0.72	10.6	5.3	0	0	30.43	2.73	49.56	**5.41**
II	1090	0.335	0.388	2.48	0.818	-2.26	-17.5	-2.34	4.23	44.7	22.3	0	0	54.44	7.85	105.5	**5.7**
III	1087	0.02	0.481	2.984	0.985	-2.53	-8.6	-1.75	3.16	41.9	20.8	0	0	58.28	7.09	106.4	**5.68**
IV	1035	0.441	0.612	2.984	0.985	-2.53	-8.6	0.18	1.33	41.9	20.8	0	0	57.16	7.08	104.5	**5.65**

Grudelsky

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	-0.225	6E-04	0.003	0.001	0.7	-7.99	-0.58	1.18	5.36	2.68	0.43	0.43	2.256	1.44	9.038	**4.94**
I	1014	-0.174	0	0.004	0.001	0.27	-6.38	-0.48	0.82	4.48	2.24	0.37	0.08	1.501	1.17	6.86	**4.85**
II	1108	-0.174	9E-04	0.004	0.001	0.27	-6.38	-0.17	1.08	4.48	2.24	0.08	0.37	1.751	1.17	7.256	**4.93**
III	1089	-0.007	0.006	0.054	0.018	0.24	-0.15	-0.03	0.14	0.27	0.13	-0.14	-0	0.665	0.06	1.001	**4.64**
IV	1031	-0.009	-0.004	0.054	0.018	0.23	-0.12	-0.01	0	0.68	0.34	-0	-0.2	1.151	0.12	1.764	**4.64**

Holiday Inn - Inner Harbor Hotel

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	0.9	0	0	0	0.1	42.3	1.8	-0.3	-15	-7.6	0	0	20.83	6.43	45.68	**4.17**
I	1016	0.3	0	0	0	1.1	18.2	0.5	0.5	6.1	3	0	0	28.93	2.75	44.86	**4.89**
II	1105	0.3	0	0	0	1.2	21.6	1.1	-0.2	3.7	1.8	0	0	28.22	3.09	43.12	**4.48**
III	1076	-0.6	0	-0.1	0	0	-14.1	-0.5	-0.2	27.8	13.9	-0.6	0	25.99	5.02	53.81	**5.31**
IV	1032	-1	0	0	0	1.7	-21.6	-1.2	1.1	29.6	14.8	0	-0.2	23.59	5.78	54.74	**5.32**

Market Center West Apts

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	0.9	0	0	0	0.1	42.3	1.3	-0.6	-15	-7.6	0	0	20.55	6.43	45.38	**4.17**
I	1016	0.3	0	0	0	1.1	18.2	1	-0.3	6.1	3	0	0	28.17	2.75	43.81	**4.86**
II	1093	-0.9	0	0	0	0.5	-26.7	-2.1	1.7	32.4	16.2	-0.3	0	22.21	6.56	56.21	**5.23**
III	1076	-0.6	0	-0.1	0	0	-14.1	-0.9	1.6	27.8	13.9	-0.6	0	27.7	5.03	56.41	**5.39**
IV	1032	-1	0	0	0	1.7	-21.6	-0.5	-0.2	29.6	14.8	0	-0.2	22.35	5.78	52.89	**5.26**

Marriot Hotel

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1005	-0.819	0.339	1.941	0.64	-1.69	-34.8	0	-0.14	29.3	14.6	0	0.45	9.249	6.89	41.82	**4.97**
I	1029	-0.914	0.522	2.054	0.678	-2.78	-27.6	0	0.28	45.3	22.5	0	0.59	39.79	8.46	89	**5.55**
II	1091	-0.827	0.359	2.422	0.799	-3.08	-23.5	0	0.61	48	23.9	0	-0.02	47.71	8.63	99.76	**5.59**
III	1087	-0.549	0.479	3.007	0.992	-3.28	-13.7	0	1.42	44.7	22.2	0	-0	54.42	7.68	104.3	**5.67**
IV	1034	-0.626	0.595	2.884	0.952	-3.29	-16.2	0	0.3	46.6	23.2	0	0.02	53.53	8.08	104.7	**5.64**

Table B3. Reliability indices for the moment capacity based on the Tunnel Manual, $\varphi = 0.85$

	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L
Load	1.25	1.5	1.75	1.75	1	1.35	1.35	1.35	1.5	1.5	1.35	1.35
Factor	0.9	0.65	1.75	1.75	1	0.9	0.9	0.9	0.75	0.75	0.9	0.9
λ_Q	1.05	1.03	1.25	1.25	0.9	1	0.95	0.95	1	0.95	1	1
V_Q	0.1	0.08	0.18	0.18	0.15	0.14	0.15	0.15	0.15	0.15	0.12	0.12

λ_R	V_R	ϕ
1.140	0.080	0.85

36 S

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1002	-0.297	-0	0.001	4E-04	1.32	-10	-1.3	1.22	11.9	5.92	0.3	0.36	10.13	2.43	25.01	**5.81**
I	1021	-0.061	-0.01	0.006	0.002	1.09	0.263	0.581	1.47	8.3	4.15	0.48	0.77	16	1.41	27.92	**6.02**
II	1101	-0.061	4E-04	0.006	0.002	1.09	0.263	-0.59	1.63	8.29	4.15	0.77	0.48	16.14	1.41	28.16	**6.03**
III	1090	-0.011	0.006	0.054	0.018	0.4	-0.1	-0.04	0.67	4.58	2.29	-0.24	0.02	7.729	0.77	13.46	**5.76**
IV	1032	-0.011	-0.01	0.054	0.018	0.4	-0.1	0.005	0.08	4.6	2.29	0.02	-0.2	7.175	0.77	12.55	**5.64**

100 S. South Charles Street

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1002	-0.3	0	0	0	1.2	-11.6	-2.5	1.4	11.6	5.7	0.2	0.3	8.01	2.53	22.36	**5.61**
I	1007	-0.1	0	0	0	1	-4.4	-1.9	1.1	6.6	3.3	0.1	0.6	7.875	1.28	16.74	**6.05**
II	1103	-0.3	0	0	0	1.2	-8.1	-1.3	0.6	12.5	6.2	0.2	0.2	12.03	2.37	27.11	**5.86**
III	1089	0	0	0	0	0.6	0.1	0	0.1	4.2	2.1	0	0	6.93	0.7	12.14	**5.77**
IV	1031	0	0	0	0	0.4	0.1	0	1.1	5.2	2.6	0	-0.1	9.075	0.88	16.04	**5.93**

Bromo Seltzer Tower

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	0.3	0	0	0	-1.1	18.7	2.3	-1	38.1	-9.7	-0.2	-0.2	45.96	6.44	86.04	**5.51**
I	1006	0.2	0	0	0	-0.9	11.1	0.9	-0.8	35.2	-7	-0.1	-0.3	38.29	5.6	71.54	**5.38**
II	1116	0.2	0	0	0	-0.9	11.1	1.8	-0.7	35.3	-7	-0.3	-0.1	38.49	5.61	71.82	**5.38**
III	1089	0	0	0	0	0.6	0.4	0	1.2	22.7	4.7	0.1	0	29.34	3.48	51.76	**5.47**
IV	1031	0	0	0	0	0.4	0.2	0	0	24.2	5.2	0	-0.1	29.7	3.71	52.56	**5.37**

Eastern Ave 1401 Garage

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1006	0.812	0.316	1.655	0.546	-0.92	-34.1	-0.04	1.18	26.4	13.1	0	0	8.421	6.48	40.67	**5.19**
I	1016	0.013	0.037	0.737	0.243	-0.49	14.85	1.931	-0.72	10.6	5.3	0	0	30.43	2.73	52.47	**5.88**
II	1090	0.335	0.388	2.48	0.818	-2.26	-17.5	-2.34	4.23	44.7	22.3	0	0	54.44	7.85	111.7	**6.13**
III	1087	0.02	0.481	2.984	0.985	-2.53	-8.6	-1.75	3.16	41.9	20.8	0	0	58.28	7.09	112.7	**6.12**
IV	1035	0.441	0.612	2.984	0.985	-2.53	-8.6	0.178	1.33	41.9	20.8	0	0	57.16	7.08	110.7	**6.09**

Grudelsky

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	-0.225	6E-04	0.003	0.001	0.7	-7.99	-0.58	1.18	5.36	2.68	0.43	0.43	2.256	1.44	9.569	**5.25**
I	1014	-0.174	0	0.004	0.001	0.27	-6.38	-0.48	0.82	4.48	2.24	0.37	0.08	1.501	1.17	7.263	**5.15**
II	1108	-0.174	9E-04	0.004	0.001	0.27	-6.38	-0.17	1.08	4.48	2.24	0.08	0.37	1.751	1.17	7.682	**5.25**
III	1089	-0.007	0.006	0.054	0.018	0.24	-0.15	-0.03	0.14	0.27	0.13	-0.14	-0	0.665	0.06	1.059	**5.13**
IV	1031	-0.009	-0	0.054	0.018	0.23	-0.12	-0.01	0	0.68	0.34	-0	-0.2	1.151	0.12	1.868	**5.12**

Holiday Inn - Inner Harbor Hotel

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	0.9	0	0	0	0.1	42.3	1.8	-0.3	-15	-7.6	0	0	20.83	6.43	48.36	**4.52**
I	1016	0.3	0	0	0	1.1	18.2	0.5	0.5	6.1	3	0	0	28.93	2.75	47.49	**5.37**
II	1105	0.3	0	0	0	1.2	21.6	1.1	-0.2	3.7	1.8	0	0	28.22	3.09	45.65	**4.96**
III	1076	-0.6	0	-0.1	0	0	-14.1	-0.5	-0.2	27.8	13.9	-0.6	0	25.99	5.02	56.97	**5.72**
IV	1032	-1	0	0	0	1.7	-21.6	-1.2	1.1	29.6	14.8	0	-0.2	23.59	5.78	57.96	**5.71**

Market Center West Apts

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	0.9	0	0	0	0.1	42.3	1.3	-0.6	-15	-7.6	0	0	20.55	6.43	48.05	**4.52**
I	1016	0.3	0	0	0	1.1	18.2	1	-0.3	6.1	3	0	0	28.17	2.75	46.38	**5.34**
II	1093	-0.9	0	0	0	0.5	-26.7	-2.1	1.7	32.4	16.2	-0.3	0	22.21	6.56	59.51	**5.6**
III	1076	-0.6	0	-0.1	0	0	-14.1	-0.9	1.6	27.8	13.9	-0.6	0	27.7	5.03	59.72	**5.81**
IV	1032	-1	0	0	0	1.7	-21.6	-0.5	-0.2	29.6	14.8	0	-0.2	22.35	5.78	56	**5.65**

Marriot Hotel

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1005	-0.819	0.339	1.941	0.64	-1.69	-34.8	0	-0.14	29.3	14.6	0	0.45	9.249	6.89	44.28	**5.28**
I	1029	-0.914	0.522	2.054	0.678	-2.78	-27.6	0	0.28	45.3	22.5	0	0.59	39.79	8.46	94.23	**5.95**
II	1091	-0.827	0.359	2.422	0.799	-3.08	-23.5	0	0.61	48	23.9	0	-0	47.71	8.63	105.6	**6.01**
III	1087	-0.549	0.479	3.007	0.992	-3.28	-13.7	0	1.42	44.7	22.2	0	-0	54.42	7.68	110.4	**6.11**
IV	1034	-0.626	0.595	2.884	0.952	-3.29	-16.2	0	0.3	46.6	23.2	0	0.02	53.53	8.08	110.9	**6.07**

Table B4. Reliability indices for the shear capacity based on the Tunnel Manual, $\varphi = 0.90$

	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L
Load Factor	1.25	1.5	1.75	1.75	1	1.35	1.35	1.35	1.5	1.5	1.35	1.35
	0.9	0.65	1.75	1.75	1	0.9	0.9	0.9	0.75	0.75	0.9	0.9
λ_Q	1.05	1.03	1.25	1.25	0.9	1	0.95	0.95	1	0.95	1	1
V_Q	0.1	0.08	0.18	0.18	0.15	0.14	0.15	0.15	0.15	0.15	0.12	0.12

λ_R	V_R	ϕ
1.200	0.150	0.90

36 S

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.2	4E-04	0.01	0.002	37.76	33.19	5.013	1.929	19.9	9.98	0.59	0.6	100.8	7.66	147.98	**3.36**
I	1006	1.262	5E-04	0.01	0.002	37.75	35.14	5.313	1.826	18	9.02	0.54	0.66	99.93	7.7	146.06	**3.33**
II	1095	1.451	0.006	0.05	0.016	42.17	38.8	6.57	4.709	9.51	4.76	4.41	0.49	101.5	8.07	145.37	**3.21**
III	1089	1.149	0.011	0.09	0.031	44.76	28.33	4.806	5.153	8.3	4.15	5.56	0.55	92.9	7.42	131.73	**3.15**
IV	1025	1.441	0.017	0.04	0.013	41.58	39.9	5.495	1.296	9.09	4.54	0.48	4.04	97.83	8.08	139.62	**3.17**

100 S. South Charles Street

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.4	0	0	0	42.4	37.6	7.8	3.1	24.1	11.9	0.8	0.9	117.2	8.76	172.66	**3.39**
I	1007	1.4	0	0	0	42.5	40.4	8.4	3.1	21.4	10.6	0.7	1	116.2	8.82	170.3	**3.36**
II	1116	1.4	0	0	0	42.5	39.5	8.2	2.9	22.3	11	0.8	0.8	116.2	8.8	170.67	**3.37**
III	1089	1.3	0	0.1	0	50.4	32.7	6.1	1.6	8.4	4.2	6.4	0.7	100.2	8.36	141.1	**3.09**
IV	1032	1.2	0	0.1	0	50.4	32.7	7.6	6.9	8.4	4.2	0.6	6.6	105.2	8.41	149.06	**3.14**

Bromo Seltzer Tower

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.2	0	0	0	25.4	51.7	2.3	1.9	38.1	18.8	0.5	0.5	134.5	10.2	206.62	**3.61**
I	1007	1.3	0	0	0	25.4	56.1	0.5	1.8	34	16.8	0.4	0.6	132.9	10.3	203.04	**3.57**
II	1116	1.2	0	0	0	25.4	54.6	1.8	1.9	35.3	17.4	0.5	0.4	133.2	10.2	203.82	**3.58**
III	1089	1.3	0	0	0	32.6	45.2	0	5.1	22.7	11.2	6.2	0.3	120.3	8.64	179.73	**3.49**
IV	1031	1.2	0	0	0	32.3	48.6	0	1.1	24.2	11.9	0.3	6.1	121.6	9.05	181.87	**3.48**

Eastern Ave 1401 Garage

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.133	0.095	0.49	0.16	46.13	30.97	1.699	4.116	51.1	25.5	0	0	153.6	11.4	234.47	**3.59**
I	1007	1.124	0.092	0.48	0.159	46.29	32.92	1.865	3.398	49.7	24.8	0	0	153	11.3	233.14	**3.58**
II	1114	1.151	0.086	0.48	0.159	46.29	33.83	1.63	4.61	49	24.5	0	0	154	11.3	234.46	**3.58**
III	1062	1.29	-0.03	-0.4	-0.12	52.66	21.89	1.502	3.273	54.8	27.4	0	0	154	12	234	**3.54**
IV	1058	1.277	-0.03	-0.5	-0.17	52.65	21.93	1.429	2.972	54.1	27	0	0	152.5	11.9	231.35	**3.53**

Grudelsky

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.258	0.001	0.01	0.002	42.15	40.47	4.549	1.908	11.4	5.7	0.63	0.63	99.57	8.26	142.58	**3.18**
I	1006	1.319	0.001	0.01	0.002	42.13	42.59	4.778	1.846	9.33	4.67	0.59	0.66	98.62	8.39	140.54	**3.14**
II	1115	1.342	0.001	0.01	0.002	42.13	43.34	4.821	1.581	8.58	4.29	0.66	0.59	98.03	8.44	139.41	**3.12**
III	1089	0.757	0.012	0.1	0.033	49.15	24.06	3.55	5.857	1.79	0.9	5.29	0.52	82.95	7.52	114	**2.9**
IV	1031	0.693	0.014	0.09	0.031	48.81	25.46	2.467	0.959	1.92	0.96	0.51	5.32	79.54	7.52	108.64	**2.83**

Holiday Inn - Inner Harbor Hotel

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.2	0	0	0	33	47.5	2.1	1.7	79.7	40.2	0.1	0.1	198.2	15.5	312.27	**3.73**
I	1007	1.3	0	0	0	33.2	49.5	2.2	1.9	78.2	39.4	0.1	0.2	198.5	15.4	312.24	**3.72**
II	1116	1.3	0	0	0	33.1	49.2	2.2	1.5	78.2	39.4	0.1	0.1	197.6	15.4	310.93	**3.72**
III	1079	2.9	0	0	0	38.3	58.1	2.3	0.9	73.7	37	0.7	0.1	206.1	15.6	320.78	**3.68**
IV	1041	3	0	0	0	35.7	61.1	3.2	2.4	72.4	36.3	0.1	0.3	205.9	15.5	320.85	**3.69**

Market Center West Apts

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.2	0	0	0	33	47.5	2	2.7	79.7	40.2	0.1	0.1	199.1	15.5	313.77	**3.73**
I	1007	1.3	0	0	0	33.2	49.5	2	2.3	78.2	39.4	0.1	0.2	198.8	15.4	312.84	**3.73**
II	1116	1.3	0	0	0	33.1	49.2	2	3	78.2	39.4	0.1	0.1	199	15.4	313.18	**3.73**
III	1079	2.9	0	0	0	38.3	58.1	3.8	3.6	73.7	37	0.7	0.1	208.6	15.6	324.83	**3.69**
IV	1041	3	0	0	0	35.7	61.1	1.9	1.5	72.4	36.3	0.1	0.3	205.1	15.5	319.5	**3.69**

Marriot Hotel

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.193	0.104	0.48	0.159	57.38	25.88	0	0.425	53.8	26.9	0	0	159.3	12.4	240.75	**3.51**
I	1006	1.162	0.097	0.46	0.152	57.51	26.74	0	0.399	52.8	26.4	0	0	158.7	12.3	239.53	**3.51**
II	1116	1.19	0.098	0.47	0.154	57.47	26.98	0	0.476	52.6	26.3	0	0	158.7	12.3	239.52	**3.51**
III	1061	1.39	0.028	-0.1	-0.02	63.35	17.88	0	1.209	57.7	28.8	0	0	162.5	13.1	244.94	**3.48**
IV	1059	1.384	0.026	-0.2	-0.05	63.34	17.9	0	1.136	57.4	28.6	0	0	161.8	13	243.85	**3.48**

Table B5. Reliability indices for the shear capacity based on the Tunnel Manual, $\varphi = 0.85$

	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L
Load Factor	1.25	1.5	1.75	1.75	1	1.35	1.35	1.35	1.5	1.5	1.35	1.35
	0.9	0.65	1.75	1.75	1	0.9	0.9	0.9	0.75	0.75	0.9	0.9
λ_Q	1.05	1.03	1.25	1.25	0.9	1	0.95	0.95	1	0.95	1	1
V_Q	0.1	0.08	0.18	0.18	0.15	0.14	0.15	0.15	0.15	0.15	0.12	0.12

λ_R	V_R	ϕ
1.200	0.150	0.85

36 S

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.2	4E-04	0.01	0.002	37.76	33.19	5.013	1.929	19.9	9.98	0.59	0.6	100.8	7.66	156.68	**3.67**
I	1006	1.262	5E-04	0.01	0.002	37.75	35.14	5.313	1.826	18	9.02	0.54	0.66	99.93	7.7	154.65	**3.64**
II	1095	1.451	0.006	0.05	0.016	42.17	38.8	6.57	4.709	9.51	4.76	4.41	0.49	101.5	8.07	153.92	**3.52**
III	1089	1.149	0.011	0.09	0.031	44.76	28.33	4.806	5.153	8.3	4.15	5.56	0.55	92.9	7.42	139.48	**3.46**
IV	1025	1.441	0.017	0.04	0.013	41.58	39.9	5.495	1.296	9.09	4.54	0.48	4.04	97.83	8.08	147.83	**3.48**

100 S. South Charles Street

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.4	0	0	0	42.4	37.6	7.8	3.1	24.1	11.9	0.8	0.9	117.2	8.76	182.81	**3.70**
I	1007	1.4	0	0	0	42.5	40.4	8.4	3.1	21.4	10.6	0.7	1	116.2	8.82	180.32	**3.67**
II	1116	1.4	0	0	0	42.5	39.5	8.2	2.9	22.3	11	0.8	0.8	116.2	8.8	180.71	**3.67**
III	1089	1.3	0	0.1	0	50.4	32.7	6.1	1.6	8.4	4.2	6.4	0.7	100.2	8.36	149.4	**3.40**
IV	1032	1.2	0	0.1	0	50.4	32.7	7.6	6.9	8.4	4.2	0.6	6.6	105.2	8.41	157.83	**3.46**

Bromo Seltzer Tower

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.2	0	0	0	25.4	51.7	2.3	1.9	38.1	18.8	0.5	0.5	134.5	10.2	218.78	**3.91**
I	1007	1.3	0	0	0	25.4	56.1	0.5	1.8	34	16.8	0.4	0.6	132.9	10.3	214.99	**3.87**
II	1116	1.2	0	0	0	25.4	54.6	1.8	1.9	35.3	17.4	0.5	0.4	133.2	10.2	215.81	**3.88**
III	1089	1.3	0	0	0	32.6	45.2	0	5.1	22.7	11.2	6.2	0.3	120.3	8.64	190.3	**3.79**
IV	1031	1.2	0	0	0	32.3	48.6	0	1.1	24.2	11.9	0.3	6.1	121.6	9.05	192.57	**3.78**

Eastern Ave 1401 Garage

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.133	0.095	0.49	0.16	46.13	30.97	1.699	4.116	51.1	25.5	0	0	153.6	11.4	248.26	**3.88**
I	1007	1.124	0.092	0.48	0.159	46.29	32.92	1.865	3.398	49.7	24.8	0	0	153	11.3	246.85	**3.87**
II	1114	1.151	0.086	0.48	0.159	46.29	33.83	1.63	4.61	49	24.5	0	0	154	11.3	248.26	**3.88**
III	1062	1.29	-0.03	-0.4	-0.12	52.66	21.89	1.502	3.273	54.8	27.4	0	0	154	12	247.76	**3.84**
IV	1058	1.277	-0.03	-0.5	-0.17	52.65	21.93	1.429	2.972	54.1	27	0	0	152.5	11.9	244.96	**3.83**

