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Bridge Rating Based on In-Situ Weigh-In-Motion and Health Monitoring Data

Dana Feng

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BRIDGE RATING BASED ON IN-SITU WEIGH-IN-MOTION AND HEALTH MONITORING DATA

A Dissertation

Submitted to the Graduate Faculty of Louisiana State University and Agricultural and Mechanical College in partial fulfillment of the requirements for the degree of Doctor of Philosophy

in

The Department of Civil & Environmental Engineering

by

Dana Feng
B.S., Harbin Institute of Technology, 1988
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December 2016
To my parents
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DEFINITION AND NOTATIONS

Inventory level rating - Load ratings based on the inventory level allow comparisons with the capacity for new structures and, therefore, result in a live load, which can safely utilized an existing structure for an indefinite period of time.

Lever Rule - The statistical summation of moments about one point to calculate the reaction at a second point.

Load Rating - The determination of the live-load carrying capacity of an existing bridge.

Margin of safety - Defined as R-S, where S is the maximum loading and R is the corresponding resistance.

Operating level rating – The maximum load level to which a structure may be subjected. Generally corresponds to the rating at the operating level of reliability in past load rating practice.

Posting – Signing a bridge for load restriction.

Target reliability – A desired level of reliability (safety) in a proposed evaluation.

F or G = cumulative distribution function (CDF)

f = probability density function (PDF)

F^{-1} = quantile function pertaining to the CDF, F

Φ( ) = Standard Normal distribution function

V or COV = variance of a random variable x

μ = mean

σ = standard deviation

λ_b = bias factor: the ratio of mean to nominal value of a random variable

S = total load effect

R = resistance or capacity

P_r = probability

β = reliability index

BM = Block Maxima method
GVW  = gross vehicle weight
POT  = peak over threshold method
GEV  = generalized extreme value
DL   = dead load
LL   = vehicular live load
DW   = dead load of wearing surface and utilities
IM   = vehicular dynamic load allowance
ε    = strain
E    = modulus of elasticity (ksi)
EI   = flexural stiffness (kip-in²)
g    = distribution factor
S    = section modulus (in³); spacing of beams (ft)
L    = span length of beam (ft)
K₉g  = longitudinal stiffness parameter (in⁴)
tₚ   = depth of concrete slab (in)
RF   = rating factor
WIM  = weigh-in-motion
SHM  = structural health monitoring
ABSTRACT

Parallel to the bridge design methodology changes from the Allowable Stress Design to the Load Factor Design, and then to the reliability based Load and Resistance Factor Design (LRFD), bridge load rating method has also been evolving. Applying the reliability theory to the bridge load rating is more complex than applying to the LRFD since any conservatism can have a significant effect on the assessment of bridge capacity, particularly in load posting and bridge replacement. Although the current Load and Resistant Factor Rating (LRFR) method applying the concept of reliability analyses, it uses very limited site-specific data due to practical constraints and the limited availability of site-specific data.

The objective of this study is to develop a reliability based rating approach, grounded in in-situ responses from long-term structural health monitoring systems and actual unbiased traffic data from weigh-in-motion stations. Rating bridges that use actual bridge in-service measurements and site-specific traffic can remove conservatism and uncertainties in association with load distribution factors, dynamic impact, and secondary and non-structural element effects. The end goal is to achieve a continuous bridge evaluation model for real-time vehicle loads, which in turn can be used for speedy truck permitting, bridge management, and identifying sudden condition changes to ensure public safety.

The bridge site-specific truck data and bridge peak strains under ambient traffic for the instrumented bridge have been continuously collected for over a year. The time dependent values of the maximum live load effects are obtained from the statistical analysis of the in-service responses and traffic data. The site-specific live load distribution factors are developed and live load factors are re-calibrated based on reliability analysis. Statistical distribution and projection methods have been compared and validated. This study suggests that the Gumbel distribution and the Parent Tail projection method will be the most suitable methods for the live load distribution and maximum live load effect projection. The reliability-based in-service traffic rating result is compared to three other rating methods: the simplified distribution method, the finite element method, and the live load testing method. The load rating results based on the updated load and load distribution have improved tremendously compared with other rating methods. This systematic rating approach can provide essential information for future bridge maintenance and replacement prioritization. Additionally, a more accurate posting sign is recommended for future bridge load limits.
CHAPTER 1  INTRODUCTION

1.1 BACKGROUND

After the collapse of the U.S. Highway 35 Silver Bridge on December 15, 1967, the United States Congress established the National Bridge Inspection Standard (NBIS) in 1971 to ensure public safety while traveling. The Federal Highway Administration (FHWA) requires each state to use these published standards to develop their own program following the National Bridge Inspection Program (NBIP) and to report National Bridge Inventory (NBI) bridge data annually for bridge assessment. The NBIS regulations apply to all highway bridges located on all public roads.

Bridge condition assessment and bridge load rating are the principal components of the FHWA NBIS. The NBIS defines load rating as “the determination of the live load carrying capacity of a bridge using bridge plans and supplemented by information gathered from a field inspection.” The objective of load rating is to accurately evaluate bridge capacity in order to ensure the safety of the traveling public. Accurate bridge load rating is also an important factor in bridge rehabilitation/replacement, load posting, and overload truck permitting.

Bridge design specifications generally include three design methodologies. In 1931, the American Association of State Highway and Transportation Officials (AASHTO) specification adopted the Allowable Stress Design (ASD) method, in which the uncertainties in loads and resistance were lumped into a single factor of safety. In the 1970s, the specifications evolved to the Standard Specifications for Highway Bridges incorporating the Load Factor Design methodology (LFD). The LFD method introduced different load factors to reflect the relative uncertainty and predictability of different loads. Based on the growing knowledge and understanding of bridge performance, the Transportation Research Board (TRB) concluded that the Standard Specifications for Highway Bridges included gaps and inconsistencies, so the AASHTO Bridge Subcommittee voted to stop maintaining the LFD Standard Specifications for Highway Bridges in 1999. The current AASHTO LRFD (Load and Resistance Factor Design) Bridge Design Specifications were first published in 1994 with the intention of eventually replacing the AASHTO Standard Specifications for Highway Bridges. The LRFD specifications integrate the reliability theory, which takes statistical variations of both loads and resistances into the consideration for the design procedure. Bridges designed using these new specifications should have more uniform levels of safety, which can lead to consistent serviceability and long-term maintainability of bridges.

Parallel to the bridge design methodology changes, bridge load rating methodology has also been evolved from the Allowable Stress Rating (ASR) to the Load Factor Rating (LFR), and then to the Load and Resistant Factor Rating (LRFR) method. Similar to ASD, the ASR method published in the AASHTO Manual for Maintenance Inspection of Bridges in the 1970s is also deterministic. The ASR method uses a fraction of the load carrying capacity of a structural element as the allowable limit. The shortcoming of this method is the use of a single safety factor to cover all uncertainties without considering the risk level of each individual contributing factor; as a result, there may be aspects of the rating that are inadequate. To overcome this shortcoming, corresponding to the LFD, LFR was incorporated into the AASHTO Manual for Condition
Evaluation of Bridges in 1994. LFR is still not a “risk” based analysis since it lacks the consideration of uncertainties or variability of the structural resistances. Following the AASHTO LFRD, the corresponding reliability based LRFR rating was first published in the AASHTO Manual for the Condition Evaluation and Resistance Factor Rating of Highway Bridges in 2003 (MCE LRFR). The MCE LRFR replaced the 1994 edition of the AASHTO Manual for Condition Evaluation of Bridges (MCEB), which covers the LFR and the ASR methods. The MCE LRFR was later changed to The Manual for Bridge Evaluation (MBE) in 2008. The intent of the LRFR is to provide a reliability-based methodology that is consistent with the LRFD Specifications and to extend the provision of the LRFD Specifications to the areas of inspection, load rating, posting, and permitting rules to existing bridges.

In the structure reliability methodology, safety is notionally measured in terms of the reliability index (safety index), beta (β). When designing a bridge, a conservative reliability index is used to ensure serviceability and durability without incurring a major cost impact. Overly conservative assumptions in load ratings, however, can be costly and may lead to unnecessary load restrictions, rehabilitations, and even replacement of bridges.

The reliability-based LRFD specifications provide load and resistance factors that should lead to more consistent target reliability levels for the design of components over a wide range of bridge spans and material applications (Moses, 2001). Therefore, the LRFD specification has been calibrated to provide a uniform and acceptable level of safety for all bridges within the 75-year designed lifespan. Bridge evaluation, on the other hand, focuses on the safety of individual bridges at the time of evaluation and five years onward under certain conditions. The application of reliability theory to bridge load rating is much more complex due to the significant difference among bridge types and the time-dependent variations in traffic conditions and even structure resistance. Among those statistical variances, live load would have the highest level of uncertainties.

The AASHTO LRFD specification design live load model, HL-93, was developed using the database adopted from a truck survey conducted in the 1970s in Ontario, Canada by the Ontario Ministry of Transportation (Nowak, 1999). The Ontario study was based on weighing approximately 9,250 selected trucks from a single site for only a two-week period. The data was then used to project a 75-year live load occurrence. The maximum values were determined by the use of the extreme value theory (Castillo et al. 1998, Gumbel 1941, 1954, 1958). It has been recognized that a considerable degree of uncertainty exists in projections to long-term traffic load due to the limited duration of the study and size of the Ontario database. In fact, the same site was later used to replicate the original Ontario truck weight data acquisition, and the observations showed an increase in heavy truck load effects (Ontario General Report 1997). Similarly, while implementing the MBE in 2009, the Louisiana Department of Transportation and Development (LADOTD) convened a study comprising of Louisiana legal trucks, AASHTO legal trucks, and design truck HL-93. The results indicated that LA legal truck loads are heavier than the AASHTO legal loads included in the MBE. It was also indicated in this study that the HL-93 truck does not envelope the Louisiana routine permit trucks or trucks observed from the LA weigh-in-motion (WIM) stations. Therefore, the MBE recommended rating methods might be insufficient for all Louisiana load ratings. A more accurate rating method is required for future bridge evaluation, bridge posting, and truck permitting.
Moreover, the bridge reliability index generally decreases with time due to deterioration, accidental damage, fatigue, and traffic growth. Neglecting these factors can result in unrealistic structural reliability estimations when rating the bridge. Improvement can be made when rating a specific bridge by including in-situ traffic data, performance data, and material and geometry data. The improved accuracy in the evaluation of the bridge reliability index based on the site-specific data may also improve bridge load posting evaluation and postpone the need of rehabilitation. Most importantly, it can identify problem bridges so that proper action can be taken to ensure public safety.

1.2 RESEARCH OBJECTIVE AND SCOPE

Accurate load rating is the determination of the live-load carrying capacity of an existing bridge based on the current structural conditions, material properties, loads, and traffic conditions at the bridge site. The objective of this study is to establish an accurate in-service response-based bridge load rating through a reliability analysis by using on-site specific information at the time of evaluation to remove some uncertainties inherent in bridge design regarding traffic, construction, and analysis.

Currently, the load rating engineer has the choice of three levels of load rating depending upon complexity and accuracy. The first and most commonly selected level is the approximate method of live load distribution analysis as described in the AASHTO LRFD. The live load distribution formulas were developed for common bridge types and dimensions and for the HS family of trucks (MBE, 2012).

When those conditions are not representative of a specific bridge, a refined level II analysis method should be performed. Refined analysis uses a proper finite element model (FEM) to present the relative stiffness of all bridge components. In this case, the uncertainties associated with the load distribution assumption and the simplified structural details from the level I analysis can be reduced. To further reduce the uncertainties, the next level of evaluation is the non-destructive load testing (NDT) method. NDT can provide actual response of an existing structure accounting for design details, construction deviation, deterioration, damage, repair, and current environmental and operational conditions (NSF, 1992). The baseline finite element model can be validated using live load testing results for level III analysis.

The above load rating methods are all based on the pre-defined live loads, which were developed from the limited traffic data as we described previously. The code-specified design loads or legal loads may not reflect the traffic condition at the bridge site. To reduce the uncertainties and to alleviate some conservative or unsafe assumptions in the design specifications, the strain-based long-term structural health monitoring system (SHM) and the site-specific traffic data collection system, weigh-in-motion (WIM) station, are incorporated in the present study. The in-service measurement system will provide actual site-specific load and bridge behavior under ambient traffic. With the measured load-response and the truck loads, the bridge can be rated with reduced conservatism and less uncertainties from live load distribution, dynamic impact, and secondary and non-structural element effects. Through a statistical reliability assessment, a more accurate rating should result. Since this level of analysis exceeds
the three levels of analysis defined in the MBE, it is termed “level IV” analysis in this dissertation.

For this analysis method, the available traffic and strain data are always limited to the duration of the measurement. Thus, a statistical prediction tool is necessary to properly evaluate these parameters. A site-specific live load distribution factor and the live load factors can also be developed for future rating and permitting.

All four-level load rating methods were performed, and the rating results are summarized for comparison in the study. An improved new posting sign recommendation is also included. To accomplish the research objective, the following research tasks will be conducted:

1. Load rate the selected bridge with the live load distribution method (Level I) and the finite element method (Level II) in order to establish a baseline model. Use this model to develop the SHM and WIM instrumentation plans.
2. Perform NDT field load testing to calibrate the bridge model and to rate the bridge (Level III).
3. Monitor and collect SHM and WIM data under ambient traffic over a year.
4. Perform WIM truck statistical analysis and projection to develop the site-specific trucks. Use these trucks to rate the bridge (Level IV).
5. Perform SHM data statistical analysis to develop a site-specific girder live load strain statistical model. Develop site-specific live load distribution factors. Calibrate the live load factors to meet a target reliability index. Rate the bridge based on the measured strain data and the reliability index (Level IV).
6. Compare WIM and SHM and recommend future posting sign.