Grudelsky

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.258	0.001	0.01	0.002	42.15	40.47	4.549	1.908	11.4	5.7	0.63	0.63	99.57	8.26	150.97	**3.49**
I	1006	1.319	0.001	0.01	0.002	42.13	42.59	4.778	1.846	9.33	4.67	0.59	0.66	98.62	8.39	148.81	**3.45**
II	1115	1.342	0.001	0.01	0.002	42.13	43.34	4.821	1.581	8.58	4.29	0.66	0.59	98.03	8.44	147.61	**3.44**
III	1089	0.757	0.012	0.1	0.033	49.15	24.06	3.55	5.857	1.79	0.9	5.29	0.52	82.95	7.52	120.7	**3.22**
IV	1031	0.693	0.014	0.09	0.031	48.81	25.46	2.467	0.959	1.92	0.96	0.51	5.32	79.54	7.52	115.03	**3.16**

Holiday Inn - Inner Harbor Hotel

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.2	0	0	0	33	47.5	2.1	1.7	79.7	40.2	0.1	0.1	198.2	15.5	330.64	**4.02**
I	1007	1.3	0	0	0	33.2	49.5	2.2	1.9	78.2	39.4	0.1	0.2	198.5	15.4	330.61	**4.01**
II	1116	1.3	0	0	0	33.1	49.2	2.2	1.5	78.2	39.4	0.1	0.1	197.6	15.4	329.22	**4.01**
III	1079	2.9	0	0	0	38.3	58.1	2.3	0.9	73.7	37	0.7	0.1	206.1	15.6	339.65	**3.97**
IV	1041	3	0	0	0	35.7	61.1	3.2	2.4	72.4	36.3	0.1	0.3	205.9	15.5	339.72	**3.98**

Market Center West Apts

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.2	0	0	0	33	47.5	2	2.7	79.7	40.2	0.1	0.1	199.1	15.5	332.22	**4.02**
I	1007	1.3	0	0	0	33.2	49.5	2	2.3	78.2	39.4	0.1	0.2	198.8	15.4	331.25	**4.02**
II	1116	1.3	0	0	0	33.1	49.2	2	3	78.2	39.4	0.1	0.1	199	15.4	331.61	**4.02**
III	1079	2.9	0	0	0	38.3	58.1	3.8	3.6	73.7	37	0.7	0.1	208.6	15.6	343.94	**3.98**
IV	1041	3	0	0	0	35.7	61.1	1.9	1.5	72.4	36.3	0.1	0.3	205.1	15.5	338.29	**3.98**

Marriot Hotel

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.193	0.104	0.48	0.159	57.38	25.88	0	0.425	53.8	26.9	0	0	159.3	12.4	254.91	**3.81**
I	1006	1.162	0.097	0.46	0.152	57.51	26.74	0	0.399	52.8	26.4	0	0	158.7	12.3	253.62	**3.81**
II	1116	1.19	0.098	0.47	0.154	57.47	26.98	0	0.476	52.6	26.3	0	0	158.7	12.3	253.6	**3.81**
III	1061	1.39	0.028	-0.1	-0.02	63.35	17.88	0	1.209	57.7	28.8	0	0	162.5	13.1	259.34	**3.78**
IV	1059	1.384	0.026	-0.2	-0.05	63.34	17.9	0	1.136	57.4	28.6	0	0	161.8	13	258.19	**3.78**

Table B6. Reliability indices for the shear capacity based on the Tunnel Manual, $\varphi = 0.80$

	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L
Load Factor	1.25	1.5	1.75	1.75	1	1.35	1.35	1.35	1.5	1.5	1.35	1.35
	0.9	0.65	1.75	1.75	1	0.9	0.9	0.9	0.75	0.75	0.9	0.9
λ_Q	1.05	1.03	1.25	1.25	0.9	1	0.95	0.95	1	0.95	1	1
V_Q	0.1	0.08	0.18	0.18	0.15	0.14	0.15	0.15	0.15	0.15	0.12	0.12

λ_R	V_R	ϕ
1.200	0.150	0.80

36 S

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.2	4E-04	0.01	0.002	37.76	33.19	5.013	1.929	19.9	9.98	0.59	0.6	100.8	7.659	166.47	**3.98**
I	1006	1.262	5E-04	0.01	0.002	37.75	35.14	5.313	1.826	18	9.02	0.54	0.66	99.93	7.697	164.32	**3.95**
II	1095	1.451	0.006	0.05	0.016	42.17	38.8	6.57	4.709	9.51	4.76	4.41	0.49	101.5	8.07	163.54	**3.84**
III	1089	1.149	0.011	0.09	0.031	44.76	28.33	4.806	5.153	8.3	4.15	5.56	0.55	92.9	7.423	148.2	**3.78**
IV	1025	1.441	0.017	0.04	0.013	41.58	39.9	5.495	1.296	9.09	4.54	0.48	4.04	97.83	8.078	157.07	**3.80**

100 S. South Charles Street

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.4	0	0	0	42.4	37.6	7.8	3.1	24.1	11.9	0.8	0.9	117.2	8.755	194.24	**4.01**
I	1007	1.4	0	0	0	42.5	40.4	8.4	3.1	21.4	10.6	0.7	1	116.2	8.817	191.59	**3.98**
II	1116	1.4	0	0	0	42.5	39.5	8.2	2.9	22.3	11	0.8	0.8	116.2	8.795	192	**3.98**
III	1089	1.3	0	0.1	0	50.4	32.7	6.1	1.6	8.4	4.2	6.4	0.7	100.2	8.355	158.74	**3.72**
IV	1032	1.2	0	0.1	0	50.4	32.7	7.6	6.9	8.4	4.2	0.6	6.6	105.2	8.411	167.69	**3.78**

Bromo Seltzer Tower

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.2	0	0	0	25.4	51.7	2.3	1.9	38.1	18.8	0.5	0.5	134.5	10.2	232.45	**4.20**
I	1007	1.3	0	0	0	25.4	56.1	0.5	1.8	34	16.8	0.4	0.6	132.9	10.26	228.43	**4.17**
II	1116	1.2	0	0	0	25.4	54.6	1.8	1.9	35.3	17.4	0.5	0.4	133.2	10.22	229.3	**4.18**
III	1089	1.3	0	0	0	32.6	45.2	0	5.1	22.7	11.2	6.2	0.3	120.3	8.637	202.19	**4.09**
IV	1031	1.2	0	0	0	32.3	48.6	0	1.1	24.2	11.9	0.3	6.1	121.6	9.049	204.61	**4.08**

Eastern Ave 1401 Garage

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.133	0.095	0.49	0.16	46.13	30.97	1.699	4.116	51.1	25.5	0	0	153.6	11.39	263.77	**4.18**
I	1007	1.124	0.092	0.48	0.159	46.29	32.92	1.865	3.398	49.7	24.8	0	0	153	11.34	262.28	**4.17**
II	1114	1.151	0.086	0.48	0.159	46.29	33.83	1.63	4.61	49	24.5	0	0	154	11.32	263.77	**4.18**
III	1062	1.29	-0.03	-0.4	-0.12	52.66	21.89	1.502	3.273	54.8	27.4	0	0	154	11.95	263.25	**4.14**
IV	1058	1.277	-0.03	-0.5	-0.17	52.65	21.93	1.429	2.972	54.1	27	0	0	152.5	11.86	260.27	**4.13**

Grudelsky

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.258	0.001	0.01	0.002	42.15	40.47	4.549	1.908	11.4	5.7	0.63	0.63	99.57	8.257	160.4	**3.81**
I	1006	1.319	0.001	0.01	0.002	42.13	42.59	4.778	1.846	9.33	4.67	0.59	0.66	98.62	8.391	158.11	**3.77**
II	1115	1.342	0.001	0.01	0.002	42.13	43.34	4.821	1.581	8.58	4.29	0.66	0.59	98.03	8.443	156.84	**3.76**
III	1089	0.757	0.012	0.1	0.033	49.15	24.06	3.55	5.857	1.79	0.9	5.29	0.52	82.95	7.518	128.25	**3.55**
IV	1031	0.693	0.014	0.09	0.031	48.81	25.46	2.467	0.959	1.92	0.96	0.51	5.32	79.54	7.524	122.22	**3.49**

Holiday Inn - Inner Harbor Hotel

Segment	Member	DL	SDL	LL	I	B	EV	ES-V	ES-H	EH1	EH2	EHL	HER	Mean Q	σ	factor Q	β
Key	1001	1.2	0	0	0	33	47.5	2.1	1.7	79.7	40.2	0.1	0.1	198.2	15.49	351.3	**4.31**
I	1007	1.3	0	0	0	33.2	49.5	2.2	1.9	78.2	39.4	0.1	0.2	198.5	15.41	351.28	**4.31**
II	1116	1.3	0	0	0	33.1	49.2	2.2	1.5	78.2	39.4	0.1	0.1	197.6	15.38	349.8	**4.31**
III	1079	2.9	0	0	0	38.3	58.1	2.3	0.9	73.7	37	0.7	0.1	206.1	15.59	360.88	**4.27**
IV	1041	3	0	0	0	35.7	61.1	3.2	2.4	72.4	36.3	0.1	0.3	205.9	15.53	360.96	**4.27**

Market Center West Apts

Segment	Member	DL	SDL	LL	I	B	EV	ES-V	ES-H	EH1	EH2	EHL	HER	Mean Q	σ	factor Q	β
Key	1001	1.2	0	0	0	33	47.5	2	2.7	79.7	40.2	0.1	0.1	199.1	15.49	352.99	**4.31**
I	1007	1.3	0	0	0	33.2	49.5	2	2.3	78.2	39.4	0.1	0.2	198.8	15.41	351.95	**4.31**
II	1116	1.3	0	0	0	33.1	49.2	2	3	78.2	39.4	0.1	0.1	199	15.39	352.33	**4.31**
III	1079	2.9	0	0	0	38.3	58.1	3.8	3.6	73.7	37	0.7	0.1	208.6	15.6	365.44	**4.27**
IV	1041	3	0	0	0	35.7	61.1	1.9	1.5	72.4	36.3	0.1	0.3	205.1	15.53	359.44	**4.27**

Marriot Hotel

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.193	0.104	0.48	0.159	57.38	25.88	0	0.425	53.8	26.9	0	0	159.3	12.37	270.84	**4.12**
I	1006	1.162	0.097	0.46	0.152	57.51	26.74	0	0.399	52.8	26.4	0	0	158.7	12.29	269.48	**4.11**
II	1116	1.19	0.098	0.47	0.154	57.47	26.98	0	0.476	52.6	26.3	0	0	158.7	12.28	269.46	**4.11**
III	1061	1.39	0.028	-0.1	-0.02	63.35	17.88	0	1.209	57.7	28.8	0	0	162.5	13.08	275.55	**4.08**
IV	1059	1.384	0.026	-0.2	-0.05	63.34	17.9	0	1.136	57.4	28.6	0	0	161.8	13.04	274.33	**4.08**

Table B7. Reliability indices for the compression capacity based on the Tunnel Manual, $\varphi = 0.80$

	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L
Load Factor	1.25	1.5	1.75	1.75	1	1.35	1.35	1.35	1.5	1.5	1.35	1.35
	0.9	0.65	1.75	1.75	1	0.9	0.9	0.9	0.75	0.75	0.9	0.9
λ_Q	1.05	1.03	1.25	1.25	0.9	1	0.95	0.95	1	0.95	1	1
V_Q	0.1	0.08	0.18	0.18	0.15	0.14	0.15	0.15	0.15	0.15	0.12	0.12

λ_R	V_R	ϕ
1.180	0.120	0.80

36 S

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1005	-0.036	-4E-04	0	1E-04	1.153	-1.649	-0.39	0.09	2.634	1.313	0.045	-0.04	3.3601	0.519	7.124	**5.17**
I	1025	0.0099	-6E-04	0.0039	0.012	1.389	0.942	0.332	0.462	3.214	1.603	0.156	0.181	7.7144	0.585	13.76	**5.03**
II	1097	0.0099	0.0015	0.0039	0.012	1.389	0.942	-0.08	0.605	3.206	1.603	0.182	0.156	7.8441	0.587	13.99	**5.04**

100 S. South Charles Street

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1005	0	0	0	0	1.2	-1.7	-0.1	0.2	2.2	1.1	0	-0.1	2.715	0.466	6	**5.20**
I	1025	0.1	0	0	0	1.6	2.6	0.1	0.9	3.2	1.6	0.2	0.3	10.22	0.693	17.91	**5.00**
II	1107	0	0	0	0	1.5	0.1	0.3	0.3	3.2	1.6	0.2	0.1	6.755	0.571	12.06	**4.96**
III	1087	0	0	0	0	-1.4	0.1	0	0	1.8	0.9	-0.3	0	1.195	0.356	3.144	**4.88**
IV	1035	0	0	0	0	-1.4	0.1	0	-0.1	1.8	0.9	0	-0.3	1.1	0.356	3.031	**4.90**

Bromo Seltzer Tower

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1004	0	0	0	0	0.7	-2.1	-0.4	0.1	2.9	1.4	0	-0.1	2.855	0.57	6.631	**5.17**
I	1025	0	0	0	0	1.1	-1.4	0	0.3	8.2	4	0.1	0.1	12.065	1.379	23.52	**5.14**
II	1106	0	0	0	0	1	0.1	-0.1	0.4	5.4	2.7	0.2	0	9.535	0.909	17.62	**5.04**
III	1086	0	0	0	0	-0.8	0.2	0	0.1	4.3	2.1	-0.2	0	5.87	0.72	11.28	**4.98**
IV	1036	0	0	0	0	-0.8	0.2	0	0.1	4.3	2.1	0	-0.2	5.87	0.72	11.28	**4.98**

Eastern Ave 1401 Garage

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1005	0.1146	-0.022	-0.026	-0.008	1.059	5.914	0.598	-0.54	-0.44	-0.22	0	0	5.7653	0.847	10.16	**4.23**
I	1025	-0.003	0.0755	0.4051	0.134	0.789	2.456	0.279	0.034	8.272	4.121	0	0	15.998	1.421	29.74	**5.14**
II	1098	-0.037	0.0692	0.4016	0.133	0.793	1.507	-0.57	0.832	8.9	4.433	0	0	16.688	1.502	31.19	**5.16**
III	1075	0.0923	0.0925	0.7134	0.236	-1.9	-0.083	-0.11	0.491	6.679	3.315	0	0	9.6379	1.145	19.49	**5.29**
IV	1049	0.1756	0.2528	1.2225	0.404	-1.91	0.869	0.066	0.389	6.368	3.16	0	0	10.966	1.112	21.91	**5.40**

Grudelsky

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	-0.225	0.0006	0.003	0.001	0.698	-7.99	-0.58	1.184	5.364	2.68	0.429	0.429	2.2556	1.445	10.17	**5.17**
I	1014	-0.174	0	0.0039	0.001	0.275	-6.378	-0.48	0.816	4.48	2.239	0.371	0.077	1.5007	1.17	7.717	**5.10**
II	1108	-0.174	0.0009	0.0039	0.001	0.275	-6.378	-0.17	1.079	4.48	2.239	0.077	0.371	1.7515	1.174	8.163	**5.17**

Holiday Inn - Inner Harbor Hotel

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1005	0.1	0	0	0	0.8	6.6	0.2	0.1	-0.1	-0.1	0	0	7.325	0.931	12.28	**4.15**
I	1024	-0.1	0	0	0	1.2	0.5	0	0.3	7.3	3.7	0	0	12.575	1.229	23.36	**5.05**
II	1101	0	0	0	0	0.9	-0.6	0	0	8.3	4.1	0	0	12.405	1.383	23.7	**5.06**
III	1079	0.6	0	0	0	0	15.4	0.6	0.2	5	2.5	-0.2	0	23.595	2.312	41.1	**4.68**
IV	1042	0.1	0	0	0	-0.2	5	0.2	0.3	7.4	3.7	0	-0.2	16.125	1.415	29.44	**5.05**

Market Center West Apts

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1005	0.1	0	0	0	0.8	6.6	0.4	-0.2	-0.1	-0.1	0	0	7.04	0.931	11.88	**4.13**
I	1021	0	0	0	0	0.9	-0.6	0.1	-0.1	8.3	4.1	0	0	12.31	1.383	23.59	**5.07**
II	1100	0	0	0	0	1	-0.2	-0.3	0.4	8.2	4.1	0	0	13.175	1.37	24.76	**5.05**
III	1079	0.6	0	0	0	0	15.4	0.9	0.2	5	2.5	-0.2	0	23.595	2.311	41.1	**4.68**
IV	1043	0.6	0	0	0	0	15.4	0.5	0.3	5	2.5	0	-0.2	23.69	2.312	41.27	**4.69**

Marriot Hotel

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1005	0.1042	-0.031	-0.088	-0.029	1.36	5.008	0	-0.06	-0.53	-0.26	0	0	5.3614	0.73	9.228	**4.21**
I	1024	-0.089	0.0719	0.3941	0.13	0.986	0.007	0	0.004	9.476	4.719	0	0	15.363	1.579	29.05	**5.09**
II	1099	-0.113	0.0703	0.3949	0.13	0.948	-0.979	0	0.196	10.03	4.993	0	0	15.311	1.676	29.73	**5.17**
III	1078	-0.074	0.0247	0.2974	0.098	-2.3	-2.532	0	0.312	8.515	4.229	0	0	8.5698	1.49	19.53	**5.34**
IV	1044	-0.074	0.1911	0.2974	0.098	-2.3	-2.534	0	0.079	8.523	4.233	0	0	8.5288	1.491	19.46	**5.33**

Table B8. Reliability indices for the compression capacity based on the Tunnel Manual, $\varphi = 0.75$

	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L
Load Factor	1.25	1.5	1.75	1.75	1	1.35	1.35	1.35	1.5	1.5	1.35	1.35
	0.9	0.65	1.75	1.75	1	0.9	0.9	0.9	0.75	0.75	0.9	0.9
λ_Q	1.05	1.03	1.25	1.25	0.9	1	0.95	0.95	1	0.95	1	1
V_Q	0.1	0.08	0.18	0.18	0.15	0.14	0.15	0.15	0.15	0.15	0.12	0.12

λ_R	V_R	ϕ
1.180	0.120	0.75

36 S

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1005	-0.036	-4E-04	0	1E-04	1.153	-1.649	-0.39	0.09	2.634	1.313	0.045	-0.04	3.3601	0.519	7.599	**5.50**
I	1025	0.0099	-6E-04	0.0039	0.012	1.389	0.942	0.332	0.462	3.214	1.603	0.156	0.181	7.7144	0.585	14.67	**5.38**
II	1097	0.0099	0.0015	0.0039	0.012	1.389	0.942	-0.08	0.605	3.206	1.603	0.182	0.156	7.8441	0.587	14.92	**5.39**

100 S. South Charles Street

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1005	0	0	0	0	1.2	-1.7	-0.1	0.2	2.2	1.1	0	-0.1	2.715	0.466	6.4	**5.53**
I	1025	0.1	0	0	0	1.6	2.6	0.1	0.9	3.2	1.6	0.2	0.3	10.22	0.693	19.1	**5.35**
II	1107	0	0	0	0	1.5	0.1	0.3	0.3	3.2	1.6	0.2	0.1	6.755	0.571	12.86	**5.31**
III	1087	0	0	0	0	-1.4	0.1	0	0	1.8	0.9	-0.3	0	1.195	0.356	3.353	**5.2**
IV	1035	0	0	0	0	-1.4	0.1	0	-0.1	1.8	0.9	0	-0.3	1.1	0.356	3.233	**5.21**

Bromo Seltzer Tower

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1004	0	0	0	0	0.7	-2.1	-0.4	0.1	2.9	1.4	0	-0.1	2.855	0.57	7.073	**5.5**
I	1025	0	0	0	0	1.1	-1.4	0	0.3	8.2	4	0.1	0.1	12.065	1.379	25.09	**5.48**
II	1106	0	0	0	0	1	0.1	-0.1	0.4	5.4	2.7	0.2	0	9.535	0.909	18.79	**5.39**
III	1086	0	0	0	0	-0.8	0.2	0	0.1	4.3	2.1	-0.2	0	5.87	0.72	12.03	**5.32**
IV	1036	0	0	0	0	-0.8	0.2	0	0.1	4.3	2.1	0	-0.2	5.87	0.72	12.03	**5.32**

Eastern Ave 1401 Garage

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1005	0.1146	-0.022	-0.026	-0.008	1.059	5.914	0.598	-0.54	-0.44	-0.22	0	0	5.7653	0.847	10.84	**4.59**
I	1025	-0.003	0.0755	0.4051	0.134	0.789	2.456	0.279	0.034	8.272	4.121	0	0	15.998	1.421	31.72	**5.48**
II	1098	-0.037	0.0692	0.4016	0.133	0.793	1.507	-0.57	0.832	8.9	4.433	0	0	16.688	1.502	33.27	**5.5**
III	1075	0.0923	0.0925	0.7134	0.236	-1.9	-0.083	-0.11	0.491	6.679	3.315	0	0	9.6379	1.145	20.78	**5.62**
IV	1049	0.1756	0.2528	1.2225	0.404	-1.91	0.869	0.066	0.389	6.368	3.16	0	0	10.966	1.112	23.37	**5.73**

Grudelsky

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	-0.225	0.0006	0.003	0.001	0.698	-7.99	-0.58	1.184	5.364	2.68	0.429	0.429	2.2556	1.445	10.85	**5.46**
I	1014	-0.174	0	0.0039	0.001	0.275	-6.378	-0.48	0.816	4.48	2.239	0.371	0.077	1.5007	1.17	8.232	**5.39**
II	1108	-0.174	0.0009	0.0039	0.001	0.275	-6.378	-0.17	1.079	4.48	2.239	0.077	0.371	1.7515	1.174	8.707	**5.46**

Holiday Inn - Inner Harbor Hotel

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1005	0.1	0	0	0	0.8	6.6	0.2	0.1	-0.1	-0.1	0	0	7.325	0.931	13.09	**4.52**
I	1024	-0.1	0	0	0	1.2	0.5	0	0.3	7.3	3.7	0	0	12.575	1.229	24.92	**5.39**
II	1101	0	0	0	0	0.9	-0.6	0	0	8.3	4.1	0	0	12.405	1.383	25.28	**5.40**
III	1079	0.6	0	0	0	0	15.4	0.6	0.2	5	2.5	-0.2	0	23.595	2.312	43.84	**5.05**
IV	1042	0.1	0	0	0	-0.2	5	0.2	0.3	7.4	3.7	0	-0.2	16.125	1.415	31.4	**5.39**

Market Center West Apts

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1005	0.1	0	0	0	0.8	6.6	0.4	-0.2	-0.1	-0.1	0	0	7.04	0.931	12.67	**4.51**
I	1021	0	0	0	0	0.9	-0.6	0.1	-0.1	8.3	4.1	0	0	12.31	1.383	25.16	**5.41**
II	1100	0	0	0	0	1	-0.2	-0.3	0.4	8.2	4.1	0	0	13.175	1.37	26.41	**5.39**
III	1079	0.6	0	0	0	0	15.4	0.9	0.2	5	2.5	-0.2	0	23.595	2.311	43.84	**5.05**
IV	1043	0.6	0	0	0	0	15.4	0.5	0.3	5	2.5	0	-0.2	23.69	2.312	44.02	**5.05**

Marriot Hotel

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1005	0.1042	-0.031	-0.088	-0.029	1.36	5.008	0	-0.06	-0.53	-0.26	0	0	5.3614	0.73	9.843	**4.58**
I	1024	-0.089	0.0719	0.3941	0.13	0.986	0.007	0	0.004	9.476	4.719	0	0	15.363	1.579	30.99	**5.43**
II	1099	-0.113	0.0703	0.3949	0.13	0.948	-0.979	0	0.196	10.03	4.993	0	0	15.311	1.676	31.72	**5.51**
III	1078	-0.074	0.0247	0.2974	0.098	-2.3	-2.532	0	0.312	8.515	4.229	0	0	8.5698	1.49	20.83	**5.66**
IV	1044	-0.074	0.1911	0.2974	0.098	-2.3	-2.534	0	0.079	8.523	4.233	0	0	8.5288	1.491	20.76	**5.66**

Table B9. Reliability indices for the compression capacity based on the Tunnel Manual, $\varphi = 0.70$

	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L
Load Factor	1.25	1.5	1.75	1.75	1	1.35	1.35	1.35	1.5	1.5	1.35	1.35
	0.9	0.65	1.75	1.75	1	0.9	0.9	0.9	0.75	0.75	0.9	0.9
λ_Q	1.05	1.03	1.25	1.25	0.9	1	0.95	0.95	1	0.95	1	1
V_Q	0.1	0.08	0.18	0.18	0.15	0.14	0.15	0.15	0.15	0.15	0.12	0.12

λ_R	V_R	ϕ
1.180	0.120	0.70

36 S

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1005	-0.036	-4E-04	0	1E-04	1.153	-1.649	-0.39	0.09	2.634	1.313	0.045	-0.04	3.3601	0.519	8.142	**5.84**
I	1025	0.0099	-6E-04	0.0039	0.012	1.389	0.942	0.332	0.462	3.214	1.603	0.156	0.181	7.7144	0.585	15.72	**5.73**
II	1097	0.0099	0.0015	0.0039	0.012	1.389	0.942	-0.08	0.605	3.206	1.603	0.182	0.156	7.8441	0.587	15.98	**5.74**

100 S. South Charles Street

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1005	0	0	0	0	1.2	-1.7	-0.1	0.2	2.2	1.1	0	-0.1	2.715	0.466	6.857	**5.87**
I	1025	0.1	0	0	0	1.6	2.6	0.1	0.9	3.2	1.6	0.2	0.3	10.22	0.693	20.46	**5.7**
II	1107	0	0	0	0	1.5	0.1	0.3	0.3	3.2	1.6	0.2	0.1	6.755	0.571	13.78	**5.66**
III	1087	0	0	0	0	-1.4	0.1	0	0	1.8	0.9	-0.3	0	1.195	0.356	3.593	**5.54**
IV	1035	0	0	0	0	-1.4	0.1	0	-0.1	1.8	0.9	0	-0.3	1.1	0.356	3.464	**5.55**

Bromo Seltzer Tower

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1004	0	0	0	0	0.7	-2.1	-0.4	0.1	2.9	1.4	0	-0.1	2.855	0.57	7.579	**5.84**
I	1025	0	0	0	0	1.1	-1.4	0	0.3	8.2	4	0.1	0.1	12.065	1.379	26.88	**5.82**
II	1106	0	0	0	0	1	0.1	-0.1	0.4	5.4	2.7	0.2	0	9.535	0.909	20.14	**5.74**
III	1086	0	0	0	0	-0.8	0.2	0	0.1	4.3	2.1	-0.2	0	5.87	0.72	12.89	**5.67**
IV	1036	0	0	0	0	-0.8	0.2	0	0.1	4.3	2.1	0	-0.2	5.87	0.72	12.89	**5.67**

Eastern Ave 1401 Garage

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1005	0.1146	-0.022	-0.026	-0.008	1.059	5.914	0.598	-0.54	-0.44	-0.22	0	0	5.7653	0.847	11.61	**4.97**
I	1025	-0.003	0.0755	0.4051	0.134	0.789	2.456	0.279	0.034	8.272	4.121	0	0	15.998	1.421	33.99	**5.82**
II	1098	-0.037	0.0692	0.4016	0.133	0.793	1.507	-0.57	0.832	8.9	4.433	0	0	16.688	1.502	35.65	**5.84**
III	1075	0.0923	0.0925	0.7134	0.236	-1.9	-0.083	-0.11	0.491	6.679	3.315	0	0	9.6379	1.145	22.27	**5.95**
IV	1049	0.1756	0.2528	1.2225	0.404	-1.91	0.869	0.066	0.389	6.368	3.16	0	0	10.966	1.112	25.04	**6.06**

Grudelsky

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	-0.225	0.0006	0.003	0.001	0.698	-7.99	-0.58	1.184	5.364	2.68	0.429	0.429	2.2556	1.445	11.62	**5.77**
I	1014	-0.174	0	0.0039	0.001	0.275	-6.378	-0.48	0.816	4.48	2.239	0.371	0.077	1.5007	1.17	8.82	**5.70**
II	1108	-0.174	0.0009	0.0039	0.001	0.275	-6.378	-0.17	1.079	4.48	2.239	0.077	0.371	1.7515	1.174	9.329	**5.77**

Holiday Inn - Inner Harbor Hotel

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1005	0.1	0	0	0	0.8	6.6	0.2	0.1	-0.1	-0.1	0	0	7.325	0.931	14.03	**4.91**
I	1024	-0.1	0	0	0	1.2	0.5	0	0.3	7.3	3.7	0	0	12.575	1.229	26.7	**5.74**
II	1101	0	0	0	0	0.9	-0.6	0	0	8.3	4.1	0	0	12.405	1.383	27.09	**5.75**
III	1079	0.6	0	0	0	0	15.4	0.6	0.2	5	2.5	-0.2	0	23.595	2.312	46.97	**5.41**
IV	1042	0.1	0	0	0	-0.2	5	0.2	0.3	7.4	3.7	0	-0.2	16.125	1.415	33.64	**5.74**

Market Center West Apts

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1005	0.1	0	0	0	0.8	6.6	0.4	-0.2	-0.1	-0.1	0	0	7.04	0.931	13.58	**4.89**
I	1021	0	0	0	0	0.9	-0.6	0.1	-0.1	8.3	4.1	0	0	12.31	1.383	26.96	**5.76**
II	1100	0	0	0	0	1	-0.2	-0.3	0.4	8.2	4.1	0	0	13.175	1.37	28.3	**5.74**
III	1079	0.6	0	0	0	0	15.4	0.9	0.2	5	2.5	-0.2	0	23.595	2.311	46.97	**5.41**
IV	1043	0.6	0	0	0	0	15.4	0.5	0.3	5	2.5	0	-0.2	23.69	2.312	47.16	**5.42**

Marriot Hotel

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1005	0.1042	-0.031	-0.088	-0.029	1.36	5.008	0	-0.06	-0.53	-0.26	0	0	5.3614	0.73	10.55	**4.96**
I	1024	-0.089	0.0719	0.3941	0.13	0.986	0.007	0	0.004	9.476	4.719	0	0	15.363	1.579	33.2	**5.78**
II	1099	-0.113	0.0703	0.3949	0.13	0.948	-0.979	0	0.196	10.03	4.993	0	0	15.311	1.676	33.98	**5.85**
III	1078	-0.074	0.0247	0.2974	0.098	-2.3	-2.532	0	0.312	8.515	4.229	0	0	8.5698	1.49	22.32	**5.99**
IV	1044	-0.074	0.1911	0.2974	0.098	-2.3	-2.534	0	0.079	8.523	4.233	0	0	8.5288	1.491	22.24	**5.99**

Table B10. Reliability indices for moment capacity based on the proposed load factors, $\varphi = 0.95$,

	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L
Load Factor	1.25	1.5	1.75	1.75	1	1.35	1.35	1.35	1.35	1.35	1.35	1.35
	0.9	0.65	1.75	1.75	1	0.75	0.75	0.75	0.75	0.75	0.75	0.75
V_Q	1.05	1.03	1.25	1.25	0.9	1	0.95	0.95	1	0.95	1	1
λ_Q	0.1	0.08	0.18	0.18	0.15	0.14	0.15	0.15	0.15	0.15	0.12	0.12

λ_R	V_R	ϕ
1.140	0.080	0.95

36 S

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1002	-0.297	-4E-04	0.001	4E-04	1.32	-10	-1.3	1.22	11.9	5.92	0.3	0.36	10.13	2.43	21.15	**4.67**
I	1021	-0.061	-0.008	0.006	0.002	1.09	0.263	0.58	1.47	8.3	4.15	0.48	0.77	16	1.41	23.02	**4.38**
II	1101	-0.061	4E-04	0.006	0.002	1.09	0.263	-0.59	1.63	8.29	4.15	0.77	0.48	16.14	1.41	23.23	**4.4**
III	1090	-0.011	0.006	0.054	0.018	0.4	-0.1	-0.04	0.67	4.58	2.29	-0.24	0.02	7.729	0.77	11.02	**4.08**
IV	1032	-0.011	-0.008	0.054	0.018	0.4	-0.1	0	0.08	4.6	2.29	0.02	-0.24	7.175	0.77	10.19	**3.92**

100 S. South Charles Street

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1002	-0.3	0	0	0	1.2	-11.6	-2.5	1.4	11.6	5.7	0.2	0.3	8.01	2.53	19.11	**4.61**
I	1007	-0.1	0	0	0	1	-4.4	-1.9	1.1	6.6	3.3	0.1	0.6	7.875	1.28	14.11	**4.78**
II	1103	-0.3	0	0	0	1.2	-8.1	-1.3	0.6	12.5	6.2	0.2	0.2	12.03	2.37	22.58	**4.56**
III	1089	0	0	0	0	0.6	0.1	0	0.1	4.2	2.1	0	0	6.93	0.7	9.868	**4.04**
IV	1031	0	0	0	0	0.4	0.1	0	1.1	5.2	2.6	0	-0.1	9.075	0.88	13.13	**4.26**

Bromo Seltzer Tower

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	0.3	0	0	0	-1.1	18.7	2.3	-1	38.1	-9.7	-0.2	-0.2	45.96	6.44	71.19	**4.04**
I	1006	0.2	0	0	0	-0.9	11.1	0.9	-0.8	35.2	-7	-0.1	-0.3	38.29	5.6	58.64	**3.85**
II	1116	0.2	0	0	0	-0.9	11.1	1.8	-0.7	35.3	-7	-0.3	-0.1	38.49	5.61	58.86	**3.85**
III	1089	0	0	0	0	0.6	0.4	0	1.2	22.7	4.7	0.1	0	29.34	3.48	41.98	**3.77**
IV	1031	0	0	0	0	0.4	0.2	0	0	24.2	5.2	0	-0.1	29.7	3.71	42.41	**3.65**

Eastern Ave 1401 Garage

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1006	0.812	0.316	1.655	0.546	-0.92	-34.1	-0.04	1.18	26.4	13.1	0	0	8.421	6.48	35.54	**4.49**
I	1016	0.013	0.037	0.737	0.243	-0.49	14.85	1.93	-0.72	10.6	5.3	0	0	30.43	2.73	44.55	**4.5**
II	1090	0.335	0.388	2.48	0.818	-2.26	-17.5	-2.34	4.23	44.7	22.3	0	0	54.44	7.85	92.16	**4.66**
III	1087	0.02	0.481	2.984	0.985	-2.53	-8.6	-1.75	3.16	41.9	20.8	0	0	58.28	7.09	92.26	**4.54**
IV	1035	0.441	0.612	2.984	0.985	-2.53	-8.6	0.18	1.33	41.9	20.8	0	0	57.16	7.08	90.47	**4.5**

Grudelsky

Segment	Member	DL	SDL	LL	I	B	EV	ES-V	ES-H	EH1	EH2	EHL	HER	Mean Q	□	factor Q	β
Key	1001	-0.225	6E-04	0.003	0.001	0.7	-7.99	-0.58	1.18	5.36	2.68	0.43	0.43	2.256	1.44	8.553	**4.64**
I	1014	-0.174	0	0.004	0.001	0.27	-6.38	-0.48	0.82	4.48	2.24	0.37	0.08	1.501	1.17	6.445	**4.53**
II	1108	-0.174	9E-04	0.004	0.001	0.27	-6.38	-0.17	1.08	4.48	2.24	0.08	0.37	1.751	1.17	6.82	**4.61**
III	1089	-0.007	0.006	0.054	0.018	0.24	-0.15	-0.03	0.14	0.27	0.13	-0.14	-0	0.665	0.06	0.93	**4.01**
IV	1031	-0.009	-0.004	0.054	0.018	0.23	-0.12	-0.01	0	0.68	0.34	-0	-0.2	1.151	0.12	1.561	**3.59**

Holiday Inn - Inner Harbor Hotel

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	0.9	0	0	0	0.1	42.3	1.8	-0.3	-15	-7.6	0	0	20.83	6.43	45.7	**3.86**
I	1016	0.3	0	0	0	1.1	18.2	0.5	0.5	6.1	3	0	0	28.93	2.75	41.06	**4.13**
II	1105	0.3	0	0	0	1.2	21.6	1.1	-0.2	3.7	1.8	0	0	28.22	3.09	40.01	**3.86**
III	1076	-0.6	0	-0.1	0	0	-14.1	-0.5	-0.2	27.8	13.9	-0.6	0	25.99	5.02	46.74	**4.31**
IV	1032	-1	0	0	0	1.7	-21.6	-1.2	1.1	29.6	14.8	0	-0.2	23.59	5.78	48.29	**4.48**

Market Center West Apts

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	0.9	0	0	0	0.1	42.3	1.3	-0.6	-15	-7.6	0	0	20.55	6.43	43.08	**3.86**
I	1016	0.3	0	0	0	1.1	18.2	1	-0.3	6.1	3	0	0	28.17	2.75	40.11	**4.11**
II	1093	-0.9	0	0	0	0.5	-26.7	-2.1	1.7	32.4	16.2	-0.3	0	22.21	6.56	49.84	**4.46**
III	1076	-0.6	0	-0.1	0	0	-14.1	-0.9	1.6	27.8	13.9	-0.6	0	27.7	5.03	49.17	**4.39**
IV	1032	-1	0	0	0	1.7	-21.6	-0.5	-0.2	29.6	14.8	0	-0.2	22.35	5.78	46.57	**4.42**

Marriot Hotel

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1005	-0.819	0.339	1.941	0.64	-1.69	-34.8	0	-0.14	29.3	14.6	0	0.45	9.249	6.89	38.2	**4.5**
I	1029	-0.914	0.522	2.054	0.678	-2.78	-27.6	0	0.28	45.3	22.5	0	0.59	39.79	8.46	77.95	**4.62**
II	1091	-0.827	0.359	2.422	0.799	-3.08	-23.5	0	0.61	48	23.9	0	-0.02	47.71	8.63	86.88	**4.59**
III	1087	-0.549	0.479	3.007	0.992	-3.28	-13.7	0	1.42	44.7	22.2	0	-0	54.42	7.68	90.41	**4.57**
IV	1034	-0.626	0.595	2.884	0.952	-3.29	-16.2	0	0.3	46.6	23.2	0	0.02	53.53	8.08	90.74	**4.55**

Table B11. Reliability indices for moment capacity based on the proposed load factors, $\varphi = 0.90$,

	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L
Load Factor	1.25	1.5	1.75	1.75	1	1.35	1.35	1.35	1.35	1.35	1.35	1.35
	0.9	0.65	1.75	1.75	1	0.75	0.75	0.75	0.75	0.75	0.75	0.75
λ_Q	1.05	1.03	1.25	1.25	0.9	1	0.95	0.95	1	0.95	1	1
V_Q	0.1	0.08	0.18	0.18	0.15	0.14	0.15	0.15	0.15	0.15	0.12	0.12

λ_R	V_R	ϕ
1.140	0.080	0.90

36 S

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1002	-0.297	-4E-04	0.001	4E-04	1.32	-10	-1.3	1.22	11.9	5.92	0.3	0.36	10.13	2.43	22.32	**5.04**
I	1021	-0.061	-0.008	0.006	0.002	1.09	0.263	0.58	1.47	8.3	4.15	0.48	0.77	16	1.41	24.3	**4.85**
II	1101	-0.061	4E-04	0.006	0.002	1.09	0.263	-0.59	1.63	8.29	4.15	0.77	0.48	16.14	1.41	24.52	**4.87**
III	1090	-0.011	0.006	0.054	0.018	0.4	-0.1	-0.04	0.67	4.58	2.29	-0.24	0.02	7.729	0.77	11.63	**4.54**
IV	1032	-0.011	-0.008	0.054	0.018	0.4	-0.1	0	0.08	4.6	2.29	0.02	-0.24	7.175	0.77	10.76	**4.38**

100 S. South Charles Street

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1002	-0.3	0	0	0	1.2	-11.6	-2.5	1.4	11.6	5.7	0.2	0.3	8.01	2.53	20.17	**4.95**
I	1007	-0.1	0	0	0	1	-4.4	-1.9	1.1	6.6	3.3	0.1	0.6	7.875	1.28	14.89	**5.18**
II	1103	-0.3	0	0	0	1.2	-8.1	-1.3	0.6	12.5	6.2	0.2	0.2	12.03	2.37	23.83	**4.94**
III	1089	0	0	0	0	0.6	0.1	0	0.1	4.2	2.1	0	0	6.93	0.7	10.42	**4.50**
IV	1031	0	0	0	0	0.4	0.1	0	1.1	5.2	2.6	0	-0.1	9.075	0.88	13.86	**4.72**

Bromo Seltzer Tower

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	0.3	0	0	0	-1.1	18.7	2.3	-1	38.1	-9.7	-0.2	-0.2	45.96	6.44	75.14	**4.46**
I	1006	0.2	0	0	0	-0.9	11.1	0.9	-0.8	35.2	-7	-0.1	-0.3	38.29	5.6	61.89	**4.27**
II	1116	0.2	0	0	0	-0.9	11.1	1.8	-0.7	35.3	-7	-0.3	-0.1	38.49	5.61	62.13	**4.27**
III	1089	0	0	0	0	0.6	0.4	0	1.2	22.7	4.7	0.1	0	29.34	3.48	44.32	**4.22**
IV	1031	0	0	0	0	0.4	0.2	0	0	24.2	5.2	0	-0.1	29.7	3.71	44.76	**4.09**

Eastern Ave 1401 Garage

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1006	0.812	0.316	1.655	0.546	-0.92	-34.1	-0.04	1.18	26.4	13.1	0	0	8.421	6.48	37.51	**4.76**
I	1016	0.013	0.037	0.737	0.243	-0.49	14.85	1.93	-0.72	10.6	5.3	0	0	30.43	2.73	47.02	**4.96**
II	1090	0.335	0.388	2.48	0.818	-2.26	-17.5	-2.34	4.23	44.7	22.3	0	0	54.44	7.85	97.28	**5.08**
III	1087	0.02	0.481	2.984	0.985	-2.53	-8.6	-1.75	3.16	41.9	20.8	0	0	58.28	7.09	97.39	**4.98**
IV	1035	0.441	0.612	2.984	0.985	-2.53	-8.6	0.18	1.33	41.9	20.8	0	0	57.16	7.08	95.5	**4.93**

Grudelsky

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	-0.225	6E-04	0.003	0.001	0.7	-7.99	-0.58	1.18	5.36	2.68	0.43	0.43	2.256	1.44	9.029	**4.93**
I	1014	-0.174	0	0.004	0.001	0.27	-6.38	-0.48	0.82	4.48	2.24	0.37	0.08	1.501	1.17	6.803	**4.8**
II	1108	-0.174	9E-04	0.004	0.001	0.27	-6.38	-0.17	1.08	4.48	2.24	0.08	0.37	1.751	1.17	7.199	**4.89**
III	1089	-0.007	0.006	0.054	0.018	0.24	-0.15	-0.03	0.14	0.27	0.13	-0.14	-0	0.665	0.06	0.982	**4.48**
IV	1031	-0.009	-0.004	0.054	0.018	0.23	-0.12	-0.01	0	0.68	0.34	-0	-0.2	1.151	0.12	1.648	**4.06**

Holiday Inn - Inner Harbor Hotel

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	0.9	0	0	0	0.1	42.3	1.8	-0.3	-15	-7.6	0	0	20.83	6.43	45.73	**4.18**
I	1016	0.3	0	0	0	1.1	18.2	0.5	0.5	6.1	3	0	0	28.93	2.75	43.34	**4.60**
II	1105	0.3	0	0	0	1.2	21.6	1.1	-0.2	3.7	1.8	0	0	28.22	3.09	42.23	**4.31**
III	1076	-0.6	0	-0.1	0	0	-14.1	-0.5	-0.2	27.8	13.9	-0.6	0	25.99	5.02	49.34	**4.69**
IV	1032	-1	0	0	0	1.7	-21.6	-1.2	1.1	29.6	14.8	0	-0.2	23.59	5.78	50.97	**4.84**

Market Center West Apts

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	0.9	0	0	0	0.1	42.3	1.3	-0.6	-15	-7.6	0	0	20.55	6.43	45.48	**4.18**
I	1016	0.3	0	0	0	1.1	18.2	1	-0.3	6.1	3	0	0	28.17	2.75	42.34	**4.58**
II	1093	-0.9	0	0	0	0.5	-26.7	-2.1	1.7	32.4	16.2	-0.3	0	22.21	6.56	52.61	**4.80**
III	1076	-0.6	0	-0.1	0	0	-14.1	-0.9	1.6	27.8	13.9	-0.6	0	27.7	5.03	51.91	**4.79**
IV	1032	-1	0	0	0	1.7	-21.6	-0.5	-0.2	29.6	14.8	0	-0.2	22.35	5.78	49.16	**4.78**

Marriot Hotel

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1005	-0.819	0.339	1.941	0.64	-1.69	-34.8	0	-0.14	29.3	14.6	0	0.45	9.249	6.89	40.32	**4.78**
I	1029	-0.914	0.522	2.054	0.678	-2.78	-27.6	0	0.28	45.3	22.5	0	0.59	39.79	8.46	82.28	**5.00**
II	1091	-0.827	0.359	2.422	0.799	-3.08	-23.5	0	0.61	48	23.9	0	-0.02	47.71	8.63	91.7	**4.98**
III	1087	-0.549	0.479	3.007	0.992	-3.28	-13.7	0	1.42	44.7	22.2	0	-0	54.42	7.68	95.44	**4.99**
IV	1034	-0.626	0.595	2.884	0.952	-3.29	-16.2	0	0.3	46.6	23.2	0	0.02	53.53	8.08	95.78	**4.96**

Table B12. Reliability indices for moment capacity based on the proposed load factors, $\varphi = 0.85$,