This study uses a systematic approach to including all bridge related data into bridge evaluation. The following topics are unique to this study and differ from traditional rating methods:

1. This research utilizes the site-specific ambient traffic instead of the pre-defined design trucks, legal trucks, and permit trucks.
2. This research directly uses the measurement of structure element in-service responses to represent the actual bridge response under ambient live loads in order to reduce the uncertainties in modeling, dynamic factors, and live load distribution factors.
3. This research makes use of a reliability analysis to derive the site-specific LRFR live load factors, which is consistent with the AASHTO LRFD methodology.
4. Different projection methods have been investigated, and one method is recommended for maximum live load and maximum strain projection.

1.3 OVERVIEW OF THE DISSERTATION

This dissertation is organized into eight chapters.
Chapter 1 explores the motivations and objectives of the study. Chapter 2 describes the four-level bridge load rating methodologies, the concept of the reliability theory, and the study scope to develop an in-situ reliability-based load rating method. Chapter 3 provides the background details of a case study to illustrate the different levels of bridge load rating.

Chapter 4 demonstrates the level I and level II load ratings for the selected bridge: the live load distribution factor method and the finite element method. The construction concrete strengths are incorporated into the as-built load rating. Chapter 5 presents the diagnostic nondestructive live load testing method and the load rating using the calibrated bridge model developed based on the test results.

Chapter 6 discusses how to use the in-situ WIM data to develop and calibrate the live-load models for load rating. The truck traffic data, including truck weights and configurations with a timestamp, has been collected through a WIM system for over a year. The unbiased data is used to improve the live load statistical model, such as the live load multiple presence factor, the seasonal variance factor, the live load statistical distribution models, and the maximum load projection method. The bridge is evaluated using the improved site-specific live load models.

Chapter 7 describes the in-service SHM data reliability analysis method. This approach utilizes the measured bridge strain data to derive load rating factors through statistical distribution, projection, and reliability analysis. The site-specific live load distribution factor and the calibrated live load factors are also developed for future bridge rating.

Chapter 8 provides the comparisons of different rating methods, conclusions, and suggestions for future bridge load rating, posting, and research needs.
CHAPTER 2   LITERATURE REVIEW AND RESEARCH FRAMEWORK

2.1 OVERVIEW OF LOAD RATING HISTORY

AASHTO approved bridge rating methodologies have evolved from the Allowable Stress Rating method (ASR), to the Load Factor Rating method (LFR), and finally to the Load and Resistance Factor Rating (LRFR) method. The LRFR method is a methodology for load rating a bridge consistent with the LRFD philosophy, which is based on the reliability theory.

Nowak and Lind (1979) proposed the incorporation of the reliability theory into a bridge design code. Nowak and Hong (1991) performed a statistical analysis to develop the live load model using the Ontario truck data. The results were published in the National Cooperative Highway Research Program (NCHRP) Report 368 (Nowak 1999). The AASHTO adapted this method into the \textit{AASHTO LRFD Bridge Design Specifications} in 1994. Ghosn and Moses (1986, 1998) performed a calibration for the extreme events design of highway bridges applying the uniform reliability methodologies. Many other researchers also contributed to the understanding of the dynamic load effect and multiple presence effect to the live load statistical models for the LRFD (Heywood and Nowak 1989, Bakht and Jaeger 1990, Zokai et al. 1991, Schwarz and Laman 2001, Nowak 2004, Gindy and Nassif 2006, Kulicki et al. 2007).

The LRFD method introduced the notional design live load model HL-93 and the corresponding calibrated live load factors to achieve the bridge design safety. To extend the reliability methodology to load rating, Moses (2001) derived legal trucks and the associated load factors for the LRFR method in the MBE. The MBE includes options to allow for the incorporation of site-specific traffic, performance data, and target safety criteria when warranted by the evaluation needs. Weigh-in-motion was suggested by researchers for bridge evaluation (Moses and Ghosn 1983, Moses and Snyder 1985, Lee and Souny-Slitine 1998). NCHRP Project 12-76 (Sivakumar et al. 2008) presented the protocol for collecting and using WIM data in bridge design and rating. Based on the selected WIM sites, Sivakumar and Ghosn (2011) recalibrated the LRFR live load factors and developed special hauling vehicles. The representative statistical information on truck weights, truck configurations, and multiple presence data improved the understanding of uncertainties and made the live load factors more representative. Mlynarski et al. (2011) refined the LRFR live load factors through the comparisons of 1,500 sample bridge ratings.

The reliability index, $\beta$, a measure of data dispersion or reliability, is used in both the LRFD and the LRFR to manage the risk level. While developing the code, load factors were calibrated to have consistent target reliability levels for components over a wide range of bridge spans and material applications.

The specifications commonly include four limit states: strength limit, serviceability limit, extreme event limit, and fatigue limit. The calibration of the Strength I limit state was aimed to achieve a target reliability index of $\beta_{\text{target}}=3.5$ (Nowak 1999). Consequently, a reliability index of $\beta_{\text{target}}=3.5$ was used for the calibration of the design load inventory rating (first-level), and $\beta_{\text{target}}=2.5$ was used for the calibration of the operating rating (second-level) in the AASHTO
LRFR by Moses (2001). A target index of $\beta_{\text{target}}=2.5$ was used for the legal load rating and routine permit load rating. Recent observations made on truck weight data collected from WIMs from representative U.S. sites (Sivakumar et al. 2008) have shown that trucks traveling over US highways are significantly different from the trucks from the Ontario study that the current AASHTO LRFD was based on. A live load recalibration in May 2011 (Sivakumar et al. 2011) was performed based on U.S. WIM sites. The target reliability index selected for the special hauling vehicles and permit load recalibration was $\beta_{\text{target}}=2.5$ with a goal of achieving a minimum reliability index value for all conditions of 1.5. A site-specific probability assessment idea was also proposed (O’Brien et al. 2003).

More serviceability calibration efforts were made in later years (Orcesi and Frangopol 2010). NCHRP project 12-83 calibrated the LRFD serviceability limits for concrete bridges. SHRP2 project R19B provided the calibration for the service limit states (SLS), specifically for bridges beyond 100 years (Wassef et al. 2014). These calibrations include foundation deformation, cracking for reinforced concrete components, live load deflection, permanent deformation, cracking of pre-stressed concrete components, and fatigue of steel and reinforced concrete components. Since the consequences of exceeding SLSs are way less severe than those associated with ultimate limit states, most of the SLS calibrations were generally done with a target reliability index of $\beta_{\text{target}}=1.0$ to 1.5 based on the probability of a one year return period. The result suggests increasing the live load factor from 0.8 to 1.0 for tension limit state, and increase live load factor from 1.5 to 2.0 for the fatigue limit state for certain types of bridge.

The LRFD reliability analysis methods were developed and applied mostly to individual structural components, rather than structural systems. The system reliability was used to verify the selection of redundancy factors (Nowak, 1999). The load carrying capacity of the whole structure is often much larger than what is determined by the reliability of components only. The ratio of $\beta_{\text{system}}/\beta_{\text{girder}}$ varies from two to six, as compared to $\beta_{\text{girder}}$ (Nowak, 2004). To be consistent with the current specifications, the present research concentrates on the component reliability only.

The notional measurement of safety, $\beta$, was selected to be lower for operating than for design due to economic and serviceability considerations. Though the operating rating was calibrated to have a smaller safety margin, we may remove some of the conservative assumptions for bridge rating by using less biased parameters, such as in-situ data and performance experience. Consequently, through a reliability analysis, the $\beta$ value is likely to be higher than indicated and the load rating will be more accurate.

There are a couple of methods that can be used to reduce statistical uncertainties, such as structural health monitoring, non-destructive live load testing, and WIM systems. SHM is used for in-service structures to collect response data to represent structure response over time. Load testing may also be performed as part of the SHM. Diagnostic load testing has been widely used for bridge evaluation and load rating in addition to visual inspection and analysis (Fu and Tang 1992; Moses et al. 1984; Chajes et al. 2000). Instead of using the test data sole for improving the finite element modeling, this proposed study will utilize the bridge response data from SHM and the live load data from WIMs to establish a reliability-based in-service bridge evaluation system.
2.2 INTRODUCTION TO RELIABILITY ASSESSMENT

The designation of a reliability-based design format usually refers to procedures in which specification bodies consider the statistical distributions of loadings (e.g., dead, live, and environmental loads) and the statistical distribution of component strength (e.g., members, connections, and substructures) (Moses 2001). The objective of the structural reliability-based rating theory is to account for the uncertainties encountered while evaluating the safety of structural systems. The uncertainties could include material properties, geometries, fabrication procedures, load models, numerical models, etc. Many researchers have studied the bridge reliability analysis topic (Nowak and Collins, 2000; Ang and Tang, 1984; and Ayyub and McCuen, 1997).

The reliability-based design method includes the considerations of the statistical distribution of load effects, \( S \), and resistance, \( R \), as random variables (RV) rather than predetermined constants of the ASD. The random variables reflect the uncertainty of their values at the time that the individual component is checked. To reflect the uncertainties, the random variables are described by probability distribution functions. That is, a random variable may take a specific value with a certain probability, and the ensemble of these values and their probabilities are described by a probability distribution function (PDF).

The cumulative distribution function (CDF) (denoted by \( F_X(x) \)) of a random variable, \( X \), is defined as the sum of all probability functions corresponding to the random variables having values less than or equal to \( x \). The first derivative of \( F_X(x) \) is called the probability distribution function (PDF), denoted by \( f_X(x) \). The first two moments of random variables are the mean, \( \mu_X \), and the standard deviation, \( \sigma_X \). The coefficient of variation, \( \text{COV}_X \) or \( V_X \), is the standard deviation normalized against the mean. The Bias factor \( (\lambda) \) is the ratio of mean to nominal \( (X_n) \).

For a sequence of independent random variables, they are defined as:

\[
\mu_X = \int_{-\infty}^{+\infty} x f_X(x) \, dx
\]  

(2.1)

\[
\sigma_X^2 = \int_{-\infty}^{+\infty} (x - \mu_X)^2 f_X(x) \, dx
\]  

(2.2)

\[
V_X = \frac{\sigma_X}{\mu_X}
\]  

(2.3)

\[
\lambda_X = \frac{\mu_X}{X_n}
\]  

(2.4)

A structure fails when it can no longer perform its intended function (Nowak and Zhou 1990). To define failure in the context of structural reliability, the concept of limit states is used. A limit state should express the margin of safety for any type of failure mode in a deterministic fashion, including: strength limit states, serviceability limit state, fatigue and fracture limit states, extreme limit state, etc. The extreme and fatigue limit states are not considered here for bridge rating purposes.
When the resistance and the load effect can be modelled as random variables that are independent of each other, the limit state function or safety margin can be defined as:

\[ G(R,S) = \text{Resistance} - \text{load effect} = R - S \]  

(2.5)

In a structural analysis, safety may be described as the situation where the capacity, \( R \), exceeds demand, \( S \). Probability of failure, i.e., the probability that the capacity is less than the applied load effects, may be formally calculated; however, its accuracy depends upon the probability distributions of load variables and resistance variables. The probability of failure, \( P_f \), may be expressed by integrating over the load frequency distribution curve as follows:

\[ P_f = P[R < S] = \int P[R < S]f_S ds \]  

(2.6)

Thus, integrating Eq. (2.6) or summing numerically over each value of load finds the probability of failure. The density function of load times the probability of failure decreases if there is less overlap of the load and the resistance frequency curves, as shown in Figure 2-1.

Figure 2-1 Basic reliability model and failure probability

In general, the value of \( P_f \) increases with decreasing safety factors and increasing coefficients of variation. The safety or reliability index is often denoted as beta (\( \beta \)):

\[ \beta = \frac{\mu_g}{\sigma_g} \]  

(2.7)

If the load and the resistance are normally distributed, any linear combinations are also normally distributed. The reliability index, thus, can be computed as:
\[ \mu_g = \mu_R - \mu_S \quad (2.8) \]
\[ \sigma_g = \sqrt{\sigma_R^2 + \sigma_S^2} \quad (2.9) \]

Therefore, \[ \beta = \frac{\mu_R - \mu_S}{\sqrt{\sigma_R^2 + \sigma_S^2}} \quad (2.10) \]

For normally distributed random variables, the reliability index is related to the probability of failure by:
\[ \beta = \Phi^{-1}(P_f) \quad (2.11) \]

Where, \( \Phi^{-1} \) is the inverse standard Normal distribution function (Cornell 1967).

If all random variables follow Normal distribution, the First Order Second Moment (FOSM) can be exact. Otherwise, the reliability indices for tail end distributions other than normal based on FOSM include a considerable level of error (Nowak and Collins, 2000).

A higher level reliability method may also be needed for the cases when the variables are neither normal nor lognormal. Two frequently used high level reliability methods, referred to as FORM (First Order Reliability Methods) or FOSM (Nowak and Collins, 2000) involve an iterative calculation to obtain the failure probability. The FORM method is based on the derivation, which uses a first order Taylor series and the second moments of the input variables (\( \sigma \) and \( \mu \)). For a linear limit state function of variables, form \( g(X_1, X_2, \ldots, X_n) \) is shown in Equation 2.12.

\[ g(X_1, X_2, \ldots, X_n) = a_0 + a_1X_1 + \cdots + a_nX_n = a_0 + \sum_{i=1}^{n} a_iX_i \quad (2.12) \]

Where, \( a_i \) is a constant and \( X_i \) is an uncorrelated random variable. \( \beta \) could be obtained by the following expression:
\[ \beta = \frac{a_0 + \sum_{i=1}^{n} a_i\mu_i}{\sqrt{\sum_{i=1}^{n} (a_i\sigma_{X_i})^2}} \quad (2.13) \]

More advanced techniques including SORM (Second Order Reliability Methods) may also be needed. The Monte Carlo Simulation (MCS), a computational algorithm based on repeated random sampling and simulations to compute the reliability index, can also be used for the reliability analysis. The MCS method is straightforward but computationally intensive and is frequently used for complex nonlinear state limit functions.
2.3 TIME VARIANT RELIABILITY

In contrast to the design, the reliability of bridge performances or load rating is time-dependent and subject to the influences of traffic, maintenance, and deterioration. It is also subject to analysis modification by additional site data. Low values of calculated reliability due to large uncertainties from assumed properties may lead to costly bridge posting, rehabilitation, or replacement.