	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L
Load	1.25	1.5	1.75	1.75	1	1.35	1.35	1.35	1.35	1.35	1.35	1.35
Factor	0.9	0.65	1.75	1.75	1	0.75	0.75	0.75	0.75	0.75	0.75	0.75
λ_Q	1.05	1.03	1.25	1.25	0.9	1	0.95	0.95	1	0.95	1	1
V_Q	0.1	0.08	0.18	0.18	0.15	0.14	0.15	0.15	0.15	0.15	0.12	0.12

λ_R	V_R	ϕ
1.140	0.080	0.85

36 S

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1002	-0.3	-0	0.001	0	1.321	-10	-1.3	1.22	11.9	5.923	0.3	0.36	10.13	2.43	23.64	**5.42**
I	1021	-0.06	-0.01	0.006	0	1.086	0.263	0.58	1.47	8.3	4.146	0.48	0.77	16	1.41	25.73	**5.34**
II	1101	-0.06	4E-04	0.006	0	1.086	0.263	-0.59	1.63	8.29	4.146	0.77	0.48	16.14	1.41	25.96	**5.36**
III	1090	-0.01	0.006	0.054	0.02	0.395	-0.1	-0.04	0.67	4.58	2.293	-0.2	0.02	7.729	0.77	12.31	**5.03**
IV	1032	-0.01	-0.01	0.054	0.02	0.395	-0.1	0	0.08	4.6	2.293	0.02	-0.24	7.175	0.77	11.39	**4.85**

100 S. South Charles Street

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1002	-0.3	0	0	0	1.2	-11.6	-2.5	1.4	11.6	5.7	0.2	0.3	8.01	2.53	21.35	**5.31**
I	1007	-0.1	0	0	0	1	-4.4	-1.9	1.1	6.6	3.3	0.1	0.6	7.875	1.28	15.77	**5.61**
II	1103	-0.3	0	0	0	1.2	-8.1	-1.3	0.6	12.5	6.2	0.2	0.2	12.03	2.37	25.24	**5.35**
III	1089	0	0	0	0	0.6	0.1	0	0.1	4.2	2.1	0	0	6.93	0.7	11.03	**4.98**
IV	1031	0	0	0	0	0.4	0.1	0	1.1	5.2	2.6	0	-0.1	9.075	0.88	14.68	**5.2**

Bromo Seltzer Tower

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	0.3	0	0	0	-1.1	18.7	2.3	-1	38.1	-9.7	-0.2	-0.2	45.96	6.44	79.56	**4.91**
I	1006	0.2	0	0	0	-0.9	11.1	0.9	-0.8	35.2	-7	-0.1	-0.3	38.29	5.6	65.54	**4.71**
II	1116	0.2	0	0	0	-0.9	11.1	1.8	-0.7	35.3	-7	-0.3	-0.1	38.49	5.61	65.78	**4.71**
III	1089	0	0	0	0	0.6	0.4	0	1.2	22.7	4.7	0.1	0	29.34	3.48	46.92	**4.69**
IV	1031	0	0	0	0	0.4	0.2	0	0	24.2	5.2	0	-0.1	29.7	3.71	47.39	**4.55**

Eastern Ave 1401 Garage

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1006	0.812	0.316	1.655	0.55	-0.92	-34.1	-0.04	1.18	26.4	13.11	0	0	8.421	6.48	39.72	**5.06**
I	1016	0.013	0.037	0.737	0.24	-0.49	14.85	1.93	-0.7	10.6	5.296	0	0	30.43	2.73	49.79	**5.45**
II	1090	0.335	0.388	2.48	0.82	-2.26	-17.5	-2.34	4.23	44.7	22.25	0	0	54.44	7.85	103	**5.51**
III	1087	0.02	0.481	2.984	0.98	-2.53	-8.6	-1.75	3.16	41.9	20.83	0	0	58.28	7.09	103.1	**5.43**
IV	1035	0.441	0.612	2.984	0.98	-2.53	-8.6	0.18	1.33	41.9	20.84	0	0	57.16	7.08	101.1	**5.39**

Grudelsky

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	B
Key	1001	-0.23	6E-04	0.003	0	0.698	-7.99	-0.58	1.18	5.36	2.68	0.43	0.43	2.256	1.44	9.56	**5.24**
I	1014	-0.17	0	0.004	0	0.275	-6.38	-0.48	0.82	4.48	2.239	0.37	0.08	1.501	1.17	7.203	**5.10**
II	1108	-0.17	9E-04	0.004	0	0.275	-6.38	-0.17	1.08	4.48	2.239	0.08	0.37	1.751	1.17	7.622	**5.20**
III	1089	-0.01	0.006	0.054	0.02	0.238	-0.15	-0.03	0.14	0.27	0.134	-0.1	-0	0.665	0.06	1.04	**4.97**
IV	1031	-0.01	-0	0.054	0.02	0.233	-0.12	-0.01	0	0.68	0.339	-0	-0.2	1.151	0.12	1.745	**4.54**

Holiday Inn - Inner Harbor Hotel

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	B
Key	1001	0.9	0	0	0	0.1	42.3	1.8	-0.3	-15	-7.6	0	0	20.83	6.43	48.42	**4.53**
I	1016	0.3	0	0	0	1.1	18.2	0.5	0.5	6.1	3	0	0	28.93	2.75	45.89	**5.08**
II	1105	0.3	0	0	0	1.2	21.6	1.1	-0.2	3.7	1.8	0	0	28.22	3.09	44.72	**4.78**
III	1076	-0.6	0	-0.1	0	0	-14.1	-0.5	-0.2	27.8	13.9	-0.6	0	25.99	5.02	52.24	**5.10**
IV	1032	-1	0	0	0	1.7	-21.6	-1.2	1.1	29.6	14.8	0	-0.2	23.59	5.78	53.97	**5.22**

Market Center West Apts

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	B
Key	1001	0.9	0	0	0	0.1	42.3	1.3	-0.6	-15	-7.6	0	0	20.55	6.43	48.15	**4.53**
I	1016	0.3	0	0	0	1.1	18.2	1	-0.3	6.1	3	0	0	28.17	2.75	44.83	**5.06**
II	1093	-0.9	0	0	0	0.5	-26.7	-2.1	1.7	32.4	16.2	-0.3	0	22.21	6.56	55.7	**5.17**
III	1076	-0.6	0	-0.1	0	0	-14.1	-0.9	1.6	27.8	13.9	-0.6	0	27.7	5.03	54.96	**5.20**
IV	1032	-1	0	0	0	1.7	-21.6	-0.5	-0.2	29.6	14.8	0	-0.2	22.35	5.78	52.05	**5.15**

Marriot Hotel

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	B
Key	1005	-0.82	0.339	1.941	0.64	-1.69	-34.8	0	-0.1	29.3	14.57	0	0.45	9.249	6.89	42.7	**5.08**
I	1029	-0.91	0.522	2.054	0.68	-2.78	-27.6	0	0.28	45.3	22.53	0	0.59	39.79	8.46	87.13	**5.40**
II	1091	-0.83	0.359	2.422	0.8	-3.08	-23.5	0	0.61	48	23.85	0	-0.02	47.71	8.63	97.1	**5.40**
III	1087	-0.55	0.479	3.007	0.99	-3.28	-13.7	0	1.42	44.7	22.21	0	-0	54.42	7.68	101.1	**5.43**
IV	1034	-0.63	0.595	2.884	0.95	-3.29	-16.2	0	0.3	46.6	23.16	0	0.02	53.53	8.08	101.4	**5.40**

Table B13. Reliability indices for shear capacity based on the proposed load factors, $\varphi = 0.90$,

	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L
Load Factor	1.25	1.5	1.75	1.75	1	1.35	1.35	1.35	1.35	1.35	1.35	1.35
	0.9	0.65	1.75	1.75	1	0.75	0.75	0.75	0.75	0.75	0.75	0.75
λ_Q	1.05	1.03	1.25	1.25	0.9	1	0.95	0.95	1	0.95	1	1
V_Q	0.1	0.08	0.18	0.18	0.15	0.14	0.15	0.15	0.15	0.15	0.12	0.12

λ_R	V_R	ϕ
1.200	0.150	0.90

36 S

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.2	4E-04	0.01	0.002	37.76	33.19	5.013	1.929	19.9	9.98	0.59	0.6	100.8	7.66	142.99	**3.17**
I	1006	1.262	5E-04	0.01	0.002	37.75	35.14	5.313	1.826	18	9.02	0.54	0.66	99.93	7.7	141.55	**3.16**
II	1095	1.451	0.006	0.05	0.016	42.17	38.8	6.57	4.709	9.51	4.76	4.41	0.49	101.5	8.07	142.99	**3.11**
III	1089	1.149	0.011	0.09	0.031	44.76	28.33	4.806	5.153	8.3	4.15	5.56	0.55	92.9	7.42	129.66	**3.06**
IV	1025	1.441	0.017	0.04	0.013	41.58	39.9	5.495	1.296	9.09	4.54	0.48	4.04	97.83	8.08	137.34	**3.07**

100 S. South Charles Street

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.4	0	0	0	42.4	37.6	7.8	3.1	24.1	11.9	0.8	0.9	117.2	8.76	166.66	**3.20**
I	1007	1.4	0	0	0	42.5	40.4	8.4	3.1	21.4	10.6	0.7	1	116.2	8.82	164.97	**3.18**
II	1116	1.4	0	0	0	42.5	39.5	8.2	2.9	22.3	11	0.8	0.8	116.2	8.8	165.12	**3.18**
III	1089	1.3	0	0.1	0	50.4	32.7	6.1	1.6	8.4	4.2	6.4	0.7	100.2	8.36	139	**3.00**
IV	1032	1.2	0	0.1	0	50.4	32.7	7.6	6.9	8.4	4.2	0.6	6.6	105.2	8.41	146.96	**3.06**

Bromo Seltzer Tower

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.2	0	0	0	25.4	51.7	2.3	1.9	38.1	18.8	0.5	0.5	134.5	10.2	197.14	**3.36**
I	1007	1.3	0	0	0	25.4	56.1	0.5	1.8	34	16.8	0.4	0.6	132.9	10.3	194.58	**3.34**
II	1116	1.2	0	0	0	25.4	54.6	1.8	1.9	35.3	17.4	0.5	0.4	133.2	10.2	195.04	**3.35**
III	1089	1.3	0	0	0	32.6	45.2	0	5.1	22.7	11.2	6.2	0.3	120.3	8.64	174.08	**3.31**
IV	1031	1.2	0	0	0	32.3	48.6	0	1.1	24.2	11.9	0.3	6.1	121.6	9.05	175.86	**3.29**

Eastern Ave 1401 Garage

Segment	Joint	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.133	0.095	0.49	0.16	46.13	30.97	1.699	4.116	51.1	25.5	0	0	153.6	11.4	221.71	**3.28**
I	1007	1.124	0.092	0.48	0.159	46.29	32.92	1.865	3.398	49.7	24.8	0	0	153	11.3	220.71	**3.28**
II	1114	1.151	0.086	0.48	0.159	46.29	33.83	1.63	4.61	49	24.5	0	0	154	11.3	222.23	**3.29**
III	1062	1.29	-0.03	-0.4	-0.12	52.66	21.89	1.502	3.273	54.8	27.4	0	0	154	12	220.31	**3.21**
IV	1058	1.277	-0.03	-0.5	-0.17	52.65	21.93	1.429	2.972	54.1	27	0	0	152.5	11.9	217.84	**3.20**

Grudelsky

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	B
Key	1001	1.258	0.001	0.01	0.002	42.15	40.47	4.549	1.908	11.4	5.7	0.63	0.63	99.57	8.26	139.73	**3.07**
I	1006	1.319	0.001	0.01	0.002	42.13	42.59	4.778	1.846	9.33	4.67	0.59	0.66	98.62	8.39	138.21	**3.05**
II	1115	1.342	0.001	0.01	0.002	42.13	43.34	4.821	1.581	8.58	4.29	0.66	0.59	98.03	8.44	137.27	**3.03**
III	1089	0.757	0.012	0.1	0.033	49.15	24.06	3.55	5.857	1.79	0.9	5.29	0.52	82.95	7.52	113.55	**2.87**
IV	1031	0.693	0.014	0.09	0.031	48.81	25.46	2.467	0.959	1.92	0.96	0.51	5.32	79.54	7.52	108.16	**2.81**

Holiday Inn - Inner Harbor Hotel

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	B
Key	1001	1.2	0	0	0	33	47.5	2.1	1.7	79.7	40.2	0.1	0.1	198.2	15.5	292.28	**3.38**
I	1007	1.3	0	0	0	33.2	49.5	2.2	1.9	78.2	39.4	0.1	0.2	198.5	15.4	292.64	**3.38**
II	1116	1.3	0	0	0	33.1	49.2	2.2	1.5	78.2	39.4	0.1	0.1	197.6	15.4	291.33	**3.38**
III	1079	2.9	0	0	0	38.3	58.1	2.3	0.9	73.7	37	0.7	0.1	206.1	15.6	302.33	**3.36**
IV	1041	3	0	0	0	35.7	61.1	3.2	2.4	72.4	36.3	0.1	0.3	205.9	15.5	302.73	**3.38**

Market Center West Apts

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	B
Key	1001	1.2	0	0	0	33	47.5	2	2.7	79.7	40.2	0.1	0.1	199.1	15.5	293.78	**3.38**
I	1007	1.3	0	0	0	33.2	49.5	2	2.3	78.2	39.4	0.1	0.2	198.8	15.4	293.24	**3.38**
II	1116	1.3	0	0	0	33.1	49.2	2	3	78.2	39.4	0.1	0.1	199	15.4	293.58	**3.38**
III	1079	2.9	0	0	0	38.3	58.1	3.8	3.6	73.7	37	0.7	0.1	208.6	15.6	306.38	**3.37**
IV	1041	3	0	0	0	35.7	61.1	1.9	1.5	72.4	36.3	0.1	0.3	205.1	15.5	301.38	**3.37**

Marriot Hotel

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.193	0.104	0.48	0.159	57.38	25.88	0	0.425	53.8	26.9	0	0	159.3	12.4	227.3	**3.20**
I	1006	1.162	0.097	0.46	0.152	57.51	26.74	0	0.399	52.8	26.4	0	0	158.7	12.3	226.34	**3.20**
II	1116	1.19	0.098	0.47	0.154	57.47	26.98	0	0.476	52.6	26.3	0	0	158.7	12.3	226.37	**3.20**
III	1061	1.39	0.028	-0.1	-0.02	63.35	17.88	0	1.209	57.7	28.8	0	0	162.5	13.1	230.52	**3.15**
IV	1059	1.384	0.026	-0.2	-0.05	63.34	17.9	0	1.136	57.4	28.6	0	0	161.8	13	229.51	**3.14**

Table B14. Reliability indices for shear capacity based on the proposed load factors, $\varphi = 0.85$,

		DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L
Load Factor		1.25	1.5	1.75	1.75	1	1.35	1.35	1.35	1.35	1.35	1.35	1.35
		0.9	0.65	1.75	1.75	1	0.75	0.75	0.75	0.75	0.75	0.75	0.75
λ_Q		1.05	1.03	1.25	1.25	0.9	1	0.95	0.95	1	0.95	1	1
V_Q		0.1	0.08	0.18	0.18	0.15	0.14	0.15	0.15	0.15	0.15	0.12	0.12

λ_R	V_R	ϕ
1.200	0.150	0.85

36 S

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.2	4E-04	0.01	0.002	37.76	33.19	5.013	1.929	19.9	9.98	0.59	0.6	100.8	7.66	151.4	**3.49**
I	1006	1.262	5E-04	0.01	0.002	37.75	35.14	5.313	1.826	18	9.02	0.54	0.66	99.93	7.7	149.87	**3.48**
II	1095	1.451	0.006	0.05	0.016	42.17	38.8	6.57	4.709	9.51	4.76	4.41	0.49	101.5	8.07	151.4	**3.43**
III	1089	1.149	0.011	0.09	0.031	44.76	28.33	4.806	5.153	8.3	4.15	5.56	0.55	92.9	7.42	137.28	**3.38**
IV	1025	1.441	0.017	0.04	0.013	41.58	39.9	5.495	1.296	9.09	4.54	0.48	4.04	97.83	8.08	145.42	**3.39**

100 S. South Charles Street

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.4	0	0	0	42.4	37.6	7.8	3.1	24.1	11.9	0.8	0.9	117.2	8.76	176.46	**3.51**
I	1007	1.4	0	0	0	42.5	40.4	8.4	3.1	21.4	10.6	0.7	1	116.2	8.82	174.67	**3.50**
II	1116	1.4	0	0	0	42.5	39.5	8.2	2.9	22.3	11	0.8	0.8	116.2	8.8	174.83	**3.50**
III	1089	1.3	0	0.1	0	50.4	32.7	6.1	1.6	8.4	4.2	6.4	0.7	100.2	8.36	147.18	**3.32**
IV	1032	1.2	0	0.1	0	50.4	32.7	7.6	6.9	8.4	4.2	0.6	6.6	105.2	8.41	155.61	**3.38**

Bromo Seltzer Tower

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.2	0	0	0	25.4	51.7	2.3	1.9	38.1	18.8	0.5	0.5	134.5	10.2	208.74	**3.66**
I	1007	1.3	0	0	0	25.4	56.1	0.5	1.8	34	16.8	0.4	0.6	132.9	10.3	206.02	**3.65**
II	1116	1.2	0	0	0	25.4	54.6	1.8	1.9	35.3	17.4	0.5	0.4	133.2	10.2	206.51	**3.65**
III	1089	1.3	0	0	0	32.6	45.2	0	5.1	22.7	11.2	6.2	0.3	120.3	8.64	184.32	**3.62**
IV	1031	1.2	0	0	0	32.3	48.6	0	1.1	24.2	11.9	0.3	6.1	121.6	9.05	186.2	**3.60**

Eastern Ave 1401 Garage

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.133	0.095	0.49	0.16	46.13	30.97	1.699	4.116	51.1	25.5	0	0	153.6	11.4	234.75	**3.59**
I	1007	1.124	0.092	0.48	0.159	46.29	32.92	1.865	3.398	49.7	24.8	0	0	153	11.3	233.69	**3.59**
II	1114	1.151	0.086	0.48	0.159	46.29	33.83	1.63	4.61	49	24.5	0	0	154	11.3	235.3	**3.60**
III	1062	1.29	-0.03	-0.4	-0.12	52.66	21.89	1.502	3.273	54.8	27.4	0	0	154	12	233.27	**3.52**
IV	1058	1.277	-0.03	-0.5	-0.17	52.65	21.93	1.429	2.972	54.1	27	0	0	152.5	11.9	230.66	**3.52**

Grudelsky

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.258	0.001	0.01	0.002	42.15	40.47	4.549	1.908	11.4	5.7	0.63	0.63	99.57	8.26	147.95	**3.39**
I	1006	1.319	0.001	0.01	0.002	42.13	42.59	4.778	1.846	9.33	4.67	0.59	0.66	98.62	8.39	146.34	**3.36**
II	1115	1.342	0.001	0.01	0.002	42.13	43.34	4.821	1.581	8.58	4.29	0.66	0.59	98.03	8.44	145.34	**3.35**
III	1089	0.757	0.012	0.1	0.033	49.15	24.06	3.55	5.857	1.79	0.9	5.29	0.52	82.95	7.52	120.23	**3.20**
IV	1031	0.693	0.014	0.09	0.031	48.81	25.46	2.467	0.959	1.92	0.96	0.51	5.32	79.54	7.52	114.52	**3.13**

Holiday Inn - Inner Harbor Hotel

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.2	0	0	0	33	47.5	2.1	1.7	79.7	40.2	0.1	0.1	198.2	15.5	309.48	**3.68**
I	1007	1.3	0	0	0	33.2	49.5	2.2	1.9	78.2	39.4	0.1	0.2	198.5	15.4	309.86	**3.68**
II	1116	1.3	0	0	0	33.1	49.2	2.2	1.5	78.2	39.4	0.1	0.1	197.6	15.4	308.47	**3.68**
III	1079	2.9	0	0	0	38.3	58.1	2.3	0.9	73.7	37	0.7	0.1	206.1	15.6	320.12	**3.67**
IV	1041	3	0	0	0	35.7	61.1	3.2	2.4	72.4	36.3	0.1	0.3	205.9	15.5	320.54	**3.68**

Market Center West Apts

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.2	0	0	0	33	47.5	2	2.7	79.7	40.2	0.1	0.1	199.1	15.5	311.06	**3.69**
I	1007	1.3	0	0	0	33.2	49.5	2	2.3	78.2	39.4	0.1	0.2	198.8	15.4	310.49	**3.69**
II	1116	1.3	0	0	0	33.1	49.2	2	3	78.2	39.4	0.1	0.1	199	15.4	310.85	**3.69**
III	1079	2.9	0	0	0	38.3	58.1	3.8	3.6	73.7	37	0.7	0.1	208.6	15.6	324.41	**3.68**
IV	1041	3	0	0	0	35.7	61.1	1.9	1.5	72.4	36.3	0.1	0.3	205.1	15.5	319.11	**3.68**

Marriot Hotel

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.193	0.104	0.48	0.159	57.38	25.88	0	0.425	53.8	26.9	0	0	159.3	12.4	240.67	**3.51**
I	1006	1.162	0.097	0.46	0.152	57.51	26.74	0	0.399	52.8	26.4	0	0	158.7	12.3	239.65	**3.51**
II	1116	1.19	0.098	0.47	0.154	57.47	26.98	0	0.476	52.6	26.3	0	0	158.7	12.3	239.69	**3.51**
III	1061	1.39	0.028	-0.1	-0.02	63.35	17.88	0	1.209	57.7	28.8	0	0	162.5	13.1	244.08	**3.46**
IV	1059	1.384	0.026	-0.2	-0.05	63.34	17.9	0	1.136	57.4	28.6	0	0	161.8	13	243.01	**3.46**

Table B15. Reliability indices for shear capacity based on the proposed load factors, $\varphi = 0.80$

	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L
Load	1.25	1.5	1.75	1.75	1	1.35	1.35	1.35	1.35	1.35	1.35	1.35
Factor	0.9	0.65	1.75	1.75	1	0.75	0.75	0.75	0.75	0.75	0.75	0.75
λ_Q	1.05	1.03	1.25	1.25	0.9	1	0.95	0.95	1	0.95	1	1
V_Q	0.1	0.08	0.18	0.18	0.15	0.14	0.15	0.15	0.15	0.15	0.12	0.12

λ_R	V_R	ϕ
1.200	0.150	0.80

36 S

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.2	4E-04	0.01	0.002	37.76	33.19	5.013	1.929	19.9	9.98	0.59	0.6	100.8	7.659	160.86	**3.81**
I	1006	1.262	5E-04	0.01	0.002	37.75	35.14	5.313	1.826	18	9.02	0.54	0.66	99.93	7.697	159.24	**3.79**
II	1095	1.451	0.006	0.05	0.016	42.17	38.8	6.57	4.709	9.51	4.76	4.41	0.49	101.5	8.07	160.87	**3.75**
III	1089	1.149	0.011	0.09	0.031	44.76	28.33	4.806	5.153	8.3	4.15	5.56	0.55	92.9	7.423	145.86	**3.70**
IV	1025	1.441	0.017	0.04	0.013	41.58	39.9	5.495	1.296	9.09	4.54	0.48	4.04	97.83	8.078	154.51	**3.71**

100 S. South Charles Street

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	Β
Key	1001	1.4	0	0	0	42.4	37.6	7.8	3.1	24.1	11.9	0.8	0.9	117.2	8.755	187.49	**3.83**
I	1007	1.4	0	0	0	42.5	40.4	8.4	3.1	21.4	10.6	0.7	1	116.2	8.817	185.59	**3.82**
II	1116	1.4	0	0	0	42.5	39.5	8.2	2.9	22.3	11	0.8	0.8	116.2	8.795	185.76	**3.82**
III	1089	1.3	0	0.1	0	50.4	32.7	6.1	1.6	8.4	4.2	6.4	0.7	100.2	8.355	156.38	**3.65**
IV	1032	1.2	0	0.1	0	50.4	32.7	7.6	6.9	8.4	4.2	0.6	6.6	105.2	8.411	165.33	**3.70**

Bromo Seltzer Tower

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	Β
Key	1001	1.2	0	0	0	25.4	51.7	2.3	1.9	38.1	18.8	0.5	0.5	134.5	10.2	221.78	**3.97**
I	1007	1.3	0	0	0	25.4	56.1	0.5	1.8	34	16.8	0.4	0.6	132.9	10.26	218.9	**3.96**
II	1116	1.2	0	0	0	25.4	54.6	1.8	1.9	35.3	17.4	0.5	0.4	133.2	10.22	219.42	**3.96**
III	1089	1.3	0	0	0	32.6	45.2	0	5.1	22.7	11.2	6.2	0.3	120.3	8.637	195.84	**3.94**
IV	1031	1.2	0	0	0	32.3	48.6	0	1.1	24.2	11.9	0.3	6.1	121.6	9.049	197.84	**3.92**

Eastern Ave 1401 Garage

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	Β
Key	1001	1.133	0.095	0.49	0.16	46.13	30.97	1.699	4.116	51.1	25.5	0	0	153.6	11.39	249.42	**3.91**
I	1007	1.124	0.092	0.48	0.159	46.29	32.92	1.865	3.398	49.7	24.8	0	0	153	11.34	248.3	**3.90**
II	1114	1.151	0.086	0.48	0.159	46.29	33.83	1.63	4.61	49	24.5	0	0	154	11.32	250	**3.91**
III	1062	1.29	-0.03	-0.4	-0.12	52.66	21.89	1.502	3.273	54.8	27.4	0	0	154	11.95	247.85	**3.84**
IV	1058	1.277	-0.03	-0.5	-0.17	52.65	21.93	1.429	2.972	54.1	27	0	0	152.5	11.86	245.07	**3.83**

Grudelsky

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.258	0.001	0.01	0.002	42.15	40.47	4.549	1.908	11.4	5.7	0.63	0.63	99.57	8.257	157.19	**3.71**
I	1006	1.319	0.001	0.01	0.002	42.13	42.59	4.778	1.846	9.33	4.67	0.59	0.66	98.62	8.391	155.48	**3.69**
II	1115	1.342	0.001	0.01	0.002	42.13	43.34	4.821	1.581	8.58	4.29	0.66	0.59	98.03	8.443	154.43	**3.68**
III	1089	0.757	0.012	0.1	0.033	49.15	24.06	3.55	5.857	1.79	0.9	5.29	0.52	82.95	7.518	127.74	**3.53**
IV	1031	0.693	0.014	0.09	0.031	48.81	25.46	2.467	0.959	1.92	0.96	0.51	5.32	79.54	7.524	121.68	**3.46**

Holiday Inn - Inner Harbor Hotel

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.2	0	0	0	33	47.5	2.1	1.7	79.7	40.2	0.1	0.1	198.2	15.49	328.82	**3.99**
I	1007	1.3	0	0	0	33.2	49.5	2.2	1.9	78.2	39.4	0.1	0.2	198.5	15.41	329.23	**3.99**
II	1116	1.3	0	0	0	33.1	49.2	2.2	1.5	78.2	39.4	0.1	0.1	197.6	15.38	327.75	**3.99**
III	1079	2.9	0	0	0	38.3	58.1	2.3	0.9	73.7	37	0.7	0.1	206.1	15.59	340.13	**3.98**
IV	1041	3	0	0	0	35.7	61.1	3.2	2.4	72.4	36.3	0.1	0.3	205.9	15.53	340.58	**3.99**

Market Center West Apts

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.2	0	0	0	33	47.5	2	2.7	79.7	40.2	0.1	0.1	199.1	15.49	330.51	**4.00**
I	1007	1.3	0	0	0	33.2	49.5	2	2.3	78.2	39.4	0.1	0.2	198.8	15.41	329.9	**4.00**
II	1116	1.3	0	0	0	33.1	49.2	2	3	78.2	39.4	0.1	0.1	199	15.39	330.28	**4.00**
III	1079	2.9	0	0	0	38.3	58.1	3.8	3.6	73.7	37	0.7	0.1	208.6	15.6	344.68	**3.99**
IV	1041	3	0	0	0	35.7	61.1	1.9	1.5	72.4	36.3	0.1	0.3	205.1	15.53	339.06	**3.99**

Marriot Hotel

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	1.193	0.104	0.48	0.159	57.38	25.88	0	0.425	53.8	26.9	0	0	159.3	12.37	255.71	**3.83**
I	1006	1.162	0.097	0.46	0.152	57.51	26.74	0	0.399	52.8	26.4	0	0	158.7	12.29	254.63	**3.83**
II	1116	1.19	0.098	0.47	0.154	57.47	26.98	0	0.476	52.6	26.3	0	0	158.7	12.28	254.67	**3.83**
III	1061	1.39	0.028	-0.1	-0.02	63.35	17.88	0	1.209	57.7	28.8	0	0	162.5	13.08	259.34	**3.78**
IV	1059	1.384	0.026	-0.2	-0.05	63.34	17.9	0	1.136	57.4	28.6	0	0	161.8	13.04	258.2	**3.78**

Table B16. Reliability indices for compression capacity based on the proposed load factors, $\varphi = 0.80$

	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L
Load Factor	1.25	1.5	1.75	1.75	1	1.35	1.35	1.35	1.35	1.35	1.35	1.35
	0.9	0.65	1.75	1.75	1	0.75	0.75	0.75	0.75	0.75	0.75	0.75
λ_Q	1.05	1.03	1.25	1.25	0.9	1	0.95	0.95	1	0.95	1	1
V_Q	0.1	0.08	0.18	0.18	0.15	0.14	0.15	0.15	0.15	0.15	0.12	0.12

λ_R	V_R	ϕ
1.180	0.120	0.80

36 S

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1005	-0.036	-4E-04	0	1E-04	1.153	-1.649	-0.39	0.09	2.634	1.313	0.045	-0.04	3.3601	0.519	6.702	**4.84**
I	1025	0.0099	-6E-04	0.0039	0.012	1.389	0.942	0.332	0.462	3.214	1.603	0.156	0.181	7.7144	0.585	12.85	**4.65**
II	1097	0.0099	0.0015	0.0039	0.012	1.389	0.942	-0.08	0.605	3.206	1.603	0.182	0.156	7.8441	0.587	13.08	**4.66**

100 S. South Charles Street

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1005	0	0	0	0	1.2	-1.7	-0.1	0.2	2.2	1.1	0	-0.1	2.715	0.466	5.719	**4.95**
I	1025	0.1	0	0	0	1.6	2.6	0.1	0.9	3.2	1.6	0.2	0.3	10.22	0.693	17.01	**4.71**
II	1107	0	0	0	0	1.5	0.1	0.3	0.3	3.2	1.6	0.2	0.1	6.755	0.571	11.16	**4.51**
III	1087	0	0	0	0	-1.4	0.1	0	0	1.8	0.9	-0.3	0	1.195	0.356	2.694	**4.11**
IV	1035	0	0	0	0	-1.4	0.1	0	-0.1	1.8	0.9	0	-0.3	1.1	0.356	2.6	**4.13**

Bromo Seltzer Tower

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1004	0	0	0	0	0.7	-2.1	-0.4	0.1	2.9	1.4	0	-0.1	2.855	0.57	6.238	**4.85**
I	1025	0	0	0	0	1.1	-1.4	0	0.3	8.2	4	0.1	0.1	12.065	1.379	21.49	**4.64**
II	1106	0	0	0	0	1	0.1	-0.1	0.4	5.4	2.7	0.2	0	9.535	0.909	16.1	**4.53**
III	1086	0	0	0	0	-0.8	0.2	0	0.1	4.3	2.1	-0.2	0	5.87	0.72	10.12	**4.36**
IV	1036	0	0	0	0	-0.8	0.2	0	0.1	4.3	2.1	0	-0.2	5.87	0.72	10.12	**4.36**

Eastern Ave 1401 Garage

Segment	Member	DL	SDL	LL	I	B	EV	ES-V	ES-H	EH1	EH2	EHL	HER	Mean Q	□	factor Q	β
Key	1005	0.1146	-0.022	-0.026	-0.008	1.059	5.914	0.598	-0.54	-0.44	-0.22	0	0	5.7653	0.847	10.26	**4.20**
I	1025	-0.003	0.0755	0.4051	0.134	0.789	2.456	0.279	0.034	8.272	4.121	0	0	15.998	1.421	27.42	**4.68**
II	1098	-0.037	0.0692	0.4016	0.133	0.793	1.507	-0.57	0.832	8.9	4.433	0	0	16.688	1.502	28.69	**4.69**
III	1075	0.0923	0.0925	0.7134	0.236	-1.9	-0.083	-0.11	0.491	6.679	3.315	0	0	9.6379	1.145	17.63	**4.74**
IV	1049	0.1756	0.2528	1.2225	0.404	-1.91	0.869	0.066	0.389	6.368	3.16	0	0	10.966	1.112	20.13	**4.94**

Grudelsky

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	-0.225	0.0006	0.003	0.001	0.698	-7.99	-0.58	1.184	5.364	2.68	0.429	0.429	2.2556	1.445	10.16	**5.16**
I	1014	-0.174	0	0.0039	0.001	0.275	-6.378	-0.48	0.816	4.48	2.239	0.371	0.077	1.5007	1.17	7.653	**5.07**
II	1108	-0.174	0.0009	0.0039	0.001	0.275	-6.378	-0.17	1.079	4.48	2.239	0.077	0.371	1.7515	1.174	8.099	**5.13**

Holiday Inn - Inner Harbor Hotel

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	B
Key	1005	0.1	0	0	0	0.8	6.6	0.2	0.1	-0.1	-0.1	0	0	7.325	0.931	12.28	**4.15**
I	1024	-0.1	0	0	0	1.2	0.5	0	0.3	7.3	3.7	0	0	12.575	1.229	21.3	**4.53**
II	1101	0	0	0	0	0.9	-0.6	0	0	8.3	4.1	0	0	12.405	1.383	21.49	**4.51**
III	1079	0.6	0	0	0	0	15.4	0.6	0.2	5	2.5	-0.2	0	23.595	2.312	39.73	**4.49**
IV	1042	0.1	0	0	0	-0.2	5	0.2	0.3	7.4	3.7	0	-0.2	16.125	1.415	27.39	**4.64**

Market Center West Apts

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	B
Key	1005	0.1	0	0	0	0.8	6.6	0.4	-0.2	-0.1	-0.1	0	0	7.04	0.931	11.92	**4.15**
I	1021	0	0	0	0	0.9	-0.6	0.1	-0.1	8.3	4.1	0	0	12.31	1.383	21.39	**4.52**
II	1100	0	0	0	0	1	-0.2	-0.3	0.4	8.2	4.1	0	0	13.175	1.37	22.49	**4.51**
III	1079	0.6	0	0	0	0	15.4	0.9	0.2	5	2.5	-0.2	0	23.595	2.311	39.73	**4.49**
IV	1043	0.6	0	0	0	0	15.4	0.5	0.3	5	2.5	0	-0.2	23.69	2.312	39.9	**4.49**

Marriot Hotel

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	B
Key	1005	0.1042	-0.031	-0.088	-0.029	1.36	5.008	0	-0.06	-0.53	-0.26	0	0	5.3614	0.73	9.239	**4.21**
I	1024	-0.089	0.0719	0.3941	0.13	0.986	0.007	0	0.004	9.476	4.719	0	0	15.363	1.579	26.39	**4.55**
II	1099	-0.113	0.0703	0.3949	0.13	0.948	-0.979	0	0.196	10.03	4.993	0	0	15.311	1.676	27.1	**4.66**
III	1078	-0.074	0.0247	0.2974	0.098	-2.3	-2.532	0	0.312	8.515	4.229	0	0	8.5698	1.49	17.61	**4.79**
IV	1044	-0.074	0.1911	0.2974	0.098	-2.3	-2.534	0	0.079	8.523	4.233	0	0	8.5288	1.491	17.55	**4.79**

Table B17. Reliability indices for compression capacity based on the proposed load factors, $\varphi = 0.75$

	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L
Load Factor	1.25	1.5	1.75	1.75	1	1.35	1.35	1.35	1.35	1.35	1.35	1.35
	0.9	0.65	1.75	1.75	1	0.75	0.75	0.75	0.75	0.75	0.75	0.75
λ_Q	1.05	1.03	1.25	1.25	0.9	1	0.95	0.95	1	0.95	1	1
V_Q	0.1	0.08	0.18	0.18	0.15	0.14	0.15	0.15	0.15	0.15	0.12	0.12

λ_R	V_R	ϕ
1.180	0.120	0.75

36 S

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1005	-0.036	-4E-04	0	1E-04	1.153	-1.649	-0.39	0.09	2.634	1.313	0.045	-0.04	3.3601	0.519	7.149	**5.18**
I	1025	0.0099	-6E-04	0.0039	0.012	1.389	0.942	0.332	0.462	3.214	1.603	0.156	0.181	7.7144	0.585	13.71	**5.01**
II	1097	0.0099	0.0015	0.0039	0.012	1.389	0.942	-0.08	0.605	3.206	1.603	0.182	0.156	7.8441	0.587	13.96	**5.03**

100 S. South Charles Street

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1005	0	0	0	0	1.2	-1.7	-0.1	0.2	2.2	1.1	0	-0.1	2.715	0.466	6.1	**5.29**
I	1025	0.1	0	0	0	1.6	2.6	0.1	0.9	3.2	1.6	0.2	0.3	10.22	0.693	18.14	**5.07**
II	1107	0	0	0	0	1.5	0.1	0.3	0.3	3.2	1.6	0.2	0.1	6.755	0.571	11.9	**4.88**
III	1087	0	0	0	0	-1.4	0.1	0	0	1.8	0.9	-0.3	0	1.195	0.356	2.873	**4.43**
IV	1035	0	0	0	0	-1.4	0.1	0	-0.1	1.8	0.9	0	-0.3	1.1	0.356	2.773	**4.45**

Bromo Seltzer Tower

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1004	0	0	0	0	0.7	-2.1	-0.4	0.1	2.9	1.4	0	-0.1	2.855	0.57	6.653	**5.19**
I	1025	0	0	0	0	1.1	-1.4	0	0.3	8.2	4	0.1	0.1	12.065	1.379	22.93	**5**
II	1106	0	0	0	0	1	0.1	-0.1	0.4	5.4	2.7	0.2	0	9.535	0.909	17.17	**4.9**
III	1086	0	0	0	0	-0.8	0.2	0	0.1	4.3	2.1	-0.2	0	5.87	0.72	10.79	**4.73**
IV	1036	0	0	0	0	-0.8	0.2	0	0.1	4.3	2.1	0	-0.2	5.87	0.72	10.79	**4.73**

Eastern Ave 1401 Garage

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1005	0.1146	-0.022	-0.026	-0.008	1.059	5.914	0.598	-0.54	-0.44	-0.22	0	0	5.7653	0.847	10.95	**4.65**
I	1025	-0.003	0.0755	0.4051	0.134	0.789	2.456	0.279	0.034	8.272	4.121	0	0	15.998	1.421	29.24	**5.05**
II	1098	-0.037	0.0692	0.4016	0.133	0.793	1.507	-0.57	0.832	8.9	4.433	0	0	16.688	1.502	30.61	**5.05**
III	1075	0.0923	0.0925	0.7134	0.236	-1.9	-0.083	-0.11	0.491	6.679	3.315	0	0	9.6379	1.145	18.8	**5.10**
IV	1049	0.1756	0.2528	1.2225	0.404	-1.91	0.869	0.066	0.389	6.368	3.16	0	0	10.966	1.112	21.47	**5.29**

Grudelsky

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	B
Key	1001	-0.225	0.0006	0.003	0.001	0.698	-7.99	-0.58	1.184	5.364	2.68	0.429	0.429	2.2556	1.445	10.83	**5.45**
I	1014	-0.174	0	0.0039	0.001	0.275	-6.378	-0.48	0.816	4.48	2.239	0.371	0.077	1.5007	1.17	8.163	**5.35**
II	1108	-0.174	0.0009	0.0039	0.001	0.275	-6.378	-0.17	1.079	4.48	2.239	0.077	0.371	1.7515	1.174	8.638	**5.42**

Holiday Inn - Inner Harbor Hotel

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1005	0.1	0	0	0	0.8	6.6	0.2	0.1	-0.1	-0.1	0	0	7.325	0.931	13.09	**4.52**
I	1024	-0.1	0	0	0	1.2	0.5	0	0.3	7.3	3.7	0	0	12.575	1.229	22.72	**4.89**
II	1101	0	0	0	0	0.9	-0.6	0	0	8.3	4.1	0	0	12.405	1.383	22.92	**4.88**
III	1079	0.6	0	0	0	0	15.4	0.6	0.2	5	2.5	-0.2	0	23.595	2.312	42.38	**4.86**
IV	1042	0.1	0	0	0	-0.2	5	0.2	0.3	7.4	3.7	0	-0.2	16.125	1.415	29.22	**5.01**

Market Center West Apts

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1005	0.1	0	0	0	0.8	6.6	0.4	-0.2	-0.1	-0.1	0	0	7.04	0.931	12.71	**4.53**
I	1021	0	0	0	0	0.9	-0.6	0.1	-0.1	8.3	4.1	0	0	12.31	1.383	22.82	**4.89**
II	1100	0	0	0	0	1	-0.2	-0.3	0.4	8.2	4.1	0	0	13.175	1.37	23.99	**4.87**
III	1079	0.6	0	0	0	0	15.4	0.9	0.2	5	2.5	-0.2	0	23.595	2.311	42.38	**4.86**
IV	1043	0.6	0	0	0	0	15.4	0.5	0.3	5	2.5	0	-0.2	23.69	2.312	42.56	**4.86**

Marriot Hotel

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1005	0.1042	-0.031	-0.088	-0.029	1.36	5.008	0	-0.06	-0.53	-0.26	0	0	5.3614	0.73	9.855	**4.58**
I	1024	-0.089	0.0719	0.3941	0.13	0.986	0.007	0	0.004	9.476	4.719	0	0	15.363	1.579	28.15	**4.92**
II	1099	-0.113	0.0703	0.3949	0.13	0.948	-0.979	0	0.196	10.03	4.993	0	0	15.311	1.676	28.91	**5.02**
III	1078	-0.074	0.0247	0.2974	0.098	-2.3	-2.532	0	0.312	8.515	4.229	0	0	8.5698	1.49	18.78	**5.14**
IV	1044	-0.074	0.1911	0.2974	0.098	-2.3	-2.534	0	0.079	8.523	4.233	0	0	8.5288	1.491	18.72	**5.13**

Table B18. Reliability indices for compression capacity based on the proposed load factors, $\varphi = 0.70$

	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L
Load Factor	1.25	1.5	1.75	1.75	1	1.35	1.35	1.35	1.35	1.35	1.35	1.35
	0.9	0.65	1.75	1.75	1	0.75	0.75	0.75	0.75	0.75	0.75	0.75
λ_Q	1.05	1.03	1.25	1.25	0.9	1	0.95	1	1	0.95	1	1
V_Q	0.1	0.08	0.18	0.18	0.15	0.14	0.15	0.15	0.15	0.15	0.12	0.12

λ_R	V_R	ϕ
1.180	0.120	0.70

36 S

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1005	-0.036	-4E-04	0	1E-04	1.153	-1.649	-0.39	0.09	2.634	1.313	0.045	-0.04	3.3646	0.519	7.659	**5.53**
I	1025	0.0099	-6E-04	0.0039	0.012	1.389	0.942	0.332	0.462	3.214	1.603	0.156	0.181	7.7376	0.585	14.69	**5.37**
II	1097	0.0099	0.0015	0.0039	0.012	1.389	0.942	-0.08	0.605	3.206	1.603	0.182	0.156	7.8744	0.587	14.95	**5.38**

100 S. South Charles Street

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1005	0	0	0	0	1.2	-1.7	-0.1	0.2	2.2	1.1	0	-0.1	2.725	0.466	6.536	**5.62**
I	1025	0.1	0	0	0	1.6	2.6	0.1	0.9	3.2	1.6	0.2	0.3	10.265	0.694	19.44	**5.42**
II	1107	0	0	0	0	1.5	0.1	0.3	0.3	3.2	1.6	0.2	0.1	6.77	0.571	12.75	**5.25**
III	1087	0	0	0	0	-1.4	0.1	0	0	1.8	0.9	-0.3	0	1.195	0.356	3.079	**4.78**
IV	1035	0	0	0	0	-1.4	0.1	0	-0.1	1.8	0.9	0	-0.3	1.095	0.356	2.971	**4.81**

Bromo Seltzer Tower

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1004	0	0	0	0	0.7	-2.1	-0.4	0.1	2.9	1.4	0	-0.1	2.86	0.57	7.129	**5.53**
I	1025	0	0	0	0	1.1	-1.4	0	0.3	8.2	4	0.1	0.1	12.08	1.379	24.56	**5.36**
II	1106	0	0	0	0	1	0.1	-0.1	0.4	5.4	2.7	0.2	0	9.555	0.909	18.4	**5.27**
III	1086	0	0	0	0	-0.8	0.2	0	0.1	4.3	2.1	-0.2	0	5.875	0.72	11.56	**5.11**
IV	1036	0	0	0	0	-0.8	0.2	0	0.1	4.3	2.1	0	-0.2	5.875	0.72	11.56	**5.11**