As described in Section 2.2, reliability depends on two factors, the structural capacity and the load effects. Both factors are time-variant in nature. In general, the capacities decrease with time, and the load effects increase with time as shown in Figure 2-2.

There are two ways to reduce the probability of failure and to increase the structure reliability. The fundamental approach is to increase the distance between the load and the resistance curves, as shown in Figure 2-3(a), by strengthening the structures or reducing the loads. Another approach is to improve the frequency distribution, as shown in Figure 2-3(b). A sharp peaked frequency curve occurs with a reduction in uncertainty of a variable, while a flatter distribution indicates a greater uncertainty. Using site-specific data, such as construction records, inspection records, traffic data, and bridge performance data, will reduce uncertainties associated with the general assumptions made from other observations, such as the Ontario truck data.

![Figure 2-2 Reliability changing over time](image)

The variance involved in the bridge design includes the load variances and the resistance variances. Among those variables, the factor that is the least reliable is the live load. A discussion of some of the variables and the statistical methods in estimating the parameters for these variables will be included in the next section.

2.4 VARIANCES IN BRIDGE EVALUATION

2.4.1 Load Variances (S)

The load variances include the dead load, live load, and other environmental phenomena (not considered for bridge load rating). For bridge load rating, the live load is the most important variance.
2.4.1.1 Dead Load (DL)

Dead load (DL) is the gravity load due to the self-weight of the structural components and non-structural permanent attachments. The AASHTO LRFD considers the dead load as self-weight (DC), the wearing surface (DW), and utilities (DC4). The LRFD calibration considers the DL as a normal random variable, and the statistical parameters are listed in Table 2-1 for the 30-ft to 200 ft bridge spans (Nowak 1999). For other spans, the moment effects of the dead weights are obtained from estimates of the dead weight per unit length. All of the dead load random variable parameters are taken as Normal distributions.

<table>
<thead>
<tr>
<th>Component</th>
<th>Bias Factor ($\lambda$)</th>
<th>Coefficient of Variation (V)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DC1 – Factory made members</td>
<td>1.03</td>
<td>0.08</td>
</tr>
<tr>
<td>DC2 – Cast-in-place members</td>
<td>1.05</td>
<td>0.10</td>
</tr>
<tr>
<td>DW – Asphalt DW</td>
<td>1.00*</td>
<td>0.25</td>
</tr>
<tr>
<td>DC4 – Miscellaneous</td>
<td>1.03 – 1.05</td>
<td>0.08 – 0.10</td>
</tr>
</tbody>
</table>

* Mean thickness = 3.5 inch

The bias factors for factory-made members and cast-in-place members were provided by the Ontario Ministry of Transportation based on surveys of actual bridges in conjunction with the calibration of the Ontario Highway Bridge Design Code (OHBDC 1979, 1983, 1991, Nowak and Lind 1979). The coefficients of variation used in the LRFD calibration were taken from the NBS
report 577 (Ellingwood et al. 1980), and other uncertainties including human error were also considered.

The dead load variance can be obtained using equations 2.14, 2.15, and 2.16. For bridge rating, the complete bridge material property records, such as material density and measured overlay thickness, can be used to substantiate the dead load variable uncertainty.

\[
\mu_{DL} = \mu_{DC1} + \mu_{DC2} + \mu_{DW} 
\]

\[
\sigma_{DL} = \sqrt{\sigma_{DC1}^2 + \sigma_{DC2}^2 + \sigma_{DW}^2} 
\]

\[
V_{DL} = \frac{\sigma_{DL}}{\mu_{DL}}
\]

2.4.1.2 Live Load (LL)

Vehicular live load (LL) is a transient load that is composed of static truck load and their dynamic effect. The effect of static live load depends on many parameters including the span length, truck weight (axle loads and axle configurations), truck position on the bridge, traffic volume, concurrent vehicles on the bridge, girder spacing, and stiffness of structural members. The dynamic live load is a function of three major parameters: road surface roughness, bridge dynamics, and vehicle dynamics. Live load effect was considered in terms of positive moment, negative moment, and shear force. Significant uncertainties exist regarding the live-load effects on bridges.

The AASHTO LRFD specification design live load model HL-93 was developed using the database adopted from a truck survey conducted in the 1970s in Ontario, Canada. The upper 20 percent of the Ontario truck weight data was selected to develop the model. The maximum live load effects were calculated for one-lane and two-lane girder bridges. The dynamic load was modelled based on test results and simulations of the ratio of dynamic strain to static strain (Hwang and Nowak 1991).

Nowak (1999) calibrated the live load model by extrapolation of the Normal distribution. The live load statistics that Nowak (1999) used for the LRFD calibration are shown in Table 2-2 and 2-3.

A considerable degree of uncertainty is caused by the unpredictability of future trends due to the database limitation. The NCHRP report 368 (Nowak, 1999) calibrated LRFD design code based on live load distribution method and assumed multiple truck presences, no site-to-site differences, and a constant dynamic impact of 0.33.

It is unnecessary to include any deliberate conservatism added to the LRFD load factors in load rating. Rating should remove some conservatism that was inherited in design for possible future load growth and other construction and analysis uncertainties. Any site-specific information known at the time of evaluation should be used, such as traffic, construction records,
Table 2-2 Live load bias factors and COV (HL-93)

<table>
<thead>
<tr>
<th></th>
<th>ADTT=1000</th>
<th>ADTT=5000</th>
</tr>
</thead>
<tbody>
<tr>
<td>One Lane Loaded</td>
<td></td>
<td></td>
</tr>
<tr>
<td>simple span moment</td>
<td>1.23 - 1.36</td>
<td>1.26 - 1.38</td>
</tr>
<tr>
<td>shear</td>
<td>1.17 - 1.28</td>
<td>1.21 - 1.32</td>
</tr>
<tr>
<td>negative moment</td>
<td>1.20 - 1.33</td>
<td>1.23 - 1.36</td>
</tr>
<tr>
<td>Two Lane Loaded</td>
<td></td>
<td></td>
</tr>
<tr>
<td>simple span moment</td>
<td>1.08 - 1.15</td>
<td>1.10 - 1.20</td>
</tr>
<tr>
<td>shear</td>
<td>1.04 - 1.14</td>
<td>1.08 - 1.18</td>
</tr>
<tr>
<td>negative moment</td>
<td>1.10 - 1.22</td>
<td>1.14 - 1.26</td>
</tr>
</tbody>
</table>

Coefficient of variation = 0.12 for all cases

Table 2-3 Coefficient of variation of mean maximum live load and dynamic load

<table>
<thead>
<tr>
<th></th>
<th>Short Span</th>
<th>Normal Span</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single Lane</td>
<td>0.205</td>
<td>0.19</td>
</tr>
<tr>
<td>Two Lane</td>
<td>0.19</td>
<td>0.18</td>
</tr>
</tbody>
</table>

More recent observations made on truck data collected from WIM stations at representative U.S. sites (Sivakumar et al. 2008) have shown that trucks travelling over the U.S. highway system can be significantly different from the biased Ontario truck weight data used during the calibration of the AASHTO LRFD specifications. Sivakumar (2011) performed a recalibration project, considering state-specific, site-specific truck data and actual side-by-side frequency based on six WIM sites in the US. A set of reduced live load factors were recommended and accepted by the AASHTO in July 2012. Sivakumar (2011) used the coefficient of variation, within a site and site-to-site, data limitation factor, dynamic amplification factor, and load distribution factor.

### 2.4.2 Live Load Extreme Value Projection

Observations are always only a part of the occurrence. To project the future long return period load and load effects based on the short return period observations, extreme value models are employed. The most straightforward method is the asymptotic model, sometimes called the parent distribution power rule.

If the observed sample data has a given number \( n \) of independent observations, the maximum of those events, \( X_{\text{max},n} \) is

\[
X_{\text{max},n} = \max(X_1, X_2, \ldots, X_n)
\]
If all of the independent variables, $X_i$, are from the same probability distribution, then the cumulative probability distribution $F_{X_{\text{max}, n}}(x)$ and the probability density function $f_{X_{\text{max}, n}}(x)$ are calculated as:

\[
F_{X_1}(x) = F_{X_2}(x) = \cdots = F_{X_n}(x) = F_X(x) \tag{2.18}
\]

\[
F_{X_{\text{max}, n}}(x) = [F_X(x)]^n \tag{2.19}
\]

\[
f_{X_{\text{max}, n}}(x) = n[F_X(x)]^{n-1}f(x) \tag{2.20}
\]

It can be seen from the above equations that the maximum value distribution can be obtained by raising the initial distribution $F_X(x)$ to the $n^{\text{th}}$ power, and a high precision $F_X(x)$ is crucial.

In order to calculate the 75-year live load occurrence based on a two-week observation, Nowak (1999) used the tail Normal distribution method for the prediction. The inverse cumulative distribution functions ($\Phi^{-1}[F_X(x)]$) were plotted on the normal probability paper to fit the upper tail as shown in Figure 2-4. The upper tails were assumed to have a Normal distribution and were shown as a straight line on the normal probability paper. Therefore, the effects corresponding to the probability of occurrence can be read directly from the plot. This application requires a high precision in the upper tail, which governs the behavior of the extreme value.

Figure 2-4 CDF of moment and shear effect on normal probability paper (Nowak and Hong, 1991)
Using this normal probability plot, the normalized standard deviation of the force effect can be estimated as follows. For example, let \( N \) be the total number of trucks in the time period of \( T = 2 \) weeks surveyed traffic. For \( T = 75 \) years of design life, the number of trucks \( N \) will be 75 years \( \times 52 \) weeks/2 weeks = 1950 times larger than in the survey and result in \( N = 19.5 \) million trucks. The probability level is \( 1/N = 5 \times 10^{-8} \), which corresponds to \( z = \Phi^{-1}(1 - 5 \times 10^{-8}) = 5.33 \) (the inverse standard deviation) on the vertical scale, as shown in Figure 2-4. Therefore, the correspondence moment and shear can be read directly from the figure.

2.4.2.1 Simplified Gumbel Prediction Method

The load effects do not always follow a Normal distribution at the tail, and the CDFs do not always appear as a straight line. Sivalumar et al. (2011) evaluated the normal fit of the tail method and suggested that using a short return period of one day would not be accurate enough to predict longer than one month as shown in Figure 2-5. An alternative simplified statistical analytic method was proposed as shown in the NCHRP report 683. They observed that although the whole parent WIM data may not follow any known probability distribution, the histogram of the upper 5% tail ends may match the Normal distribution well.

![Figure 2-5 Cumulative distribution maximum load effect for different period (Sivalumar et al. 2011)](image)

This method is based on the assumption that if the parent distribution has general Normal distribution, then the maximum value after \( N \) repetitions asymptotically approaches an Extreme Value Type I (Gumbel) distribution (Ang & Tang, 1984). The application of extreme value theory allows the maximum statistic value to be obtained in the closed form (Eq. 2.21 and 2.22).
\[ F_X(x) = e^{-e^{-\alpha(x-u)}} \] (2.21)

\[ f_X(x) = \alpha e^{-e^{-\alpha(x-u)}} e^{-\alpha(x-u)} \] (2.22)

The maximum load effect, \( L_{\text{max}} \), expected over a period having \( N \) repetitions can be determined by the following method:

\[
\begin{align*}
  u_N &= \mu_{\text{event}} + \sigma_{\text{event}} \left( \sqrt{2\ln(N)} - \frac{\ln(\ln(N)) + \ln(4\pi)}{2\sqrt{2\ln(N)}} \right) \\
  \alpha_N &= \sqrt{2\ln(N)} \\
  \overline{L_{\text{max}}} &= \mu_{\text{max}} = u_N + \frac{\gamma}{\alpha_N} \\
  \sigma_{L_{\text{max}}} &= \frac{\pi}{\sqrt{6\alpha_N}}
\end{align*}
\] (2.23-2.26)

In which, \( \sigma_{\text{event}} \) and \( \mu_{\text{event}} \) are the parent distribution mean and standard deviation, respectively, based on upper 5% tail that is fitted to the Normal distribution. If the tail follows the Normal distribution, the Normal plot would produce a straight line with the slope, \( m \), and the intercept, \( n \). The respective mean and the standard deviation of the equivalent Normal distribution are \( \mu_{\text{event}} = -\frac{n}{m} \) and \( \sigma_{\text{event}} = -\frac{1}{m} \). \( \alpha \) and \( u \) are distribution parameters, \( \gamma = 0.577216 \) is the Euler number, \( \alpha_N \) is the inverse dispersion coefficient and, \( L_{\text{max}} \) and \( \sigma_{\text{max}} \) are the mean and standard deviation of the maximum load effect, respectively.

Figure 2-6 illustrates the simplified Gumbel prediction method. The red line is the best-fitted equivalent Normal distribution PDF if the upper 5% tail matches the Normal distribution curve.
There are two more common statistical approaches for extreme value analysis, the Block Maxima method (BM) and the Peak Over Threshold method (POT). The BM method considers and models the maximum events in each time interval (block) or each event when predicting the future maximum value. This method may miss some of the large values if the sample size is not big enough. The POT method collects and analyzes all the peak values exceeding a high threshold value. For the BM method, a generalized extreme value distribution can be used to interpret the distribution (Davison and Smith, 1990). For the POT method, a generalized Pareto distribution can be used. Jaruskova and Hanek, 2006, reported that the two methods should produce reasonably similar results.

2.4.2.2 The Block Maxima Method and the Generalization Extreme Value Distribution (GEV)

The Block Maxima method is commonly used through the extreme value theory for the peak strain values, and it is tested for this study. This method is used for extrapolating data into an evaluation return period by identifying the maximum strain recorded during a loading event or in a reference period of time, such as a day or a week. That blocked data will fit into one of the extreme value distributions to obtain an estimate of the lifetime maximum load effect. The method is based on the assumption that individual events are independent and identically distributed.