Eastern Ave 1401 Garage

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1005	0.1146	-0.022	-0.026	-0.008	1.059	5.914	0.598	-0.54	-0.44	-0.22	0	0	5.7384	0.847	11.73	**5.04**
I	1025	-0.003	0.0755	0.4051	0.134	0.789	2.456	0.279	0.034	8.272	4.121	0	0	16	1.421	31.33	**5.41**
II	1098	-0.037	0.0692	0.4016	0.133	0.793	1.507	-0.57	0.832	8.9	4.433	0	0	16.73	1.502	32.79	**5.41**
III	1075	0.0923	0.0925	0.7134	0.236	-1.9	-0.083	-0.11	0.491	6.679	3.315	0	0	9.6624	1.145	20.15	**5.45**
IV	1049	0.1756	0.2528	1.2225	0.404	-1.91	0.869	0.066	0.389	6.368	3.16	0	0	10.985	1.113	23	**5.64**

Grudelsky

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1001	-0.225	0.0006	0.003	0.001	0.698	-7.99	-0.58	1.184	5.364	2.68	0.429	0.429	2.3148	1.446	11.61	**5.73**
I	1014	-0.174	0	0.0039	0.001	0.275	-6.378	-0.48	0.816	4.48	2.239	0.371	0.077	1.5415	1.17	8.746	**5.63**
II	1108	-0.174	0.0009	0.0039	0.001	0.275	-6.378	-0.17	1.079	4.48	2.239	0.077	0.371	1.8055	1.175	9.255	**5.70**

Holiday Inn - Inner Harbor Hotel

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1005	0.1	0	0	0	0.8	6.6	0.2	0.1	-0.1	-0.1	0	0	7.33	0.931	14.03	**4.91**
I	1024	-0.1	0	0	0	1.2	0.5	0	0.3	7.3	3.7	0	0	12.59	1.229	24.34	**5.26**
II	1101	0	0	0	0	0.9	-0.6	0	0	8.3	4.1	0	0	12.405	1.383	24.56	**5.25**
III	1079	0.6	0	0	0	0	15.4	0.6	0.2	5	2.5	-0.2	0	23.605	2.312	45.41	**5.23**
IV	1042	0.1	0	0	0	-0.2	5	0.2	0.3	7.4	3.7	0	-0.2	16.14	1.415	31.31	**5.37**

Market Center West Apts

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1005	0.1	0	0	0	0.8	6.6	0.4	-0.2	-0.1	-0.1	0	0	7.03	0.931	13.62	**4.92**
I	1021	0	0	0	0	0.9	-0.6	0.1	-0.1	8.3	4.1	0	0	12.305	1.383	24.45	**5.26**
II	1100	0	0	0	0	1	-0.2	-0.3	0.4	8.2	4.1	0	0	13.195	1.37	25.71	**5.24**
III	1079	0.6	0	0	0	0	15.4	0.9	0.2	5	2.5	-0.2	0	23.605	2.311	45.41	**5.23**
IV	1043	0.6	0	0	0	0	15.4	0.5	0.3	5	2.5	0	-0.2	23.705	2.312	45.6	**5.24**

Marriot Hotel

Segment	Member	DL	DW	LL	IM	WA	EV	EH1	EH2	ES-V	ES-H	ER-R	ER-L	Mean Q	σ	factor Q	β
Key	1005	0.1042	-0.031	-0.088	-0.029	1.36	5.008	0	-0.06	-0.53	-0.26	0	0	5.3586	0.73	10.56	**4.97**
I	1024	-0.089	0.0719	0.3941	0.13	0.986	0.007	0	0.004	9.476	4.719	0	0	15.363	1.579	30.16	**5.29**
II	1099	-0.113	0.0703	0.3949	0.13	0.948	-0.979	0	0.196	10.03	4.993	0	0	15.321	1.676	30.97	**5.38**
III	1078	-0.074	0.0247	0.2974	0.098	-2.3	-2.532	0	0.312	8.515	4.229	0	0	8.5854	1.49	20.13	**5.48**
IV	1044	-0.074	0.1911	0.2974	0.098	-2.3	-2.534	0	0.079	8.523	4.233	0	0	8.5328	1.491	20.05	**5.48**

Appendix C

Table C1. Statistical Parameters of the Deflection, Bridge Length = 60 [ft.]

Bridge Length = 60 [ft.]						
Sites	daily		weekly		monthly	
	mean	σ	mean	σ	mean	σ
Arizona I	1.27	0.45	2.28	0.41	2.58	0.12
California	1.39	0.29	2.17	0.20	2.4	0.14
Illinois	1.89	0.48	2.54	0.21	2.73	0.07
Indiana	1.36	0.25	1.82	0.27	2.06	0.23
New Mexico II	2.00	0.50	2.63	0.27	2.88	0.17
Combination	1.58	0.39	2.29	0.27	2.53	0.15

Table C2. Statistical Parameters of the Deflection, Bridge Length = 100 [ft.]

Bridge Length = 100 [ft.]						
Sites	daily		weekly		monthly	
	Mean	σ	mean	σ	mean	σ
Arizona I	1.32	0.50	2.4	0.44	2.79	0.14
California	1.46	0.32	2.07	0.32	2.5	0.27
Illinois	2	0.48	2.68	0.23	2.84	0.08
Indiana	1.38	0.28	1.93	0.23	2.15	0.14
Kansas	1.72	0.43	2.5	0.28	2.64	0.14
New Mexico II	2.01	0.5	2.73	0.22	2.94	0.10
Combination	1.71	0.4	2.38	0.26	2.61	0.15

Table C3. Statistical Parameters of the Deflection, Bridge Length = 150 [ft.]

Bridge Length = 150 [ft.]						
Sites	daily		weekly		monthly	
	mean	σ	mean	σ	mean	σ
Arizona I	1.33	0.45	2.21	0.38	2.57	0.13
California	1.38	0.30	1.89	0.20	2.35	0.13
Colorado	1.47	0.35	2.06	0.24	2.20	0.18
Illinois	1.86	0.42	2.45	0.21	2.61	0.07
New Mexico II	1.87	0.42	2.50	0.20	2.66	0.10
Combination	1.65	0.37	2.23	0.21	2.46	0.12

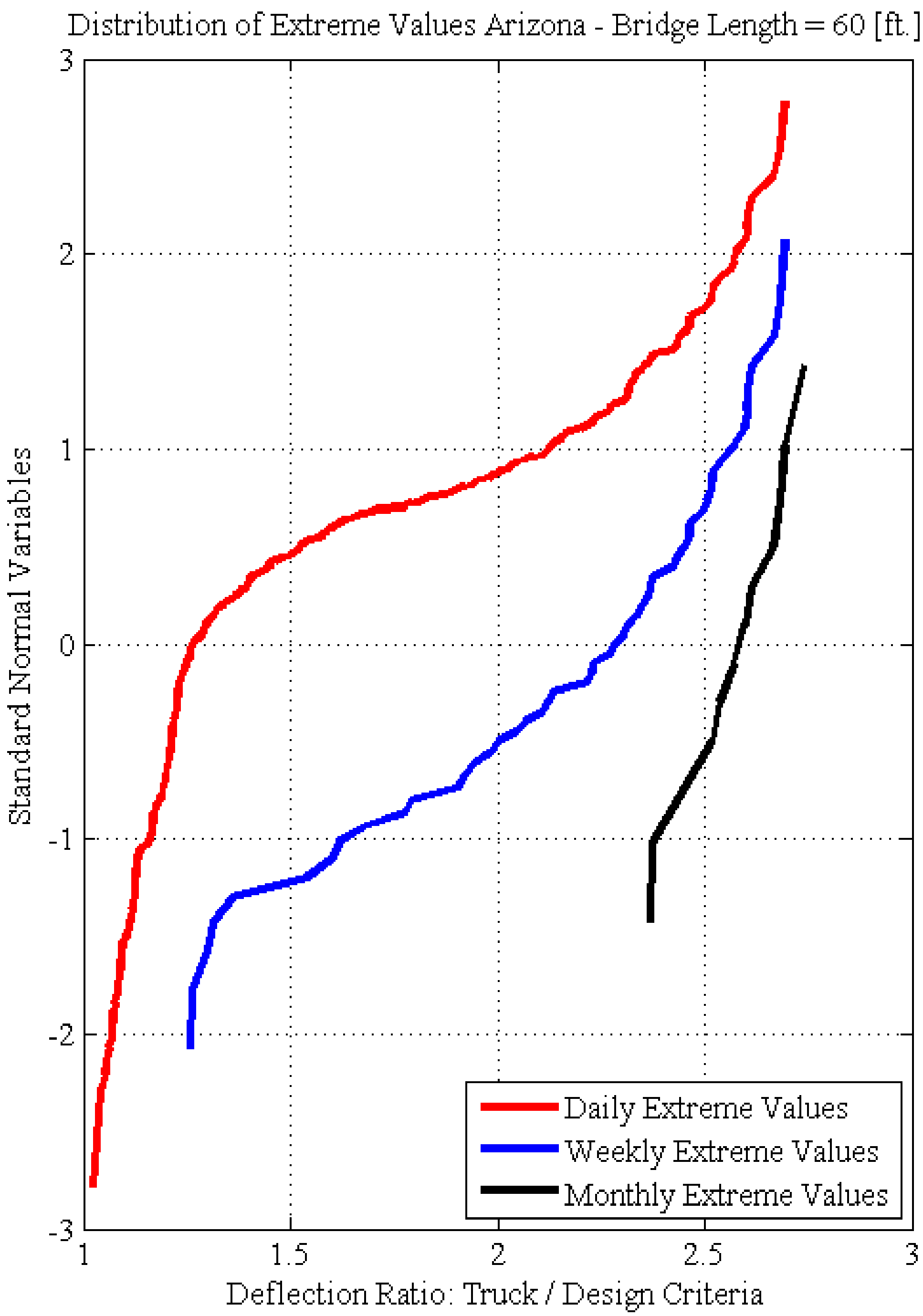

Figure C1. CDF of Deflection Ratio of WIM Deflection to the Design Truck Deflection, Arizona I, Span Length =60 [ft.]

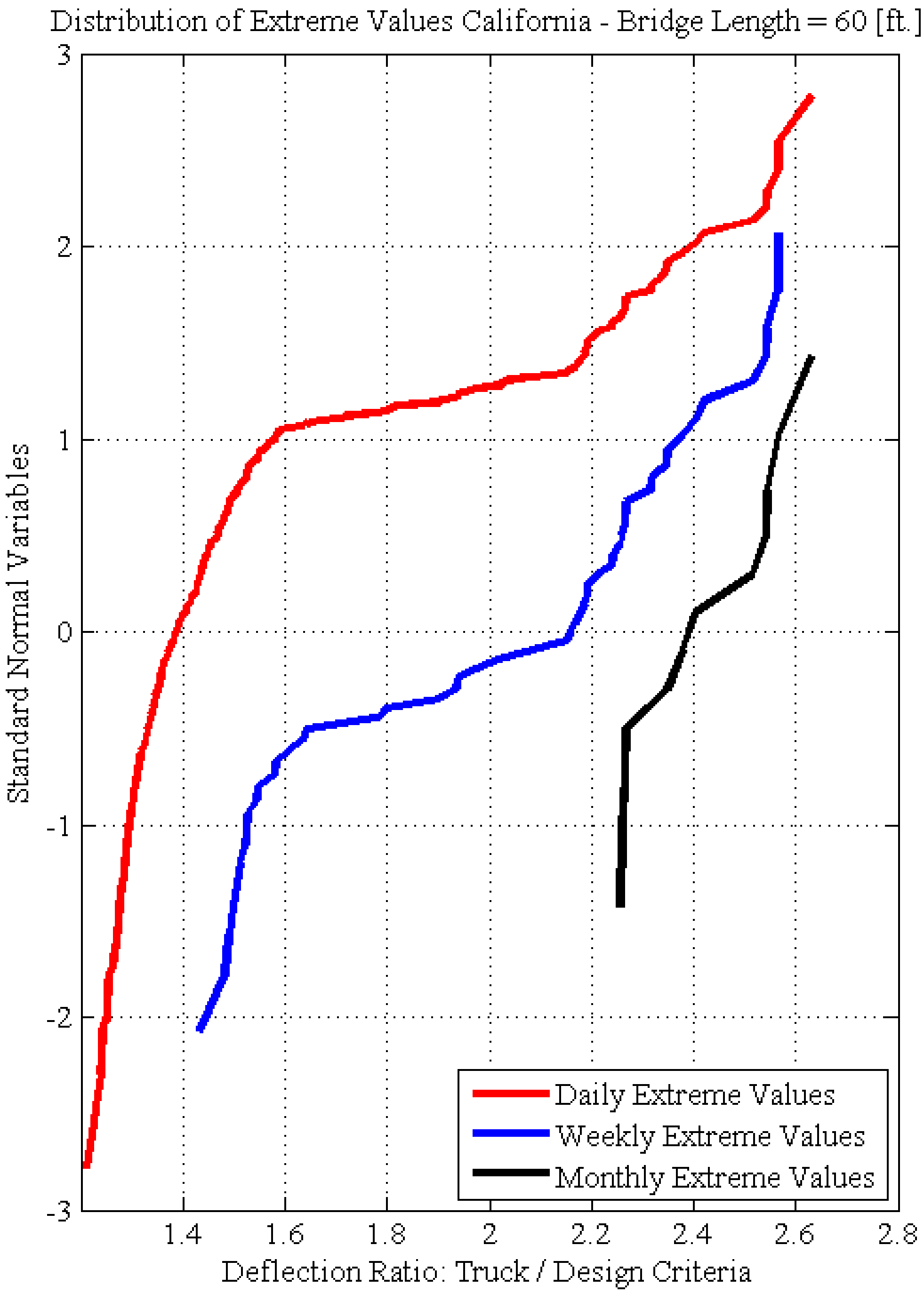

Figure C2. CDF of Deflection Ratio of WIM Deflection to the Design Truck Deflection, California, Span Length =60 [ft.]

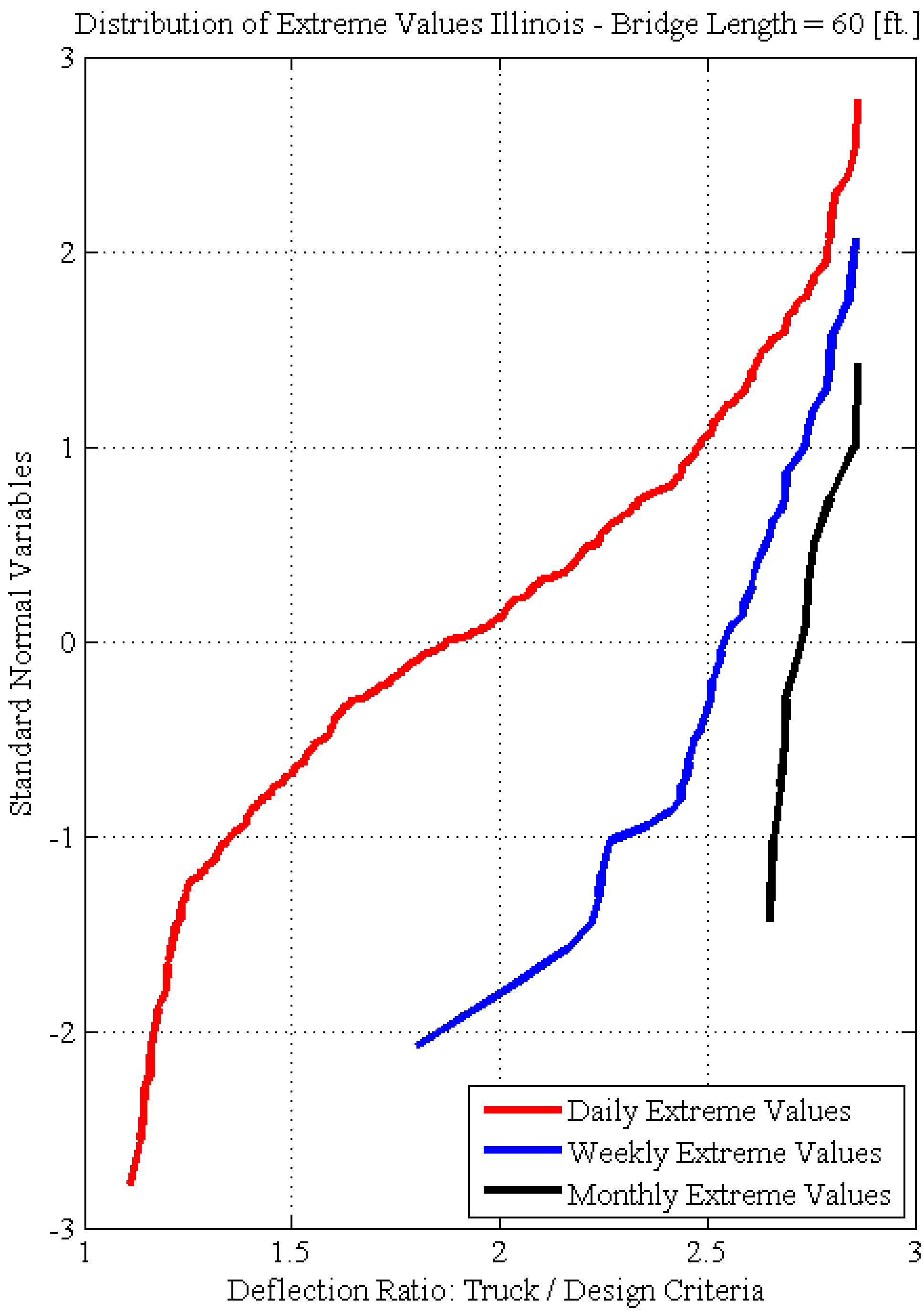

Figure C3. CDF of Deflection Ratio of WIM Deflection to the Design Truck Deflection, Illinois, Span Length =60 [ft.]

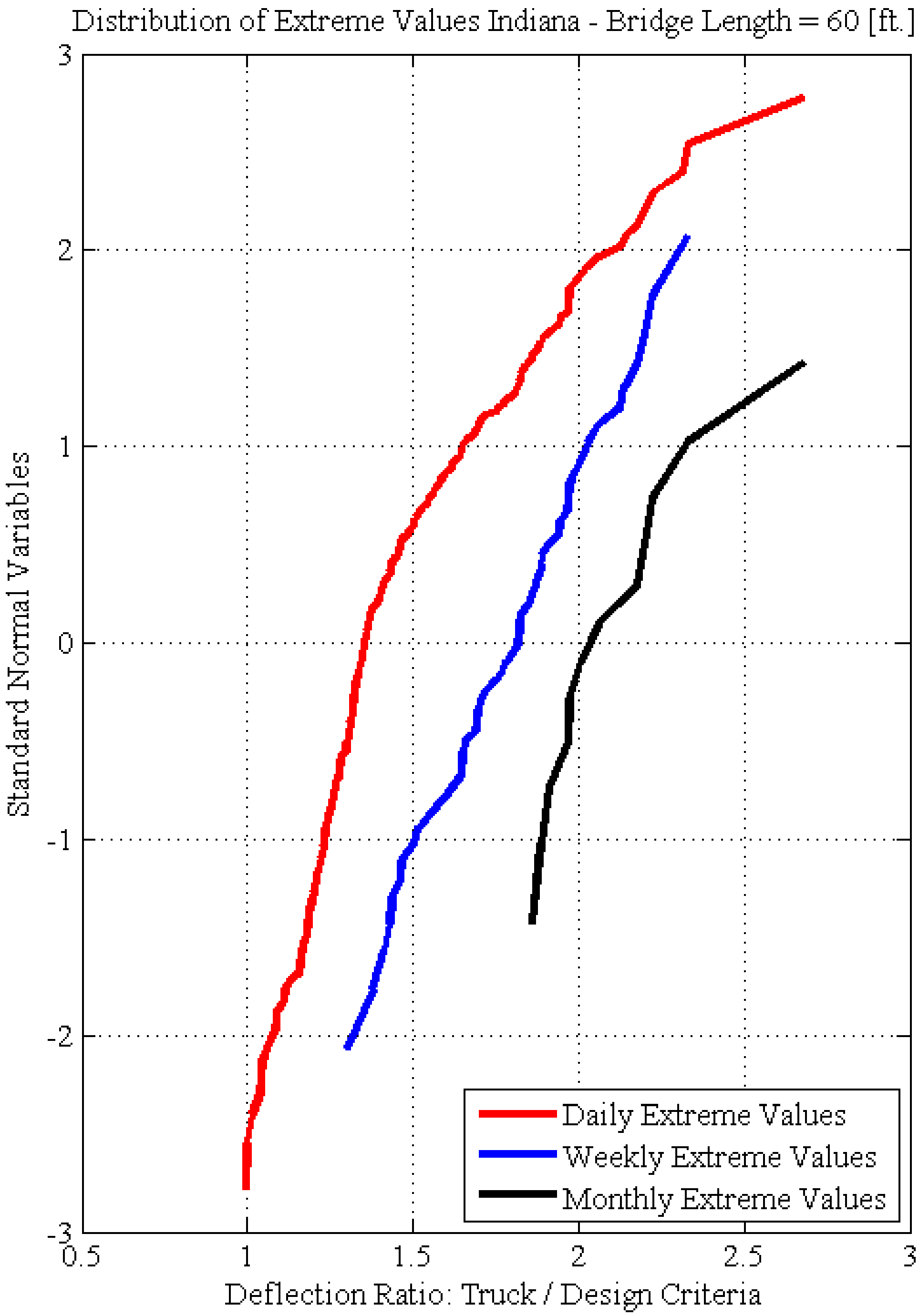

Figure C4. CDF of Deflection Ratio of WIM Deflection to the Design Truck Deflection, Indiana, Span Length =60 [ft.]

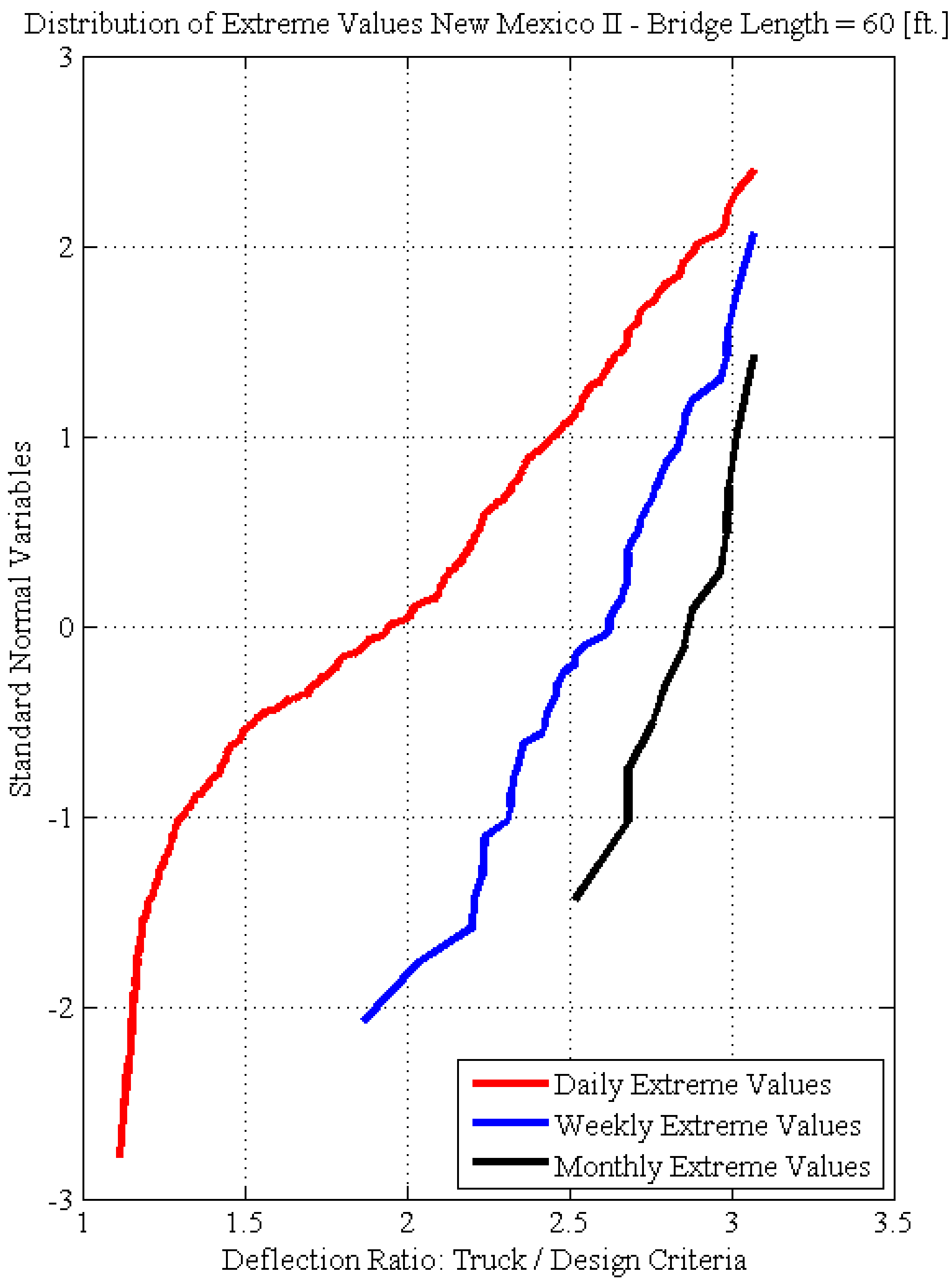

Figure C5. CDF of Deflection Ratio of WIM Deflection to the Design Truck Deflection, New Mexico II, Span Length =60 [ft.]

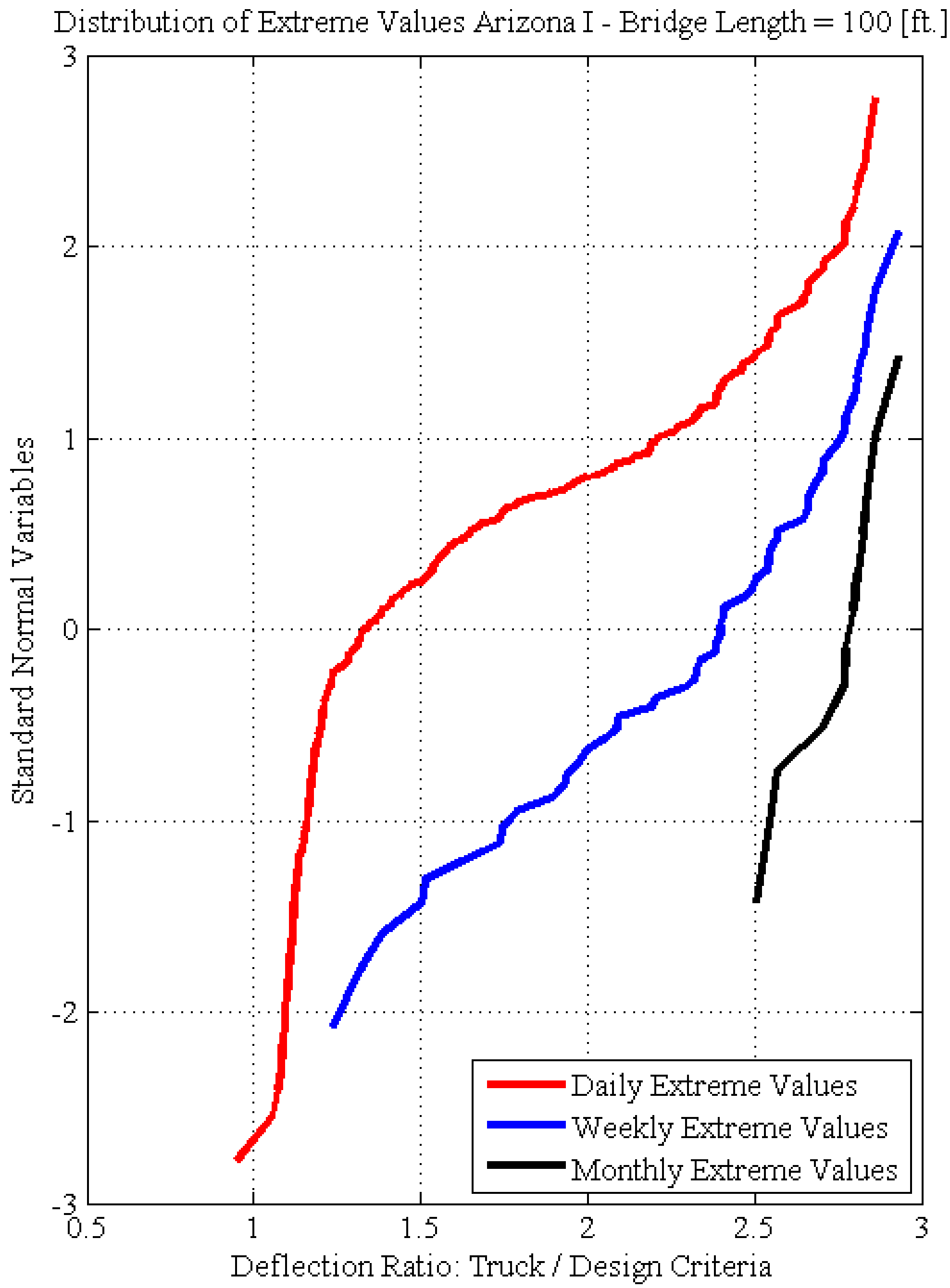

Figure C6. CDF of Deflection Ratio of WIM Deflection to the Design Truck Deflection, Arizona I, Span Length =100 [ft.]

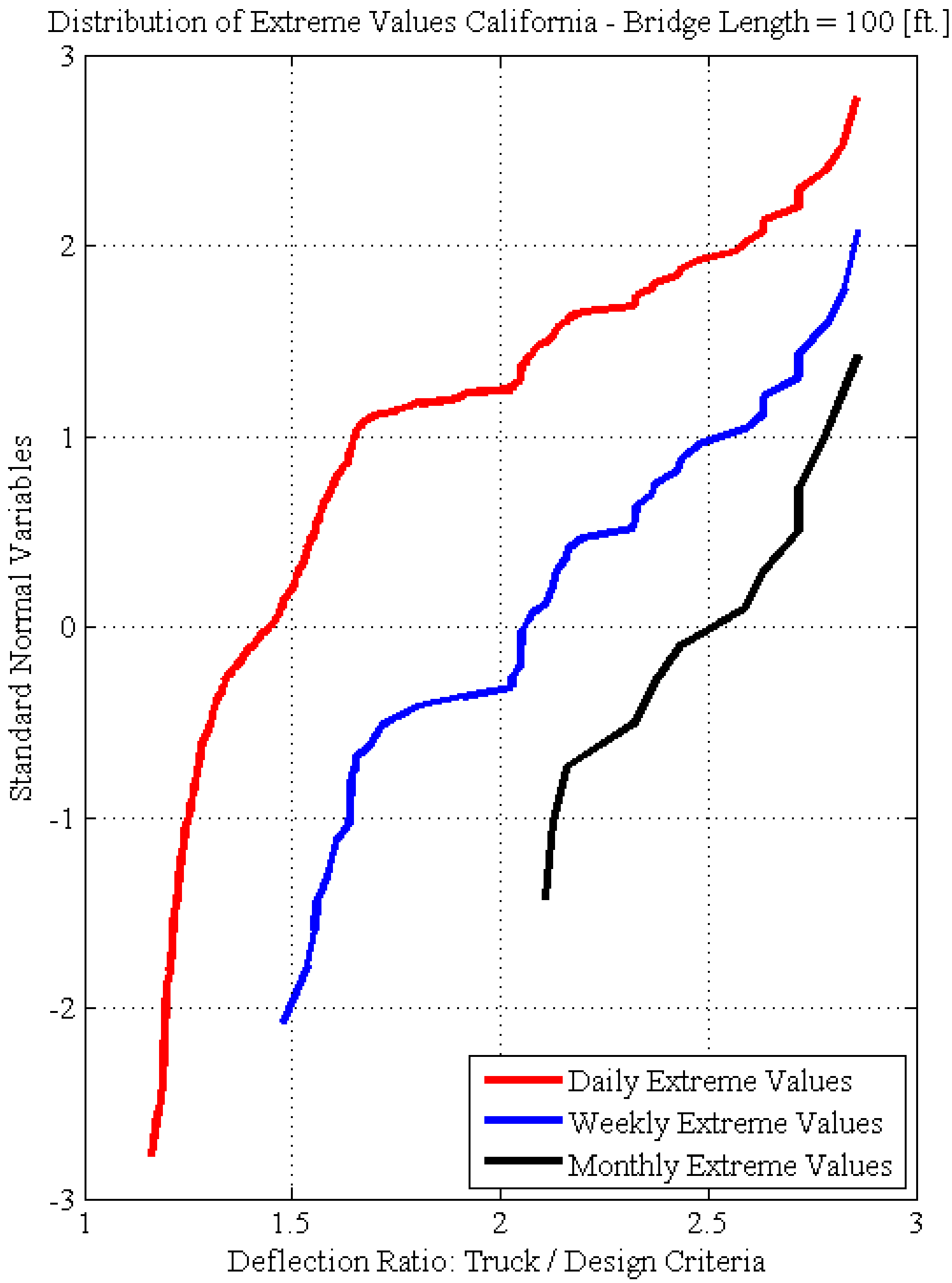

Figure C7. CDF of Deflection Ratio of WIM Deflection to the Design Truck Deflection, California, Span Length =100 [ft.]

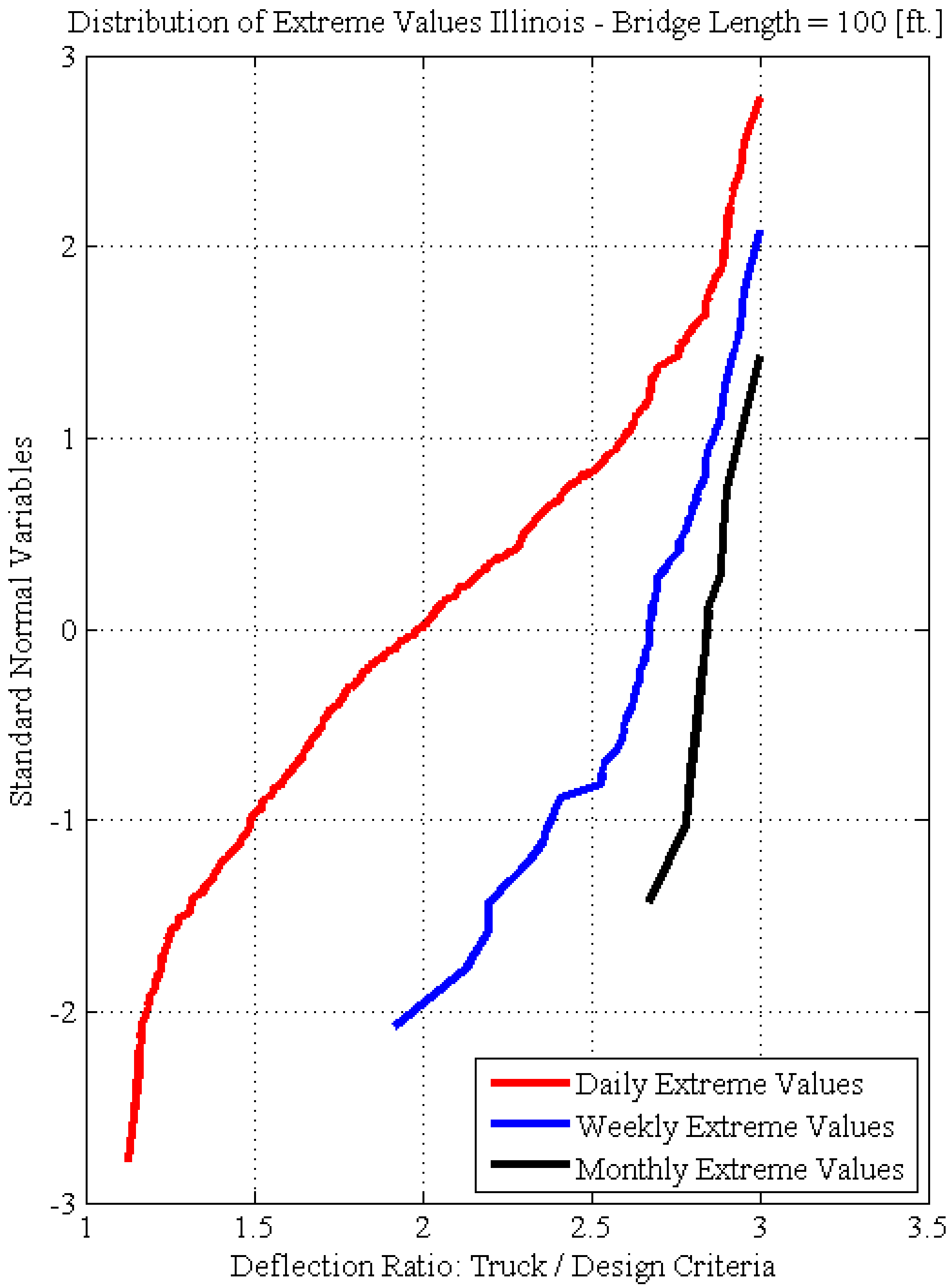

Figure C8. CDF of Deflection Ratio of WIM Deflection to the Design Truck Deflection, Illinois, Span Length =100 [ft.]

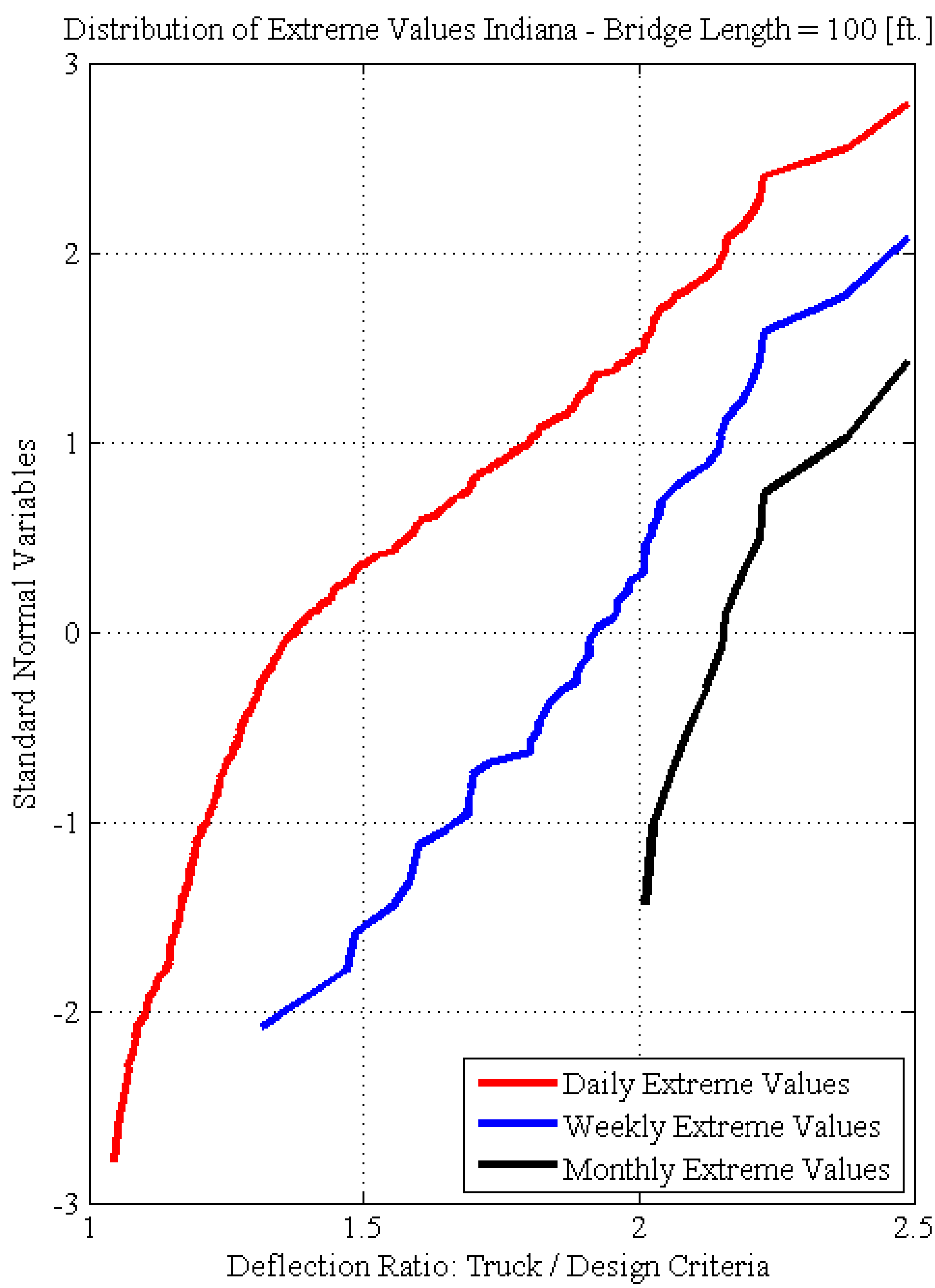

Figure C9. CDF of Deflection Ratio of WIM Deflection to the Design Truck Deflection, Indiana, Span Length =100 [ft.]

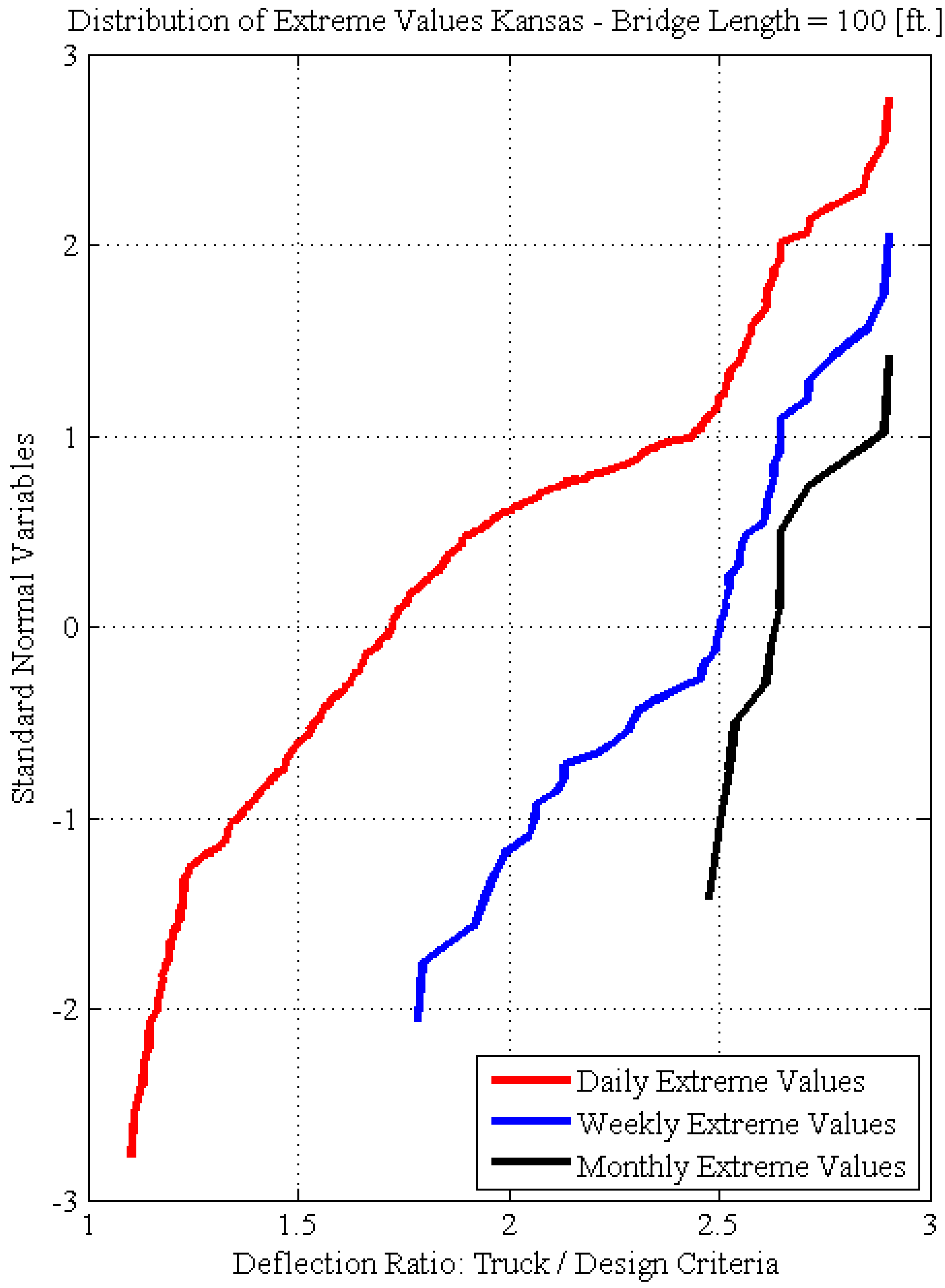

Figure C10. CDF of Deflection Ratio of WIM Deflection to the Design Truck Deflection, Kansas, Span Length =100 [ft.]