Let $x_1, x_2, \ldots, x_n$ be a series of independent observations or events. Then block the data into sequences of observations of length, $n$, generating a series of a block of maxima $X_{\text{max, } n, i}, \ldots, X_{\text{max, } n, m}$ as in Equation 2.27. Then those maxima can be fitted to the GEV distribution.

$$X_{\text{max, } n, i} = \max(x_{i1}, x_{i2}, \ldots, x_{in})$$ (2.27)

The extreme value theory is one of the most commonly used methods for estimating the long-term responses from short-term records. The extreme value analysis prediction uses the extreme value theory (Gumbel 1941, 1954, 1958; Castillo et al. 1988). The generalized extreme value (GEV) (Eq. 2.28) distribution contains three types of distributions, namely, type I-Gumbel, type II-Frechet, and type III-Weibull (Figure 2-7).

$$F(x; \mu, \sigma, \xi) = \begin{cases} e^{-(1+\xi\frac{x-\mu}{\sigma})^{-\xi}}, & \xi \neq 0, \text{ for } 1 + \xi \left(\frac{x-\mu}{\sigma}\right) < 0 \\ e^{-e^{-\left(\frac{x-\mu}{\sigma}\right)}}, & \xi = 0 \end{cases}$$ (2.28)

where $-\infty < \mu < \infty$ is the location parameter, $0 < \sigma < \infty$ is the scale parameter, and $-\infty < \xi < \infty$ is the shape parameter.

I. Gumbel is the smallest extreme value distribution, if $\xi=0$
II. Frechet is the largest extreme value distribution, if $\xi>0$
III. Weibull is the extreme value distribution with a limit, if $\xi<0$
Both Type I and Type II have an unlimited tail length that is possible for predicting the maximum future value. Type III is suitable for prediction when limits exist.

### Generalized extreme value densities

![Generalized extreme value densities](image)

The inverse of the GVE distribution function for the maxima, represents the quantile of 1-p, p is the probability as $P(x > x_p) = p$, which can be shown as:

$$x_p = \begin{cases} 
\mu - \sigma \log(-\log(1 - p)), & \text{for } \xi = 0 \\
\mu - \frac{\sigma}{\xi} (1 - (-\log(1 - p))^{-\xi}), & \text{for } \xi \neq 0
\end{cases} \quad (2.29)$$

$x_p$ is the return level with the return period of 1/p. Equation 2.29 can be used to predict the future value of the live load effects.

2.4.2.3 The Peaks Over Threshold Method (POT) and the Generalized Pareto Distribution (GPD)

If the parent sample size is small, the GEV may not be accurately fitted to the actual CDF of the maximum. The peak over threshold method can keep all maxima values for the extreme value prediction. For the POT method, a generalized Pareto distribution can be used. The

---

Figure 2-7 Generalized extreme value distribution
cumulative distribution function of the GPD with shape ($\xi$), and scale location parameters ($\sigma$), is defined as

\[
F(x; u, \sigma, \xi) = \begin{cases} 
1 - \left[1 + \frac{x - u}{\sigma} \xi \right]^{-1}, \xi \neq 0, \\
1 - e^{-\frac{x-u}{\xi}}, \xi = 0.
\end{cases}
\] (2.30)

The inverse of the distribution function of the GPD for the upper tail, $F^{-1}(1-p)$, represents the quantile of 1-p for the excess over the threshold. For $x>u$, the return level of $x_p$ can be calculated as:

\[
x_p = \begin{cases} 
u - \frac{\sigma}{\xi} (1 - p^{-\xi}), \text{for } \xi \neq 0 \\
u - \sigma \log(p), \text{for } \xi = 0
\end{cases}
\] (2.31)

### 2.4.3 Resistance Variances (R)

The capacity of a bridge depends on the resistance of its components and connections. The component resistance, $R$, is determined mostly by material strength and dimensions. The random variable, $R$, can be considered as a product of the following parameters (Nowak 1999):

\[
R = MFPR_n
\] (2.32)

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;

$R_n$ = nominal resistance.

Bias factors ($\lambda$) and coefficients of variation (V) are determined for the material factor (M), fabrication factor (F), and analysis factor (P). Factors M and F can be combined. The R parameters are calculated as follows:

\[
\lambda_R = (\lambda_{FM})(\lambda_P) \quad (2.33)
\]

\[
V_R = \sqrt{(V_{FM}^2 + V_P^2)} \quad (2.34)
\]

Many researchers have studied the statistical parameters (Ellingwood et al. 1980, Nowak et al. 1994, Nowak and Zhou 1990). The LRFD statistical parameters are shown in Table 2-4 as...
in the AASHTO calibration report (Nowak, 1999). A Lognormal distribution was used for resistance (Akgul and Frangopol 2005).

Unlike the design that relies on assumed material properties, the bridge construction records, including the material testing records are available for most bridges to use for bridge rating. As such, we may use the actual material strength instead of the nominal strength to evaluate the bridge. Frequently, the actual strength is higher than the design strength. We may also use the actual material strength distribution to analyze the reliability index.

However, the bridge capacity is also a time dependent variance. For an existing bridge having served for decades, the current bridge condition and future service conditions, such as the live load condition, environmental condition, and maintenance condition, will greatly affect the resistance. Researchers have performed studies on capacity degradation models based on deterioration and other environmental factors. (Akgul and Frangopol 2005, McCuen and Albrecht 1994). In lieu of the assumed material resistances, a load test can be used to evaluate the current bridge capacity, and a SHM system may provide information regarding the bridge component conditions and performance.

<table>
<thead>
<tr>
<th>Type of Structure</th>
<th>FM</th>
<th>P</th>
<th>R</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\lambda$</td>
<td>$\nu$</td>
<td>$\lambda$</td>
</tr>
<tr>
<td>Non-composite steel girders</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Moment (compact)</td>
<td>1.095</td>
<td>0.075</td>
<td>1.02</td>
</tr>
<tr>
<td>Moment (non-compact)</td>
<td>1.085</td>
<td>0.075</td>
<td>1.03</td>
</tr>
<tr>
<td>Shear</td>
<td>1.12</td>
<td>0.08</td>
<td>1.02</td>
</tr>
<tr>
<td>Composite steel girders</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Moment</td>
<td>1.07</td>
<td>0.08</td>
<td>1.05</td>
</tr>
<tr>
<td>Shear</td>
<td>1.12</td>
<td>0.08</td>
<td>1.02</td>
</tr>
<tr>
<td>Reinforced concrete</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Moment</td>
<td>1.12</td>
<td>0.12</td>
<td>1.02</td>
</tr>
<tr>
<td>Shear w/steel</td>
<td>1.13</td>
<td>0.12</td>
<td>1.075</td>
</tr>
<tr>
<td>Shear no steel</td>
<td>1.165</td>
<td>0.135</td>
<td>1.20</td>
</tr>
<tr>
<td>Pre-stressed concrete</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Moment</td>
<td>1.04</td>
<td>0.045</td>
<td>1.01</td>
</tr>
<tr>
<td>Shear w/ steel</td>
<td>1.07</td>
<td>0.10</td>
<td>1.075</td>
</tr>
</tbody>
</table>

2.5 RELIABILITY ASSESSMENT

Reliability is a time-dependent variable through the bridge life span. Not only is it subject to the influences of traffic, maintenance, and deterioration, but also is subject to modification from additional site data. As bridges approach the end of their service life, accurately evaluating their condition and load rating these structures become increasingly important. As previously stated, the LRFD and the MBE were calibrated using conservative assumptions of performance. In the actual evaluations, if some or all such conservative assumptions were to be replaced by measured or observed values, it is likely that the safety indices could be significantly higher. It is
also essential to consider the current bridge conditions and the bridge performance corresponding specifically to the periods between inspections, normally two years. Data from the most recent inspection provides the basis for the reduction of some uncertainties and assumptions made for design.

2.5.1 Reliability Index

Hasofer and Lind (1974) introduced the reliability index as the shortest distance from the origin of the reduced variables to the state function line \( g(Z_R, Z_S) = 0 \), which is illustrated in Figure 2-8 (Nowak and Collins 2000).

![Figure 2-8 Limit-state functions](image)

Nowak assumed that the total load, \( S \) or \( Q \), is a normal random variable and the resistance, \( R \), is a lognormal random variable. The modified first-order second-moment reliability index for this combination is expressed in the following equation (Nowak and Collins 2000):
\[ \beta = \frac{R_n\lambda_R(1 - kV_R)[1 - \ln(1 - kV_R)] - \mu_S}{\sqrt{[R_nV_R\lambda_R(1 - kV_R)]^2 + \sigma_S^2}} \]  

(2.35)

where

\( k = \) the measure of the shift from the mean value in standard deviation units; \( k \) is assumed to be equal to 2

Ghosn and Moses (1985) found that the load and resistance factors obtained following a calibration are relatively insensitive to errors in the statistical data base as long as the same statistical data and criteria used to find the target reliability index and criteria are also used to calculate the load and resistance factors for the code.

Moses (2001) recommended a first-order reliability rating approach to evaluate the safety index directly. Using the simplified lognormal format and mean values, one can obtain \( \beta \) using the following equations:

\[ \beta = \frac{\ln \left( \frac{\mu_R}{\mu_S} \right)}{\sqrt{V_R^2 + V_S^2}} \]  

(2.36)

\[ \mu_S = \mu_R \exp \left[ -\beta \sqrt{V_R^2 + V_S^2} \right] \]  

(2.37)

\[ \mu_S = \mu_{DL} + RF(\mu_{LL}) = \mu_R \exp \left[ -\beta \sqrt{V_R^2 + V_S^2} \right] \]  

(2.38)

where, \( RF=\)Rating factor

A rating factor can be derived from Equation 2.38, considering the mean values of resistance, dead load, and live load; their respective coefficients of variation; and the target reliability index. The statistics Moses (2001) used for screening are provided in Table 2-5.

<table>
<thead>
<tr>
<th>Case</th>
<th>( \lambda )</th>
<th>( V )</th>
<th>Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead Load</td>
<td>1.04</td>
<td>0.08</td>
<td>Normal</td>
</tr>
<tr>
<td>Live Load</td>
<td>1.00</td>
<td>0.18</td>
<td>Lognormal</td>
</tr>
<tr>
<td>Resistance</td>
<td>1.12</td>
<td>0.10</td>
<td>Normal</td>
</tr>
</tbody>
</table>

### 2.6 BRIDGE RATING

Instead of rating bridges using the safety index directly, the design and rating specifications use design or rating factors to avoid the complicated reliability analysis procedures. The load rating is generally expressed as a rating factor for a particular live load model and is defined using the following rating equations (MBE 6A.4.2.1-1):
\[
RF = \frac{\varphi_c \varphi_s \varphi R_n - \gamma_{DC}(D_{C1} + D_{C2}) - \gamma_{DW}D_W}{\gamma_{LL}L_L}
\]

(2.39)

\[
\varphi_c \varphi_s \geq 0.85
\]

(2.40)

Where,
- \(\varphi_c\) = Condition Factor
- \(\varphi_s\) = System Factor
- \(\varphi\) = LRFD Resistance Factor
- \(\gamma\) = Load factors for DC, DW and LL
- \(R_n\) = Nominal Resistant

The system factor, \(\varphi_s\), consider ductility and system redundancy. System reserve helps justify the reliability targets inherent in the operating stress levels since reliability indexes are calculated for individual components or member limit states. The system \(\beta\) on ultimate strength were higher than component \(\beta\), typically by an increase of 1.0 (Moses 2001). Restricting the operating level of rating only to spans with known redundancy would then ensure that an operating rating of 2.5 for components actually implied a system \(\beta\) of 3.5. The use of system properties in the evaluations leads to more uniform safety indexes among different spans with respect to failure. The system factor varies from 0.85 to 1.0 based on the MBE.

The condition factor, \(\varphi_c\), is determined based on recent field inspection. The condition factor provides a reduction to account for the increased uncertainty in the resistance of deteriorated members and the likely increased future deterioration of those members during the period between the inspection cycles. The aim is to select a value of system factors that keeps the safety index for deteriorated components at the same level as the target safety index adopted in the calibration of the evaluation factors. The condition factor varies from 0.85 to 1.0.

Load factors, \(\gamma\), depend on the type of load effect being considered. For the design truck HL-93, the inventory rating load factor is \(\gamma_{LL} = 1.75\) (for bridge design life), and the operating load factor is \(\gamma_{LL} = 1.35\) (for 5 years). For legal and permit trucks, the load factors depend on the average daily truck traffic (ADTT) at the bridge site.

If any legal truck has a RF < 1.0, a load posting (restriction) is required for that type of truck, and no overload trucks are allowed.

### 2.7 BRIDGE POSTING

Bridge load posting is required when routing legal or permit trucks exceeds the safe load capacity. Louisiana bridge postings are set to restrict the gross weight of the vehicles with a single tonnage of total gross weight. The posting signs shall be simple, easy to understand, and follow the *Manual on Uniform Traffic Control Devices* (MUTCD). Standard Louisiana load posting signs are shown in Appendix A.
The rating factor obtained may be used to determine the bridge safe load capacity in tons as in Equation (2.41). If RF < 0.3, the bridge shall be closed to that legal truck type. If the bridge capacity is less than 3 tons, it should be closed.

\[
Posting \ Load = (RF - 0.3) \left( \frac{GVW}{0.7} \right) \tag{2.41}
\]

A posting for a short truck model would be too restrictive and a posting based on long combination would be too liberal. Therefore, two types of vehicles are used to represent general vehicles: single-unit vehicles and combination vehicles.

A single-posting load cannot effectively capture the variety of vehicle types. The AASHTO and the LADOTD recommend that the posting vehicles are established to envelope most of the national and state vehicle types. The truck that produces the highest moment or shear will govern the posting values. If a certain type of vehicle is very rarely operated for a specific bridge location, then, setting a weight limit for that type of vehicle would penalize all other types of vehicles. Therefore, the bridge should be posted for the site-specific trucks also.