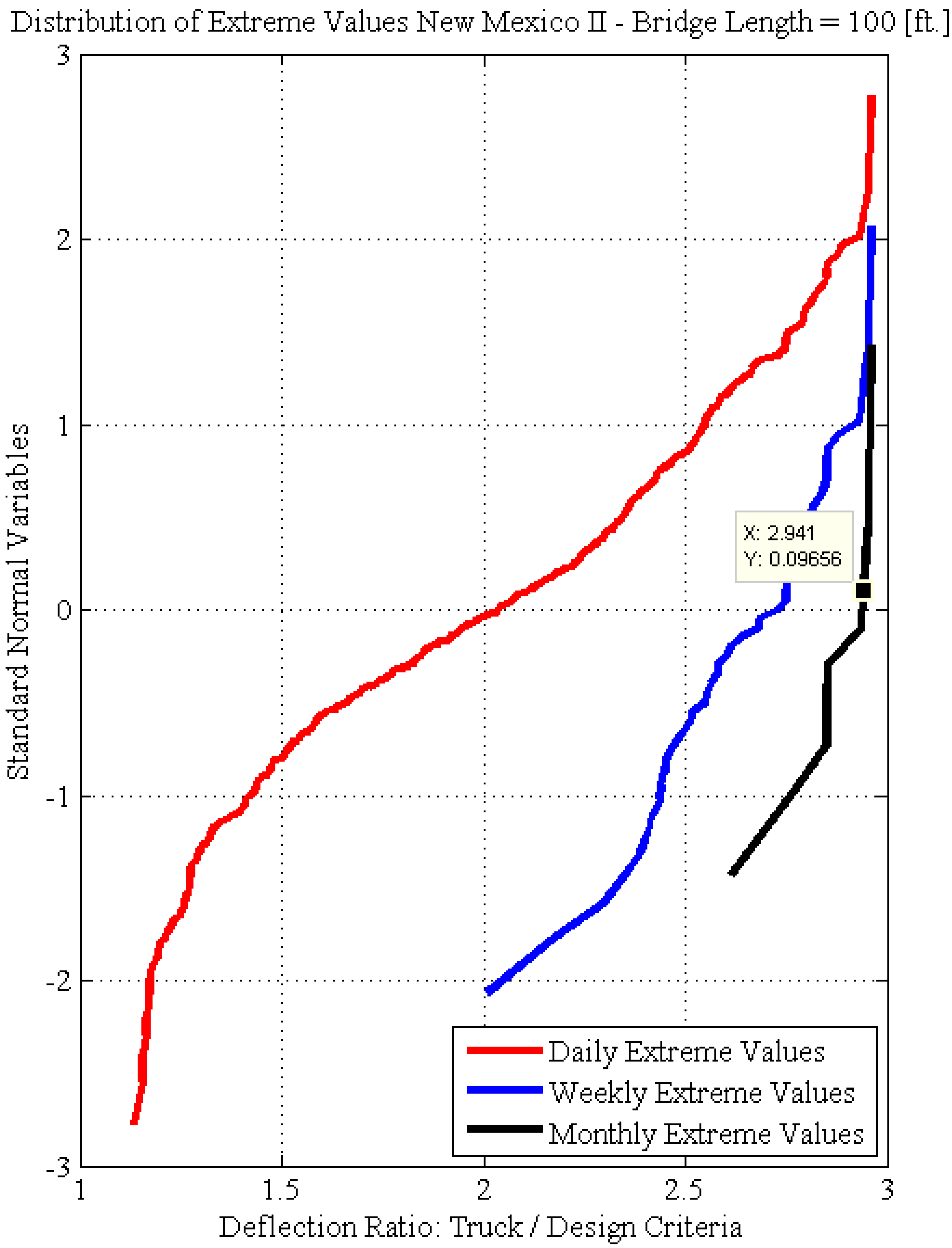

Figure C11. CDF of Deflection Ratio of WIM Deflection to the Design Truck Deflection, New Mexico II, Span Length =100 [ft.]

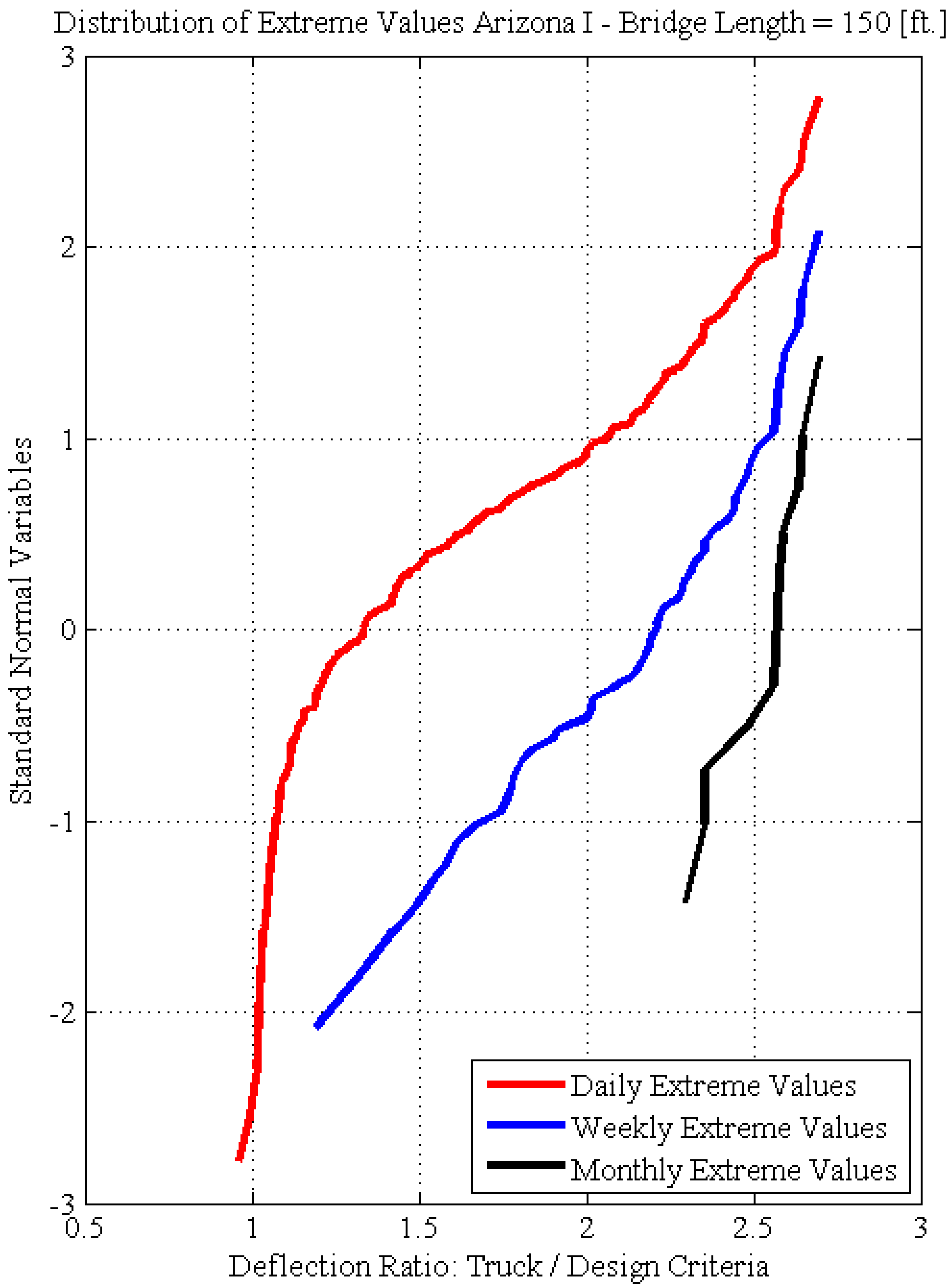

Figure C12. CDF of Deflection Ratio of WIM Deflection to the Design Truck Deflection, Arizona I, Span Length =150 [ft.]

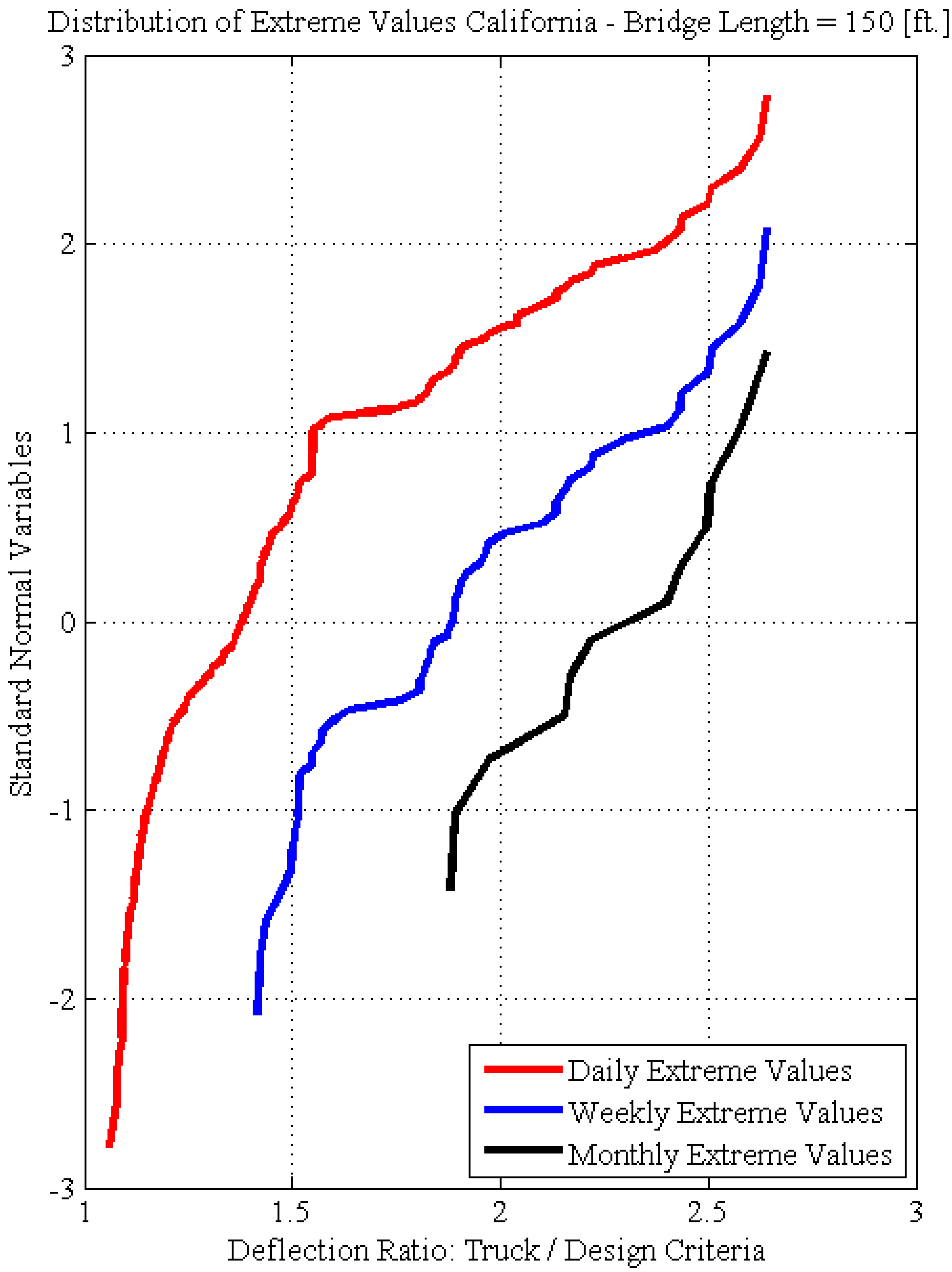

Figure C13. CDF of Deflection Ratio of WIM Deflection to the Design Truck Deflection, California, Span Length =150 [ft.]

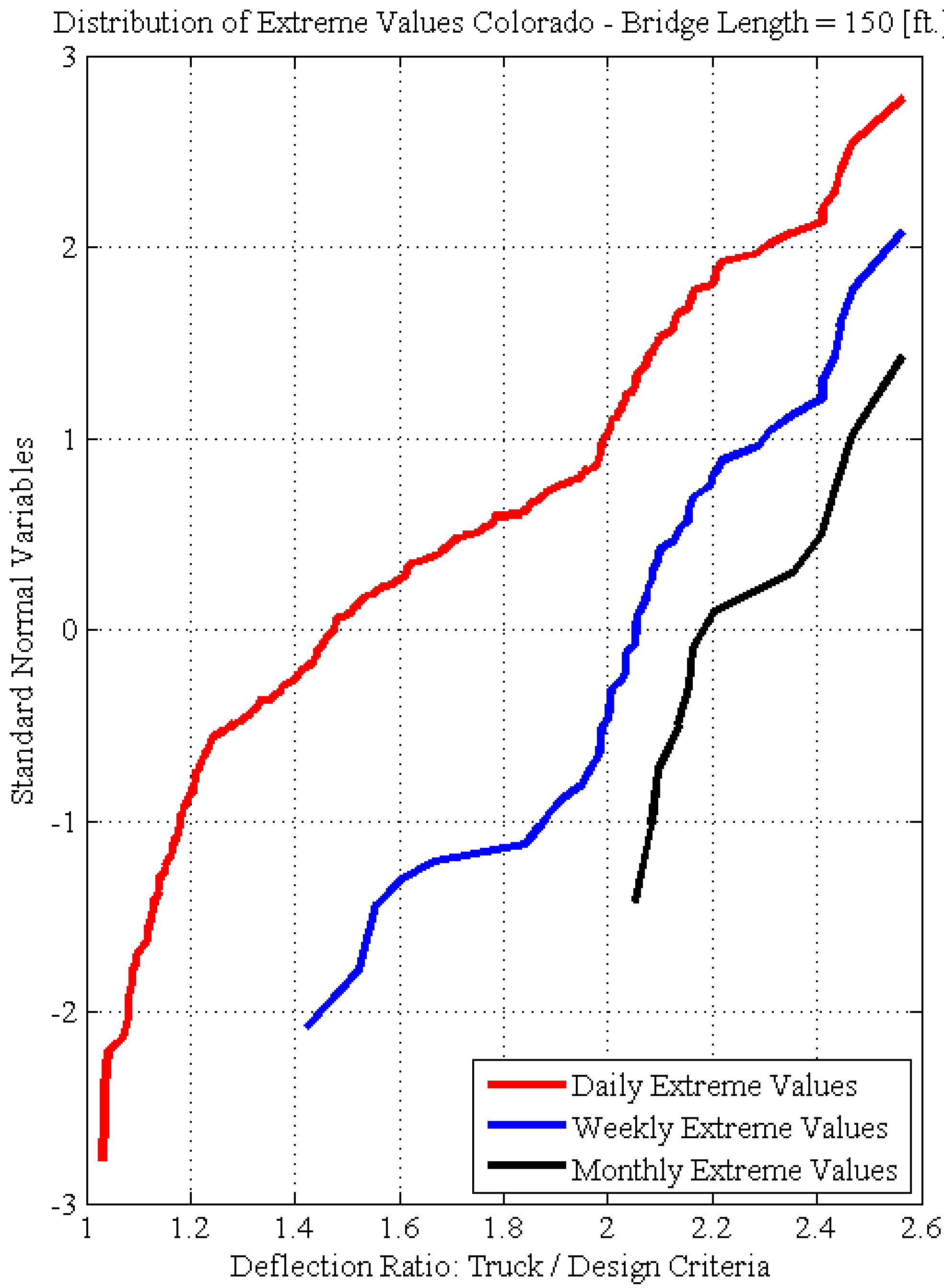

Figure C14. CDF of Deflection Ratio of WIM Deflection to the Design Truck Deflection, Colorado, Span Length =150 [ft.]

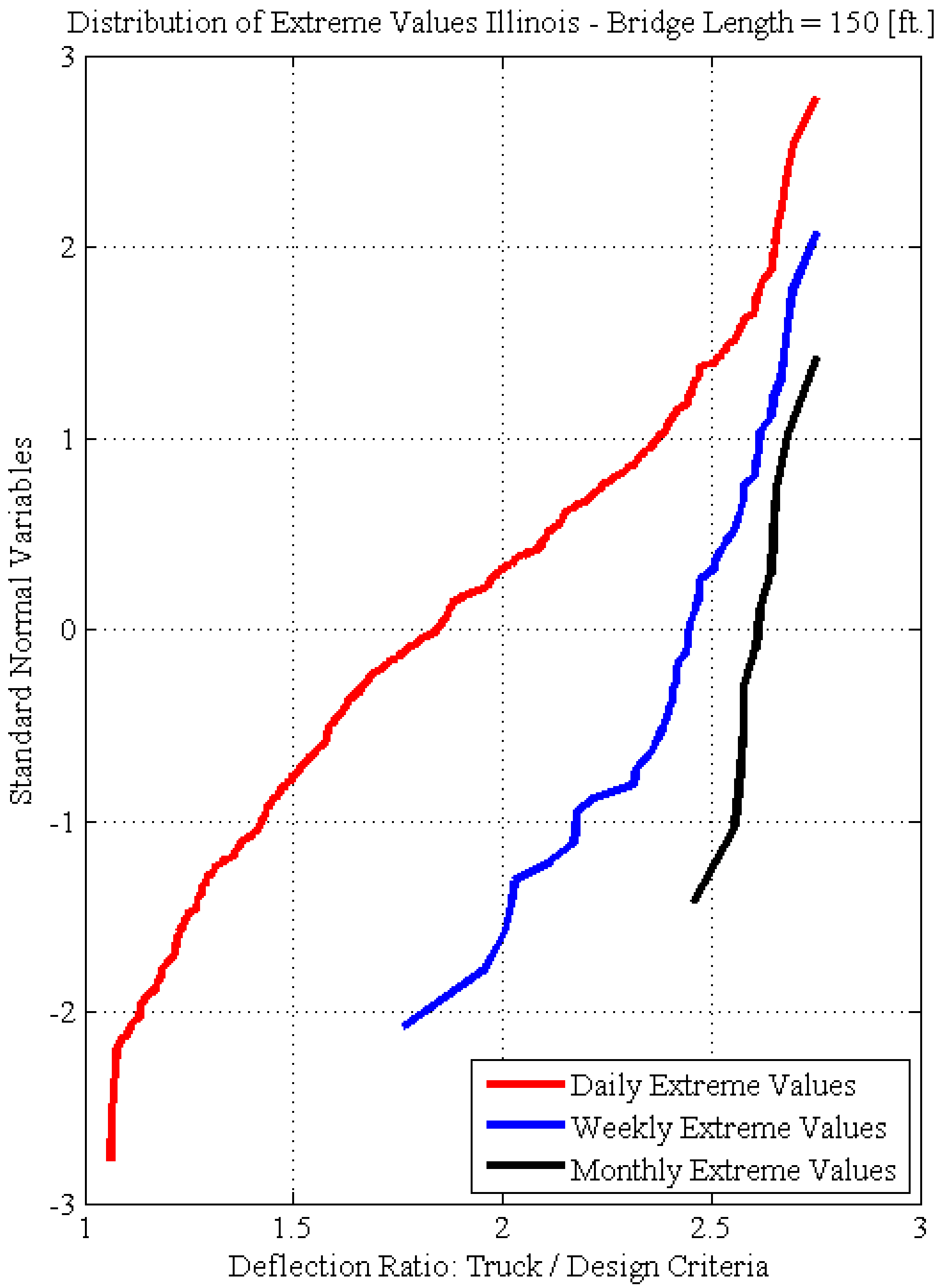

Figure C15. CDF of Deflection Ratio of WIM Deflection to the Design Truck Deflection, Illinois, Span Length =150 [ft.]

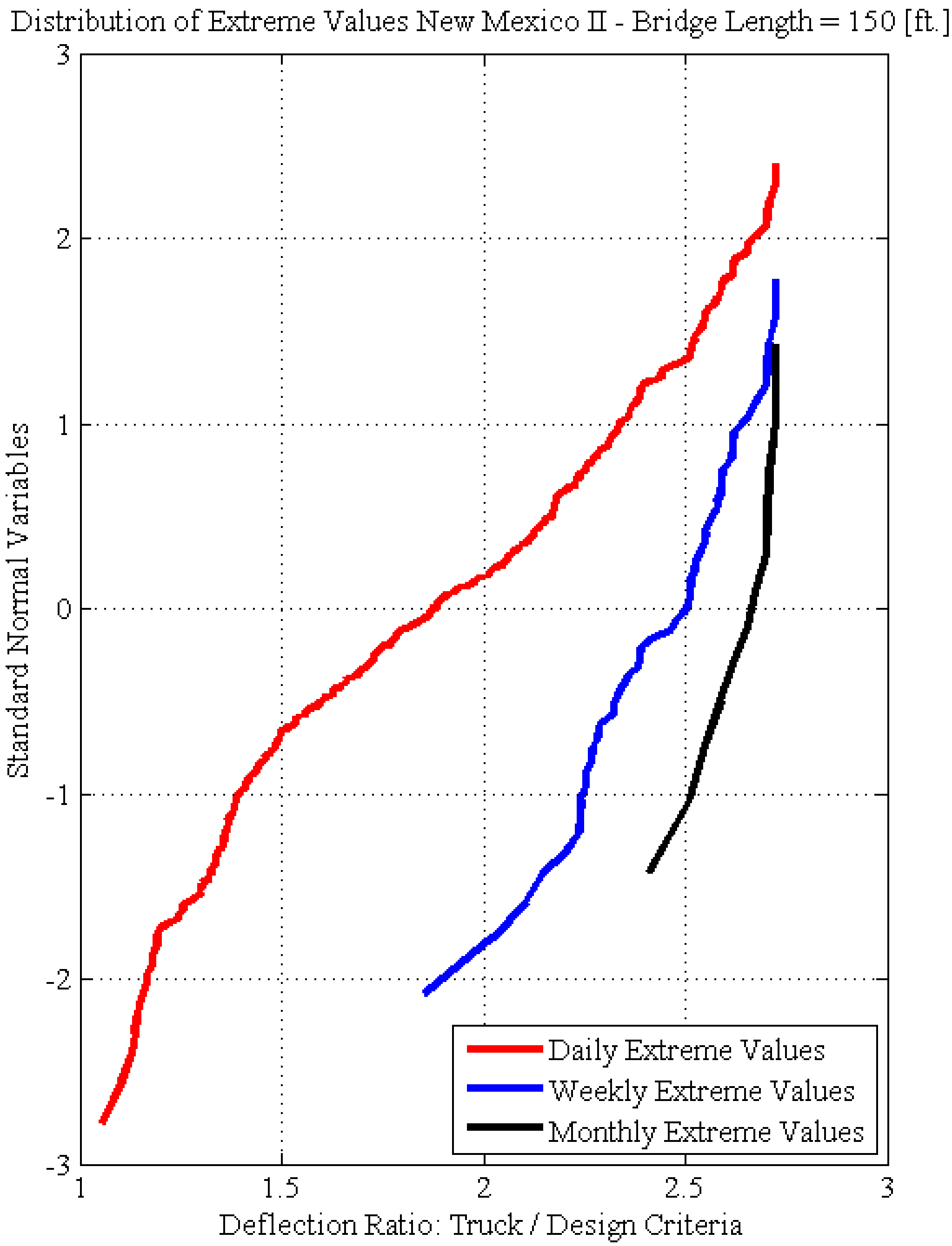

Figure C16. CDF of Deflection Ratio of WIM Deflection to the Design Truck Deflection, New Mexico II, Span Length =150 [ft.]