### 2.8 RESEARCH APPROACHES

The bridge site selected for this study is a multi-girder bridge located between New Orleans and Laplace in Louisiana on route US61 crossing the Bonnet Carre Spillway. There are major petrochemical industries along this route, and the bridge has been used to carry super overloads of up to 2,000,000 pounds gross weight. The LADOTD has installed a WIM system and a SHM system at the west end of the bridge. These systems provide the most up-to-date traffic and structural responses from ambient vehicular loads, thus, eliminating many of the assumptions needed to rate a bridge.

The most significant uncertainty associated with bridge load rating is the traffic. The traffic on a specific route is greatly dependent on the local conditions as evident from a LADOTD study. The use of design traffic for load rating may or may not represent the true loadings on the bridge. Using continuously collected vehicle record data via WIM over a sufficient period provides more accurate live load models. In addition, the safety levels among various types of structures and span lengths are not consistent due to the simplifications applied when calibrating the design codes. The SHM responses from the individual bridge being monitored can present the actual behavior of the specific bridge, including changes in structure performance. A probability-based reliability index assessment can provide the actual safety level of the structure directly. Applying the extreme value distribution theory for projecting the future maximum live load effects can overcome the limitations of the shorter data acquisition periods.

At the US61 Bonnet Carre Bridge, the real-time loads and responses (strains, deflections, truck weights and configurations, etc.) measured directly from the bridge under in-service traffic was continuously collected throughout the study period. The load rating can be obtained by comparing the element strain limit to the live load strains measured directly without calculating load effects. Through this comparison and using the measured response as the model, a
continuous reliability based load rating system based on real-time live load and load response is established.

With the measured live load-response and the truck loads, the bridge can be rated with less conservatism and reduced uncertainties from live load distribution, dynamic impact, and the secondary and non-structural element effects. Other benefits of in-service rating include the possibility of identifying any sudden bridge condition changes, and the possibility of using the extreme-value theory to forecast future rating, load permitting, and bridge management. Typically, the bridge is evaluated at four different levels:

1. Live load distribution method (Level I)

In the beginning, the as-designed bridge rating starts with a conventional approximation method based on the AASHTO LRFD live load distribution formulas. Those formulas were developed for common bridge types, dimensions, and HS trucks. Generally, this is a simplified method developed based on the reliability theory. The LRFD and LRFR methods have been calibrated for a global population of bridges based on the target reliability index of 3.5 at the inventory level and 2.5 at the operating level. This rating method considers the individual structural component’s greatest possible load effects and is generally on the conservative side.

For this rating, all bridge files have been collected and reviewed. The bridge files include as-built plans, design specifications, inspection records, and rating records. The bridge construction documents, including the construction sequence and the material testing records have also been reviewed. First, a basic bridge as-designed load rating based on the live load distribution method specified in the AASHTO code was performed. Next, the as-built concrete strengths were considered for the as-built rating.

2. Refined analysis method – finite element model (FEM) (Level II)

Following Level I, a refined finite element analysis was employed to improve the accuracy by including the overall structure system. Finite element model load rating is a common method used when the loading parameters or bridge parameters are outside of the range of limitation for a typical structure that can be simulated with simplified assumption, such as the Level I method. The FEM considers the overall bridge system behavior to evaluate the most likely bridge true response, such as live load distribution.

There are different levels of FEMs that can be chosen for slab-on-girder bridges (Hays et al. 1986, Tarhini and Frederick 1992, Zokaie et al. 1991). Several researchers have concluded that a simple 2-D model provides good accuracy relative to the field measurement (Zhang and Aktan, 1997; Mabsout et al. 1997)

A baseline model is an analytical based representation of the physical structure that predicts that bridge response under a defined loading condition. Thus, a 2-D finite element bridge model was created as the baseline model of the US61 bridge. The refined analysis was used to improve the live load distribution. Compared with the estimated live load distribution
method, the FEM model is a more realistic representation of the bridge because it considers the whole bridge behavior instead of only the element behavior used in the simplified analysis.

3. Nondestructive load testing method (Level III)

As an alternative method for analytical bridge load rating, nondestructive load testing is another load rating procedure that can reduce the difference between the theoretical bridge evaluation and the actual bridge behavior.

Load testing is the observation of the response of a bridge subjected to controlled and predetermined loadings from actual measurements without causing changes in the elastic response of the structure. The diagnostic load testing is performed to determine bridge responses to known imposed loads, to evaluate the actual live load distribution, or to validate analytical procedures within service limit. The tests can reduce the uncertainties related to material properties, boundary conditions, cross-section contribution, and damage. Cai and Shahawy (2004) have found that the field test gives better rating results from the concrete strengths and non-structural components contribution.

Interpretation of the test results means deciding how much of the load carrying capacity observed in the test should be utilized in establishing the bridge load rating instead of the predicted values. The following equation is used to modify the calculated load rating (AASHTO MBE).

\[ RF_T = RF_C K \] (2.42)

- \( RF_T \) = load-rating factor base on the load test result
- \( RF_C \) = load-rating factor base on calculation
- \( K \) = adjustment factor from the comparison of measured test behavior with the analytical model. K represents the benefits of the field load test, if any.

To obtain more actual bridge responses, the theoretical FE model will be calibrated through a load test. Sanayei et al. (1991, 1992) developed static stiffness-based and static flexibility-based error functions for optimization. Sanayei and Saletnik (1996) extended these methods to the calibration of the strain measurements for the finite element model. Sanayei et al. (1997) applied these methods to a laboratory steel frame and successfully updated section properties at the component level. Similar quadratic scalar objective functions were defined by Schlune et al. (2009) and were used in conjunction with engineering judgment for manual model updating.

The load test measurements were integrated into the model calibration and the bridge load rating. The flexural stiffness of concrete and boundary parameters were the main parameters used for the FE model calibration for this study. The calibrated model can more realistically represent the actual structural response.

4. In-service based WIM, SHM and reliability analysis method (Level IV)
The structural health monitoring system is a combination of local and global non-destructive experimental technologies coupled together with the advanced structural analysis and modeling techniques to complement inspections and provide continuous information regarding bridges behavior (Frangopol, 2001). Currently, SHM has been used as a tool for evaluating major bridge conditions that are difficult to accomplish by inspection or the routine load rating method. This study proposes a method to allow for the use of monitored data for bridge load rating and reliability analysis.

Engineers realize the advantages of using actual measured live loads and responses in bridge load rating. Many researchers have used measured in-service live loads and load responses to improve load rating models and project bridge performances. Cardini and DeWolf (2008) collected SHM strain data to develop live load distribution factors and peak strain values. Alampalli and Lund (2006) used measured strain data to predict the remaining fatigue life. Bhattacharya (2005) applied the in-service probability-based rating method to account for both site-specific traffic and as-built bridge response. Liu, et al. (2009) suggested a bridge reliability assessment using the limit state equation based on the long-term strains induced by heavy vehicles. However, none of them have ever collected data long enough to verify the prediction method or performed a reliability analysis to validate the results.

To systematically evaluate the bridge, an instrumentation plan was developed for the US61 Bonnet Carre Bridge with the objective of load rating improvement in mind. The bridge was instrumented with a synchronized structural health monitoring system, a weigh-in-motion system, and a static camera. This bridge has been monitored for over one year to cover the seasonal traffic variances.

The traffic data including the truck configuration and timestamp was first sorted and filtered to eliminate unreliable observations. The scrubbed data was then used to develop the traffic patterns, truck statistics, and site-specific truck configurations. The future truck gross weight was projected using the extreme value theory. The bridge was load rated based on these projected trucks.

The strain-based structural health monitoring system data was also collected during the same period. Statistical analyses were performed on the maximum peak strain readings under the ambient traffic for strain distribution and projections. Finally, the reliability analysis was used to recalibrate the site-specific live load factors and live load distribution factors.

The rating results of the four rating levels were summarized and compared. Based on the analysis results, a more realistic load posting sign was recommended.
CHAPTER 3  DESCRIPTION OF CASE STUDY

To take advantage of a fully instrumented bridge, the US61 bridge over the Bonnet Carre Spillway was selected to illustrate the response-based load rating method, and the results are compared with the traditional load rating methods. The subject bridge is a multi-girder prestressed concrete bridge instrumented with a SHM system and a WIM station at the west end of the bridge. The in-service traffic data and the bridge strain data have been acquired continuously for over a year. All four levels of load rating have been performed on this bridge, including the load distribution method, the finite element method, the NDT load testing method, and the in-service WIM data and SHM data reliability analysis method.

3.1 US61 BONNET CARRE SPILLWAY BRIDGE HISTORY

The westbound Bonnet Carre Spillway Bridge (Figure 3-1) is a 6,005.91 ft long prestressed concrete bridge located in St. Charles Parish, Louisiana. This is one of the US61 twin bridges constructed in 1984. The Bonnet Carre Spillway bridge provides important access for many industries along the Mississippi River. This bridge has experienced many heavy overloads, including a 1.8 million pounds overload recently (Figure 3-2) and several over one million pound overloads in its history. An accurate evaluation of its load carrying capacity is crucial to the industry and the public. Being the newer one of the twin bridges, the westbound bridge was designated to carry all of the heavy overloads. Therefore, the westbound bridge was selected for this study to ensure the safety of the structure, the hauler, and the traveling public.

![Figure 3-1 Bonnet Carre Bridge](image_url)
3.2 GEOMETRY AND CROSS SECTION PROPERTIES

This bridge consists of twenty-five 232’-6” long 3-span units (each unit consists of three 77’-6” spans) and two simple-span (33’-9”) units. There are total seventy-nine spans in the bridge. All 3-span units are essentially identical and constructed to be continuous for live loads. The existing load rating file shows that the 3-span units are the control spans for the bridge. Therefore, one of the 3-span units was selected for this study.

The bridge roadway is 40’ wide and consists of two 12’ lanes with a ten-foot and a six-foot outside and inside shoulders, respectively. The reinforced concrete Jersey barriers are continuous with joints located at 1/3 of the spans. The bridge plan and the typical cross section are shown in Figure 3-3 and Figure 3-4, respectively. Due to the symmetry of the bridge, only half of the 3-span bridge is shown here.

The superstructure is composed entirely of AASHTO Type III prestressed-precast concrete girders with a composite 7½” thick cast-in-place concrete deck. The six girders are equally spaced at 7’-4”.

The pile bent substructure is composed of cast-in-place concrete bent cap supported by 5-24” square precast concrete piles.

Figure 3-2 1.8 million pounds overload
Figure 3-3 Bridge plan
Figure 3-4 Bridge typical cross section
The girder sections and strand layout at mid-span and at girder end are shown in Figure 3-5. The girder section properties can be found in Table 3-1.

![Girder Sections and Strand Layout](image)

Figure 3-5 Girder sections and strands layout

<table>
<thead>
<tr>
<th>Girder Type</th>
<th>Area (in²)</th>
<th>Moment of Inertia (in⁴)</th>
<th>Distance to Centroid from Bottom (in)</th>
<th>Distance to Centroid from Top (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Non-Composite Girder</td>
<td>559.5</td>
<td>125,390.3</td>
<td>20.27</td>
<td>24.73</td>
</tr>
<tr>
<td>Composite Girder - Interior</td>
<td>1140.0</td>
<td>381,943.6</td>
<td>35.47</td>
<td>18.53</td>
</tr>
<tr>
<td>Composite Girder - Exterior</td>
<td>1114.8</td>
<td>373,741.2</td>
<td>35.05</td>
<td>18.98</td>
</tr>
</tbody>
</table>

The girders were designed as simple-supported spans for non-composite dead loads, and as continuous spans under live loads. This method takes advantage of the continuity connection to reduce the maximum positive moment at the mid-span. Structural continuity was achieved by providing cast-in-place continuity diaphragms and negative moment reinforcement in the deck. The concrete girders are reinforced with 6-#6 bars bent into the continuity diaphragms at the continuity ends for the positive moment at the support, as shown in Figure 3-6. Although there is currently no problem with this bridge, this detail has caused cracking for some similar bridges due to the restrained forces. This continuity detail has been abandoned by the LADOTD and a few other states.
3.3 BRIDGE CONDITION

The 2011 inspection report indicated that the bridge was in good condition with the NBI condition rating of 7, 8, and 7 for deck, superstructure, and substructure, respectively on a scale of 0 to 9. There were no visual sign of cracking on girders or diaphragms. The noted deficiencies included minor transverse cracks in the deck and a few girder end cracks (Figure 3-7), which should not affect the bridge capacity. As of the last traffic estimate, the average daily traffic (ADT) on this route is 13,320.

3.4 SPECIFIED BRIDGE MATERIAL PROPERTIES

The bridge was designed in the early 1980s based on the AASHTO Standard Specifications, and the HS-20 truck was the design live load.
The specified material properties from as-built plans are shown as following:

- **Deck concrete strength:** \( f_{c}^{'}=3.2 \text{ ksi} \)
- **Girder concrete strength:** \( f_{c}^{'}=5.0 \text{ ksi}; f_{ci}^{'}=4.0 \text{ ksi} \)
- **Pile cap concrete strength:** \( f_{c}^{'}=3.0 \text{ ksi} \)
- **Reinforced steel strength:** \( f_{y} = 60 \text{ ksi} \text{ or } f_{y}=40 \text{ ksi} \)
- **Stress-relieved strands:** 30-½”, \( f_{pu}=270 \text{ ksi}, f_{pu}=28,910 \text{ lbs}, \text{ draped} \)
- **Continuity diaphragm:** 6-#6 bars
- **Concrete modulus of elasticity:** \( E_{c} = 33000K_{1}w_{c}^{1.5} \sqrt{f_{c}^{'}^{2}} = 1820\sqrt{f_{c}^{'}^{2}} \text{ ksi} \)
- **Strand modulus of elasticity:** \( E_{s} = 29,000 \text{ ksi} \)

The measured compressive strengths of the concrete during construction were available for the prestressed concrete girders. The measured compressive strength data was used for the as-built rating as introduced in Chapter 4.

### 3.5 INSTRUMENTATION (SHM SYSTEM AND WIM STATION)

In lieu of relying on the pre-defined traffic information and a purely analytical bridge model, as typically done in routine bridge rating, a long-term health monitoring system and a weigh-in-motion station have been installed to provide real-time quantitative data for bridge evaluation. With this data, a response-based in-service bridge load rating under ambient traffic was developed, which can project future bridge ratings based on the statistical analysis of the actual live loads.

The installed SHM system includes a data acquisition system (two Campbell Scientific CR-3000s), strain transducers (BDI ST-350), tiltmeters (BDI tiltmeter), and linear variable differential transformers (LVDTs). Two spans were instrumented, and strain gages were positioned at the mid-span and also close to the girder ends. A piezoelectric WIM system was installed on the pavement on the west end of the bridge to record the unbiased traffic data. The WIM system was located at both traffic lanes to record all of the trucks. A camera mounted on the instrumentation pole offers photographic verification of the WIM data. The SHM and WIM data-acquisition systems were synchronized. A diagnostic live load test was performed to improve the bridge analytical model.

### 3.6 LOAD RATING SPECIFICATIONS

The following specifications were used in the response-based in-service bridge loading of US61 bridge.

*The AASHTO LRFD Bridge Design Specifications, 7th Edition, with 2015 Interim*

3.7 FOUR LEVELS OF BRIDGE LOAD RATING AND COMPARISON

The purpose of this study is to establish a site-specific rating based on bridge response and reliability analysis instead of using the traditional deterministic method. The site-specific load rating method can improve the accuracy of bridge load rating and provide a uniform level of bridge safety.

To correctly evaluate the bridge and compare the rating methods, four different levels of rating were performed for the same bridge. As discussed in Chapter 2, the four rating levels are live load distribution factor method, refined analysis method (FEM), nondestructive live load testing method and in-service based reliability analysis method. The site-specific live load distribution factors and the live load factors were developed based on probability analysis. The rating results are summarized in Chapter 8.
CHAPTER 4 AS-DESIGNED AND AS-BUILT LOAD RATING (LEVEL I AND LEVEL II)

Bridge load rating provides a basis for determining the load capacity of a bridge. The rating of a bridge depends on structure types, structure conditions, material properties, loads, and traffic conditions at the specific bridge site.

The initial bridge load rating is the as-designed load rating, which is based on the bridge as-bid records only. As-designed load rating is normally performed during the bridge design phase and the as-designed load rating results are shown as part of construction bidding documents. After construction, a set of as-built plans that show the state of the bridge at the end of construction shall be developed. As-built load rating shall be performed based on the as-built conditions before the bridge opened to the public.

Later, the bridge experiences deterioration and begins to degrade, the uncertainties associated with the bridge resistance will increase. A rating may require including the bridge condition factor, which provides a reduction to the bridge resistance to account for the deterioration.

Whenever feasible, the simplified evaluation procedure is applied first before shifting to higher-level evaluation methods. In general, the load rating of a typical bridge structure starts with the simplified analysis method, the live load distribution method, and is based on the construction plans or as-built plans, field inspection, and pre-defined live loads. When the simplified method indicates insufficient capacity or when the simplified methods are not suitable, a refined analysis method, such as the finite element method can be performed. To illustrate the progressive improvement of rating, details of these two methods (Level I and Level II) are presented in the chapter below.

4.1 LIVE LOAD DISTRIBUTION METHOD (LEVEL I): AS-DESIGNED RATING

The approximate method of live load distribution analysis as described in the LRFD is the first level for bridge evaluation. Its validity has been verified for parameter variations within pre-defined ranges. (Tarhini and Frederick 1992, Puckett et al. 2007). The simplified method tends to be somewhat conservative. This method is used to calculate the loading effects and identify critical locations and critical limit states. The bridge rating software AASHTOware BridgeRating (BrR) is commonly used to perform the Level I rating using the Line Girder Analysis function, which is the Distribution Factor Method for the strength limit state and the service limit state as described in the LRFD Article 4.6.2.

The Level I baseline rating of the 3-span AASHTO Type III prestressed-precast concrete girder structure was analyzed and rated in accordance with the AASHTO LRFD and the MBE LRFR 2011 edition with 2015 Interim revisions. The bridge superstructure was modeled with the software BrR as show in Figure 4-1 and Figure 4-2. The girder capacities of the US61 bridge were calculated based on the AASHTO LRFD specifications. The uncracked, transformed, composite cross-sectional properties are tabulated in Table 3-1.
Figure 4-1 Partial bridge plan in AASHTOWare BrR
Figure 4-2 Bridge section in AASHTOWare BrR
Considering that the AASHTO LRFD was first calibrated based on the approximate simple live load distribution factor method; likewise, the baseline as-designed rating also starts with the live load distribution factor method. The live load distribution factor is defined as the ratio of the live load carried by a component and is generated by a lane load placed on the girders. The distribution factors of live loads for moment in interior longitudinal beams are shown in equations (4.1) and (4.2).

\[
\text{One Lane } \quad g_{\text{moment}} = 0.06 + \frac{S}{14} \left( \frac{S}{L} \right)^{0.4} \left( \frac{K_g}{12.0Lt_s^2} \right)^{0.1} \quad (4.1)
\]

\[
\text{Multi Lanes } \quad g_{\text{moment}} = 0.075 + \frac{S}{9.5} \left( \frac{S}{L} \right)^{0.6} \left( \frac{K_g}{12.0Lt_s^2} \right)^{0.1} \quad (4.2)
\]

The distribution factors of live loads for shear in interior longitudinal beams are shown in the equations (4.3) and (4.4).

\[
\text{One Lane } \quad g_{\text{shear}} = 0.36 + \frac{S}{25.0} \quad (4.3)
\]

\[
\text{Multi Lanes } \quad g_{\text{shear}} = 0.2 + \frac{S}{12} - \left( \frac{S}{35} \right)^{2.0} \quad (4.4)
\]

where,

\[g = \text{ live load distribution factor}\]
\[S = \text{ spacing of beams (ft)}\]
\[L = \text{ span length of beam (ft)}\]
\[K_g = \text{ longitudinal stiffness parameter (in)}\]
\[t_s = \text{ depth of concrete slab (in)}\]

The calculated interior girders live load distribution factors of the bridge for this study are shown in Table 4-1:

| Table 4-1 Interior girder live load distribution factors - AASHTO |
|----------------|----------------|----------------|----------------|
|                | Moment         | Shear          |
| Single lane    | Multi-Lane     | Single lane    | Multi-Lane     |
| 0.472          | 0.652          | 0.653          | 0.767          |

Following the design method, the rating model considers multi-span structures to be simply supported for beam self-weight and uncured deck, and continuously supported for composite dead and live loads.

The methodology for the load and resistance factor rating of bridges is comprised of three distinct procedures: 1) design load rating, 2) legal load rating, and 3) permit load rating.
Design load rating is based on the HL-93 loading and its present condition. It is the measurement of the performance of existing bridges using current LRFD bridge specifications. The inventory strength rating reliability is at the same level as the LRFD design, which is β=3.5. The operating level rating is calibrated based on β=2.5. The legal rating is the load rating for the AASHTO and the State legal loads. The results of the legal load rating could be used as a basis for load posting or bridge strengthening. The LADOTD legal gross weight limit, regardless of the number and type of axles (without a tridum or a quadrum axles), is 80,000 pounds; the legal limit of vehicles having a tridum or quadrum axle is 83,400 pounds for interstate highways and 88,000 pounds for non-interstate highways. The maximum legal axle weights are 22,000 pounds, 37,000 pounds, 45,000 pounds and 53,000 pounds for single, tandem, tridum, and quadrum axles, respectively. See Appendix A.2 for the list of Louisiana legal vehicles. Permit load rating checks the safety and serviceability of bridges in the review of permit applications for the passage of vehicles above the legally established weight limitations. Louisiana typical permit vehicles up to 260,000 pounds are shown in Appendix A.3 and A.4. Any vehicle with a gross weight more than 254,000 pounds is considered as super-load, which requires special permit review and not not included in this study.

The AASHTO LRFD design truck HL-93, Louisiana legal trucks including special hauling vehicles, and LA routine permit trucks were used for this study. The strength limit state and the service III limit state were checked for rating at the tenth points of each beam and at locations near the support. The impact factor (IM) is 33%. Based on the site Average Daily Truck Traffic (ADTT), the load factors for the strength limit state and the service III limit state are shown in Table 4-2 (MBE, 2011). See Figure 4-3 to Figure 4-6 for moment and shear diagrams.

<table>
<thead>
<tr>
<th>Limit States</th>
<th>Dead Load</th>
<th>Live Loads</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Design Loads</td>
<td>Legal Loads</td>
<td>Routine Permit Loads</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Inventory</td>
<td>Operating</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strength</td>
<td>1.25</td>
<td>1.35</td>
<td>1.75</td>
<td>1.30</td>
</tr>
<tr>
<td>Service</td>
<td>1.00</td>
<td>0.8</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>

The flexural critical location is at a distance of around 40% of the first span. For the Strength Limit State, the capacity is calculated as $C = \phi_c \phi_s \phi M_n = 4611.67$ kip-ft. Based on the most recent bridge inspection, the condition factor ($\phi_c$), system factor ($\phi_s$), and resistance factor ($\phi$), are all equal to 1.0. The maximum dead load effect is $M_{DL}=919.27$ kip-ft, and the HL-93 truck flexure effect $M_{LL,HL93}=1007.16$ kip-ft. The maximum legal load effect for SU7 truck flexure effect is $M_{LL,Legal}=821.48$ kip-ft. Therefore, based on the bridge load rating Equation (2.39), the flexure strength rating factors (RF) are 1.96, 2.53 and 3.14 for inventory, operating, and legal ratings, in sequence. The routine permit load rating is 3.11 for ovl'd #3. The critical shear location is at span-1 and at 65% of the span. The shear load capacity is calculated as $C = 0.9x181.2 = 163.8$ kips. The shear forces caused by dead loads, HL-93, LA Type 8 are $V_{DL}=16.45$ kip, $V_{LL,HL93} = 57.17$ kips, and $V_{LL,Legal} = 47.53$ kips. The shear strength rating factors are 1.42, 1.85 and 2.28 respectively for design inventory, operating and legal truck rating.
Figure 4-3 Dead load moment diagram

Figure 4-4 Live load HL-93 moment diagram
Figure 4-5 Dead load shear diagram

Figure 4-6 Live load HL-93 shear diagram
The critical section for the service limit state for the design truck is at 40% of the span-1. The bottom flange dead load stress is 1.8606 ksi, and the design live load stress is 1.10 ksi. Allowable tensile stress is $0.19\sqrt{f_c} = 0.19 \sqrt{5} = 0.425 ksi$ per LRFD Article 5.7.3.4. Therefore, the design load service III limit state inventory rating factor is 1.34. The superstructure interior girder as-designed ratings are summarized in Table 4-3.

Table 4-3 Bridge as-designed rating summary by live load distribution method (G3)

<table>
<thead>
<tr>
<th>Live Load</th>
<th>Level</th>
<th>Rating Factor (RF)</th>
<th>Controls</th>
</tr>
</thead>
<tbody>
<tr>
<td>HL-93</td>
<td>Inventory</td>
<td>1.34</td>
<td>Service</td>
</tr>
<tr>
<td>HL-93</td>
<td>Inventory</td>
<td>1.42</td>
<td>Shear</td>
</tr>
<tr>
<td>HL-93</td>
<td>Inventory</td>
<td>1.96</td>
<td>Flexure</td>
</tr>
<tr>
<td>Legal</td>
<td>Legal</td>
<td>3.14</td>
<td>Flexure</td>
</tr>
<tr>
<td>Permit</td>
<td>Routine</td>
<td>3.11</td>
<td>Flexure</td>
</tr>
</tbody>
</table>

4.2 LIVE LOAD DISTRIBUTION METHOD (LEVEL 1): AS-BUILT RATING WITH ACTUAL CONCRETE COMpressive STRENGTH

As part of the rating, the bridge records were reviewed first and the bridge data is incorporated into bridge rating to reflect the actual bridge conditions. The bridge records typically include: construction records, material testing records, load test data, traffic data, inspection history, and damage and rehabilitation history.

Contrary to design, actual material strength can be used instead of the specified nominal strength to rate the bridge, if the material test data is available. Frequently, the actual concrete compressive strengths are different and most likely are higher than the specified values. The as-built concrete strengths can be considered when rating an existing bridge, if the tested data are available.

The specified 28-day concrete compressive strength for the pre-stressed concrete girders was $f_{c'} = 5.0$ ksi based on the *Louisiana Standard Specifications for Roads and Bridges*. During construction, more than 240 concrete cylinders (6x12 inches) were tested for the compressive strengths as part of the quality control process. Testing time varied from 4 to 23 days after concrete placement. The 28-day concrete strengths were calculated using the method published on the HBRC journal (Metwally 2014). The average estimated 28 day compressive strength is 6.85 ksi, with a standard deviation of 0.45 ksi. The average compressive strength at 28 days is 27% higher than the specified compressive strength. The measured concrete strength histogram fits a Normal distribution (Figure 4-7 and Figure 4-8).
Following ACI 318, 5.3.2.1 (equations ACI 5-1 and ACI 5-2), $f_c' = 6.25$ ksi can be used as the actual nominal concrete strength based on a probability of 1-in-100 may fail. This strength is still 1.25 times higher than the specified compressive strength. Any strength gained after 28 days was not considered for this study.

$$f_{cr}' = f_c' + 1.34S_s$$  \hspace{1cm} (ACI 5-1)

$$f_{cr}' = 0.9f_c' + 2.33S_s$$  \hspace{1cm} (ACI 5-3)

where,

- $f_{cr}'$ = required average compressive strength of concrete, ksi
- $f_c'$ = specified compressive strength of concrete, ksi
- $S_s$ = sample standard deviation

The bridge was re-rated with the updated concrete compressive strength, and the new rating summary is shown in Table 4-4. With the increased concrete strength, the service limit state RF increases nearly 10%, the shear rating factor increases 8.5%, and the flexure rating remains the same.

<table>
<thead>
<tr>
<th>Live Load</th>
<th>Level</th>
<th>Rating Factor (RF)</th>
<th>Controls</th>
</tr>
</thead>
<tbody>
<tr>
<td>HL-93</td>
<td>Inventory</td>
<td>1.49</td>
<td>Service</td>
</tr>
<tr>
<td>HL-93</td>
<td>Inventory</td>
<td>1.54</td>
<td>Shear</td>
</tr>
<tr>
<td>HL-93</td>
<td>Inventory</td>
<td>1.96</td>
<td>Flexure</td>
</tr>
<tr>
<td>Legal</td>
<td>Legal</td>
<td>3.14</td>
<td>Flexure</td>
</tr>
<tr>
<td>Permit</td>
<td>Routine</td>
<td>3.11</td>
<td>Flexure</td>
</tr>
</tbody>
</table>
4.3 Refined Analysis Method (Level II): Finite Element Method (FEM)

The refined method is the next analysis level, which can more properly model the relative stiffness of all bridge components and provide more accurate load distributions. To improve the analysis accuracy, a finite element analysis can be used as the refined analysis method. This is the Level II load rating methodology. The refined FE analysis model can affect the live load distribution and the dynamic load effects. Generally, the live load distribution and the load effects will be reduced for certain elements, especially since they can affect the service limit state rating, which normally controls the prestressed concrete girder bridge load rating.

The bridge for this study was rated with a simplified finite element model using BrR. The bridge was modeled using plates and line elements. This modeling scheme has been shown to be relatively simple and accurate for slab-on-girder type of bridges (Barr et al. 2001). The bridge concrete deck was modeled using the four-node quadrilateral shell elements. Girders and diaphragms were modeled using beam elements located along the centroidal axes. The deck and beams were connected at the center of gravity with rigid links as shown in Figure 4-9. The rigid links ensure member compatibility and that the plane sections remained in plane. The diaphragms are placed at the node level for the longitudinal beam. The barriers were accounted for as dead load, and not part of the bridge structural element for this FE model.

![Figure 4-9 FEM modeling elements](image)

The FEM method was used here, mainly to improve the live load distribution factors. The FE model distribution factors are 10% less than the AASHTO equations for moment and 13% less for the shear as shown in Table 4-5.
Table 4-5 Interior girder live load distribution factors-FEM

<table>
<thead>
<tr>
<th></th>
<th>Moment</th>
<th></th>
<th>Shear</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Single</td>
<td>Multi-</td>
<td>Single</td>
<td>Multi-</td>
</tr>
<tr>
<td>lane</td>
<td>Lane</td>
<td>lane</td>
<td>Lane</td>
<td>Lane</td>
</tr>
<tr>
<td>0.422</td>
<td>0.582</td>
<td>0.505</td>
<td>0.593</td>
<td></td>
</tr>
</tbody>
</table>

Based on this model, the influence surfaces for the critical location were generated and are shown in Figure 4-10 for future live load rating and overload permit review.

This FEM model was used to rate the same group of design trucks, legal trucks, and permits trucks. The FEM rating summary is shown in Table 4-6. When comparing to level I analysis, the inventory and legal load rating increase about 10%.

Table 4-6 BrR FEM rating summary

<table>
<thead>
<tr>
<th>Live Load</th>
<th>Level</th>
<th>Rating Factor (RF)</th>
<th>Controls</th>
</tr>
</thead>
<tbody>
<tr>
<td>HL-93</td>
<td>Inventory</td>
<td>2.12</td>
<td>Service</td>
</tr>
<tr>
<td>HL-93</td>
<td>Inventory</td>
<td>1.78</td>
<td>Shear</td>
</tr>
<tr>
<td>HL-93</td>
<td>Inventory</td>
<td>2.18</td>
<td>Flexure</td>
</tr>
<tr>
<td>Legal</td>
<td>Legal</td>
<td>3.45</td>
<td>Flexure</td>
</tr>
<tr>
<td>Permit</td>
<td>Routine</td>
<td>3.41</td>
<td>Flexure</td>
</tr>
</tbody>
</table>
4.4 SUBSTRUCTURE RATING

Although rating engineers do not routinely rate substructures, sometimes they can govern the load capacity of the bridge. The concrete pile bent was modeled and rated in LEAP RCPIER as shown in Figure 4-11.

The MCFT equation method (The AASHTO LRFD 5.8.3.4.2) was selected for shear and torsion analysis. The design trucks were positioned transversely and longitudinally at 1-0” spacing to obtain the maximum substructure stress. The 42” wide x 27” high 3 ksi concrete pile cape is reinforced with 7-#8 bars at top and bottom as shown in Figure 4-12. The substructure ratings are summarized in Table 4-7.
Table 4-7 Substructure rating summary

<table>
<thead>
<tr>
<th></th>
<th>Capacity</th>
<th>Dead Load</th>
<th>Live Load</th>
<th>Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Inventory</td>
</tr>
<tr>
<td>Cap Moment (k-ft)</td>
<td>568.90</td>
<td>110.17</td>
<td>150.22</td>
<td>1.64</td>
</tr>
<tr>
<td>Cap Shear (kip)</td>
<td>354.60</td>
<td>89.78</td>
<td>98.90</td>
<td>1.40</td>
</tr>
<tr>
<td>Pile (kip)</td>
<td>500.00</td>
<td>89.78</td>
<td>109.75</td>
<td>2.02</td>
</tr>
</tbody>
</table>

The critical ratings for inventory and operating are $FR_{\text{inv}}=1.40$ and $RF_{\text{opr}}=1.82$ for shear strength at the girder locations. If a strut-and-tie model was used for the refined analysis, the rating factors could be increased. The cap inventory flexure rating is 1.64, which is higher than that for the superstructure girder rating.

The prestressed-precast concrete pile structural capacity is normally much higher than the driven pile geotechnical capacity. Therefore, the structural rating of piles is not considered here. The geotechnical capacity was rated as shown in above table. Driven piles are known to gain capacity with time (Skov and Denver, 1988, Tsai and Zhang 2008, Wang et al., 2010). The phenomenon is termed “pile freeze” or “setup”. A recent research project performed by the Louisiana Transportation Research Center shows that the capacity can increase up to 5 times from the end-of-driving capacity in a period of less than 6 months in clay soils (Hague et al, 2014). Similar behavior was also observed in sandy soils albeit in smaller magnitudes (Bullock 1999). Due to the setup phenomenon, the pile geotechnical capacities are not considered when rating a bridge.

In conclusion, the substructure ratings are normally not critical. Thus, this study concentrates on the superstructure rating hereafter.

4.5 SUMMARY

This load rating study started with the Level I rating - AASHTO live load distribution method as the base line rating. When using the actual constructed material strengths for the as-built load rating, bridge load rating factors for the service limit state increased 10%. Next, the finite element method rating (Level II) resulted in a 10% increase for the strength limit state.
CHAPTER 5 LOAD RATING THROUGH NONDESTRUCTIVE LOAD TESTING (LEVEL III)

The performance of most existing bridges does not match the prediction from the conventional theory due to the simplification of structural analysis, conservative design assumptions, and changes in bridge conditions. To reduce the difference between the theoretical bridge evaluation and the actual bridge behavior, nondestructive load testing can be used as another load rating methodology. A live load test was performed on the US61 bridge in this study, and the measured responses of the bridge were compared with the results of the finite element analysis to calibrate the bridge finite element model. Then, the updated FE model was used for load rating.

Most bridges perform more favourably than what conventional theory dictates. A load test may uncover extra bridge capacity that has been ignored in the conventional calculation. The AASHTO MBE defines the diagnostic load testing as “the observation and measurement of responses of a bridge subject to controlled and predetermined loading without causing changes in the elastic responses of the structure”. The extra capacity may be attributed to the following factors: unintended composite action, unintended continuity/fixity, participation of secondary members and non-structural members, and portion of load carried by deck. Certainly, some of the factors should not be fully depended upon for the load rating.

To provide a more realistic rating, a diagnostic load test can be used to improve the understanding of the behavior of the bridge and to identify and quantify the true reserved bridge capacity. The diagnostic tests will reduce the uncertainties related to material properties, boundary conditions, cross-section contributions, effectiveness of repair, and influence of damage and deteriorations (Lichtenstein, 1998). Typical load test procedures include establishing the initial FE model and field instrumentation plans, performing the live load test, calibrating the FE model, and rating the bridge using the updated FE model.

5.1 LOAD TESTING INTRODUCTION

The two types of non-destructive load tests commonly used to evaluate existing bridges are the diagnostic test and the proof test. The main difference between these two methods is the loading level. The primary objective of the diagnostic test is the assessment of the differences between the predicted and measured responses for subsequent use in the load rating of the bridge. The proof test, on the other hand, is used to verify the load carrying capacity of the bridge. Thus multiple levels of load are applied until the target load is achieved or the elastic limit of the bridge is reached.

A typical load test involves measuring strain, displacement, rotation, and dynamic characteristics at selected locations. If a bridge exhibits linear behavior, a diagnostic load test can be used to validate and update the analytical model by comparing the analytical data to the measured responses. The diagnostic load test, which was adopted for this study, includes a couple of major procedures. Prior to the test, the preliminary condition of the bridge was investigated and the results were comprised into a baseline bridge model. The baseline model used for comparison was the as-designed load rating. The model considered bridge condition and
deterioration based on a recent field inspection. Chapter 4 described this part of the study. Next, an instrumentation plan was developed for the diagnostic load test based on the understanding of the bridge behavior from the level I and level II studies. Selected critical locations had sensors installed to record data during the live load testing.

After the load test, data was evaluated to ensure the reliability of the load test results. Next, calibration of the FE model to match the bridge testing results produced an updated model that more accurately represented the actual load distribution and bridge behavior. The calibrated model was then used to update the load rating at strength limit states and service limit states.

5.2 INSTRUMENTATION PLAN

After the preliminary investigation, a typical 3-span unit was selected to be instrumented and load tested. Due to the symmetry of the bridge unit, only one exterior span and the interior span were instrumented. The goal of the instrumentation was to measure the live load response including the longitudinal flexure characteristics and the lateral load distribution.

A BDI 64 channel dynamic monitoring system was selected for the load testing and the long term monitoring. The selected 3-span unit was instrumented with 64 sensors, including 48 extended length temperature compensating concrete strain transducers (BDI ST-350) on girders, pile caps, and piles; 12 LVDT displacement sensors on piles; and 4 rotation sensors (tiltmeters) on the diaphragms. See Figure 5-1 for instrumentation plan and Figure 5-2 to 5-6 for cross section details (BDI Instrumentation Plan). The BDI strain gage can measure both tension and compression strain along its axis of orientation with two percent accuracy. All instrumentations were environmentally protected and have been in service for more than two years.

This study focuses on the strain measurements for the strength limit state or the service limit state checks. Strain gages were installed on all six girders at section locations A-A to E-E. The girder gages on span-1 (Figure 5-2) were located at 7'-0” from the pier walls and 7’-6” from the diaphragm faces; the gages on span-2 (Figure 5-3) were located at 7’-0” from the pier surface and 3’-0” from the diaphragm faces. The bottom gages were installed at the center of the girders, and the top gages were installed at 3” from the top of each girder. The girder gage used 24” gage extensions to ensure the accuracy (Figure 5-7). The extension increased the transducer gage length to allow the recording of an “averaged” strain value in the presence of cracks associated with the concrete structure.

The girders also had displacement sensors (Figure 5-8) and rotation sensors (Figure 5-9) attached. Vertical displacement sensors were installed at the mid-span on the girder bottom, and rotation sensors were attached to the side of the bottom flange near the end of the girder.

Additional instrumentation used at two of the substructures to measure the flexural responses in the bents for strain and vertical displacement of the piles. Pile gages were installed vertically and centered about each face of the pile (Figure 5-10). Bent gages were installed both transversely and 3” from the vertical edges of the bent face. Pile gages had 24” gage extensions, and bent gages had 15” extensions.
Figure 5-1 Instrumentation plan
Figure 5-2 Instrumentation plan – section A-A to C-C

Section A-A

Section B-B

Section C-C
Figure 5-3 Instrumentation plan – section D-D and E-E
Figure 5-4 Instrumentation plan – substructure at bent-1

Figure 5-5 Instrumentation plan – substructure at bent-2
Figure 5-6 Instrumentation plan - sections at bent
Figure 5-7 Surface mounted extended strain transducers on girder

Figure 5-8 Displacement sensors near mid-span on girder
Figure 5-9 Rotation sensor (tiltmeter) near bent

Figure 5-10 Pile displacement measurement instruments
All of the sensors were connected to the CR-3000 data loggers in the data logger cabinet as shown in Figure 5-11 (a). Data loggers were set up to interface with a personal computer using the Campbell Scientific, Inc. LoggerNet software. LoggerNet was used to communicate with the logger remotely through a wireless modem or directly with a RS-232 serial connection. The SHM system also consisted of a solar power unit, an 8A31DT-DEKA 12V battery, and a digital camera. The autoclicker (Figure 5-11 (b)), a device that electronically counts wheel revolutions, was mounted on the test vehicle to identify the vehicle position. Figure 5-11 (c) and (d) gives illustrations of the sensors.

Figure 5-11 BDI dynamic monitoring system: (a) Data logger (b) Autoclicker (c) Strain transducer (d) Tiltmeter
5.3 NONDESTRUCTIVE LOAD TESTING

A diagnostic load test was performed while all the bridge lanes were closed to traffic. In order to properly measure the bridge physical behavior, diagnostic testing load shall be sufficiently high. Differing from proof testing, the load shall also be limited to not causing nonlinear behavior.

The live load test was conducted with a loaded three-axle snooper. The selected testing truck was weighed prior to the test with a gross weight of 58,620 lbs. The test vehicle configuration and the weight distribution are provided in Table 5-1 and Figure 5-12.

Table 5-1 Loading vehicle information

<table>
<thead>
<tr>
<th>Vehicle Type</th>
<th>Snooper Truck</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axle Number</td>
<td>Axle 1</td>
</tr>
<tr>
<td>Axle Weight (LBS)</td>
<td>13,040</td>
</tr>
<tr>
<td>Axle Spaces (FT)</td>
<td>20.67</td>
</tr>
<tr>
<td>Wheel Spaces (FT)</td>
<td>7.08</td>
</tr>
</tbody>
</table>

Figure 5-12 Loading vehicle configuration

To obtain the critical load effects, four load paths were pre-defined along the bridge as shown in Figure 5-13. The reference location was set at the inside edge of the barrier of the first test span end. The paths were 12’, 19.8’, 26.8’, and 38.9’ from the reference point to the truck driver side. To ensure quality, two tests were performed for each path, and one set of test data for each truck path was selected for model calibration. The speed of the loading vehicle was set at less than 5 mph to limit the dynamic amplification of the vehicle. The truck BDI AutoClicker was processed so that the corresponding strain and displacement data could be presented as a function of vehicle position.
5.4 DATA EVALUATION AND ANALYSIS

During the test, a set of the strain history data for each sensor was recorded at 40 Hz using BDI-STS. The first task after the load test was the preliminary investigation of the data and selection. This investigation included checking the reproducibility of test data and checking the structure for beam elastic behavior. For all of the strain gages with extensions, the secondary gage factors were also needed for correction.

Plots of strain history verses truck position were created using BDI-WinGRF software as shown in Figure 5-14 and Figure 5-15. The strain is expressed in microstrain, and positive values correspond to tensions. The displacement is measured in inches. The driver side front axle is the reference location of the vehicle.

First, two identical tests were performed for each path to verify the reproducibility of the data as shown in Figure 5-14. The responses from these two tests were very comparable for both girders, indicating the consistency of the load tests. Additionally, all strains and displacements returned to zero after each load test indicating that the structure was acting in a linear-elastic manner as assumed. Then, the better representative set of test data for each truck path was selected for finite-element model calibration. Therefore, four file records were selected for the FE model calibration.

Figure 5-13 Load testing truck traveling paths

Figure 5-15 shows the girder mid-span strains under the same truck path. It can be observed that the structure was able to distribute the load laterally across the section, and the strain histories are similar in shape between girders.

The neutral axis locations are another set of data that can be used for section composition action performance and the data quality control. The neutral axis occurs at the axis of bending for the section where the stress is zero. If only vertical loads are applied to the beams, the neutral
axis should coincide with the geometric centroid for the beam. Some girder strains were measured using two strain gages, one attached to the bottom flange and the other one attached to the side of top flanges, 3 inches from the top.

Figure 5-14 Mid-span displacements at span-2

Figure 5-15 Mid-span strains at span-2
The neutral axis of sections corresponding to the paired gages was calculated based on the linear distribution, as shown in Equation (5.1).

\[
\bar{y} = \frac{\varepsilon_B D_{gage}}{\varepsilon_B - \varepsilon_T}
\]  

(5.1)

Where,
- \(\varepsilon_T\) = Top flange gage strain (με)
- \(\varepsilon_B\) = Bottom flange gage strain (με)
- \(\bar{y}\) = Neutral axis depth from bottom gage (in)
- \(D_{gage}\) = Distance between the top and bottom gages (inch) = 42 inch for this bridge

Then, plots of the neutral axis depth history of the interior and exterior girders were created. The average neutral axis of the two paths is 36.02 in. and 37.16 in. from the bottom of girder with a standard deviation of 0.464 in. and 1.035 in. for interior girder at span-1 (Figure 5-16) and span-2 (Figure 5-17), respectively. These results are consistent with the calculated neutral axis located at 35.47 in. (Table 3-1), which are shown as squared symbols in Figure 5-17. The consistency indicates the composite action for the prestressed concrete girder. The average neutral axis is 38.10 in. and 37.35 in. with a standard deviation of 1.155 in. and 0.842 in. for the exterior girder at span-1 (Figure 6-19) and span-2 (Figure 5-19), respectively. These results are higher than the calculated exterior neutral axis (35.05 in.). The raised axis location is caused by the stiffness of the concrete barrier. Although the barriers were not designed as a structural element for the vertical loads, it did increase the stiffness and cross section of the exterior girders.

Figure 5-16 Interior girder mid-span neutral axis history at span-1
Figure 5-17 Interior girder mid-span neutral axis history at span-2

Figure 5-18 Exterior girder mid-span neutral axis history at span-1
Figure 5-19 Exterior girder mid-span neutral axis history at span-2

5.5 FINITE ELEMENT MODEL CALIBRATION (OPTIMIZATION)

The most critical step in the interpretation of load-testing results is model creation and calibration. In real world scenarios, structural member properties may differ from their specified values used in design due to issues during fabrication, construction, destruction, as well as deterioration after construction among other unconsidered factors in design. The process of model calibration is the updating and adjusting of the analytical models to match the observed bridge behavior.

Due to the large amount of information provided in both the analytical model and the bridge instrumentation responses to live loads, it is virtually impossible to have a 100% match in an analytical model to the data collected from bridge instrumentation. The optimization approach defines the objective functions that quantify the deviations between the analytical and experimental results, and minimizes the discrepancy by adjusting the assumed parameters used in the analytical model (Kim and Park 2004). Since only strength and service limit states are considered in the load rating for this study, the main objective for this calibration is to optimize the match of strains and deflections. The parameters that can be calibrated include stiffness cross-section area, elastic modulus, moment of inertia, and boundary condition, etc. Among these, boundary condition and flexural stiffness of concrete (EI) are the most important parameters (Kwasniewski et al. 2000).
The field load testing and analysis series software BDI-WinGEN, BDI-WinSAC, and BDI-WinGRF developed by Bridge Diagnostics, Inc. were adopted for this study. This software generates the finite element model, performs structural analysis, and performs model calibration. The BDI-WinSAC (Structural Analysis and Correlation) program is a general-purpose finite element analysis program based on stiffness matrix methods and is limited to linear-elastic models. It has a feature that compare the computed strains to the measured values and automatically identify property values to improve the correlation between the computed and measured responses. Altering various material properties used in the mathematical model to improve the agreement between the model and observations can reduce discrepancies between the measured and computed strains. BDI-WinSAC implements an identification routine, which automatically varies user specified material properties and converges on the values of the properties that result in the least amount of deviation between the analytical model and the field observed strains.

The concept of the Bridge Diagnostics, Inc. (BDI) software calibration flowchart is graphically illustrated in Figure 5-20.

![Figure 5-20 Illustration of BDI FEM model calibration approach](image-url)

After the load test, the first step was to develop the bridge initial FE model using the BDI software WinGEN (Figure 5-21). The finite elements model consists of frame elements for the girders, diaphragms, barriers, and pile caps; shell elements for the deck; and elastic springs for the piles. The supports were modeled using spring elements. Gage locations were modeled to match the physical field locations.
The next step was to calibrate the FE model to achieve the best correlation and smallest error between the theoretical and measured bridge strains and deflections. The FE model created in BDI-WinGEN computed the theoretical strains and deflections. Then, the BDI-WinSAC program was used to compare the computed strains at the gage locations with the measured strain values and automatically identify parameters that can improve the correlation between the computed and measured responses.

The first step in performing calibration is to identify the relevant parameters that have the maximum effects on the computed strains and deflections. In general, the following three properties were identified as the most important parameters for their respective bridge elements: modulus of elasticity or thickness (E or t) for the plate elements, modulus of elasticity (E) or moment of inertia (I) for the beam elements, and rotational stiffness at the supports.

In order to match commonly used statistical terminology, the deviation between the modeling result and the observed data is termed “error”. The error minimization process is based primarily on the least squares approach. The model calibration is completed through numerical and visual comparisons between the measured strains and those predicted with the finite element model to obtain the best fit. During the process, four measurements of errors are quantified to evaluate the quality of the fit. These error measurements are described below.

- Absolute Error ($E_{\text{absolute}}$): Error computed from the sum of the absolute response differences between the model results and measured values at the locations of sensors. This factor can be used to determine the relative improvement during the model calibration.
- Percent Error ($E_{\text{percent}}$): Error calculated to provide a better qualitative measurement of accuracy. The terms are squared so that the errors with different signs would not cancel each other. A model with acceptable accuracy will usually have a percent error of less than 10%.
- Scale Error ($E_{\text{scale}}$): Error parameter that is similar to the percent error except that it is based on the maximum error from each gage divided by the maximum strain value from each gage.
- Correlation Coefficient ($E_{\text{correlation}}$): A measure of the linearity between the measured and computed data. This factor determines how well shapes of the computed response histories match. A good model will generally have $E_{\text{correlation}} > 0.9$.

These error parameter calculations are shown in Equations (5.2) to (5.5). After the optimization, which is based on a live load test, a refined model is established for further bridge rating.

$$ E_{\text{absolute}} = \sum |\varepsilon_m - \varepsilon_c| $$ \hspace{1cm} (5.2)

$$ E_{\text{percent}} = \frac{\sum ((\varepsilon_m - \varepsilon_c)^2)}{\sum (\varepsilon_m^2)} \% $$ \hspace{1cm} (5.3)

$$ E_{\text{scale}} = \frac{\sum (\text{max}|\varepsilon_m - \varepsilon_c|)_{\text{gage}}}{\sum (\text{max}|\varepsilon_m|)_{\text{gage}}} $$ \hspace{1cm} (5.4)

$$ E_{\text{correlation}} = \frac{\sum (\varepsilon_m - \bar{\varepsilon}_m)(\varepsilon_c - \bar{\varepsilon}_c)}{\sqrt{\sum (\varepsilon_m - \bar{\varepsilon}_m)^2(\varepsilon_c - \bar{\varepsilon}_c)^2}} $$ \hspace{1cm} (5.5)

Where,

- $\varepsilon_m =$ measured strain
- $\varepsilon_c =$ calculated strain
- $\bar{\varepsilon}_m =$ average measured strain
- $\bar{\varepsilon}_c =$ average calculated strain

Three hundred thirty-six (336) load cases were recorded from the load tests, and 14,112 points were compared for the model calibration. The following parameters were updated during the optimization:

- Girder stiffness (EI) to consider the influence of the pre-stressing steel, dimension deviation, and cracks, especially at the negative moment region over the interior pier.
- Deck stiffness (EI) to consider the deck thickness variation, deck reinforcement, and the effect of cracks, especially at continuity diaphragm (E).
- Support stiffness $F_z$ (k/in) to include the performance of the bearing pad, pile, and soil stiffness.

Table 5-2 shows the initial and final adjusted values of the optimized parameters.
Table 5-2 FEM calibration parameters

<table>
<thead>
<tr>
<th>FEM Parameter</th>
<th>As-designed Value</th>
<th>Final Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modulus of Elasticity E (ksi) – interior and exterior girders</td>
<td>4,792</td>
<td>5,816</td>
</tr>
<tr>
<td>Moment of Inertia Ix (in⁴) - Exterior Girder</td>
<td>381,944</td>
<td>624,200</td>
</tr>
<tr>
<td>Modulus of Elasticity E (ksi) - Exterior Girder continuity</td>
<td>3,256</td>
<td>830</td>
</tr>
<tr>
<td>Modulus of Elasticity E (ksi) - Interior girder continuity</td>
<td>3,256</td>
<td>486</td>
</tr>
<tr>
<td>Modulus of Elasticity E (ksi) - Deck</td>
<td>3,256</td>
<td>2,100</td>
</tr>
<tr>
<td>Modulus of elasticity E (ksi) - deck at continuity diaphragm</td>
<td>3,256</td>
<td>300</td>
</tr>
<tr>
<td>Support Stiffness Fz (kip/in)</td>
<td>8,000</td>
<td>1,500</td>
</tr>
</tbody>
</table>

The final adjusted concrete elastic moduli or moment of inertia do not represent the actual properties of the concrete and also include the effects of the combination of factors, such as cracks, boundary conditions, and other factors that were not considered in the model. Therefore, the adjusted flexure stiffness (EI) is a better indicator for use in interpreting the bridge behavior.

As shown in Table 5-2, the interior and exterior girders were stiffer than initially assumed due to the higher concrete strength. The effective flexure stiffness (EI) was higher than initially assumed because the combination of the prestressed and mild steel effects. Additionally, the increase of the stiffness of the exterior girders was caused by the influence of the concrete barrier’s stiffness contribution to the structure. Conversely, the continuity effectiveness represented by the elastic moduli of the girder and deck were substantially smaller than assumed. This is likely the result of deck cracking at the bent location and the short element length (0.9 ft. long). Since the deck and diaphragms are the main factors affecting the load lateral distribution, the reduced deck moduli reflect the existence of cracks in the concrete.

Table 5-3 provides details of the FEM and model calibration. These details illustrate the goodness of the model fitting in comparison to the actual bridge. The model has a correlation coefficient of 98.46%, which indicates an excellent fit.

Table 5-3 FEM calibration details

<table>
<thead>
<tr>
<th>BDI-WinGen Calibrated Model Details</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of nodal points</td>
<td>1,817</td>
</tr>
<tr>
<td>Number of elements</td>
<td>2,620</td>
</tr>
<tr>
<td>Max degrees of freedom/node</td>
<td>6</td>
</tr>
<tr>
<td>Max number of nodes/element</td>
<td>4</td>
</tr>
<tr>
<td>Number of load cases</td>
<td>337</td>
</tr>
<tr>
<td>Number of instrumentation locations used</td>
<td>42</td>
</tr>
<tr>
<td>Percent error (E_percent)</td>
<td>3.10%</td>
</tr>
<tr>
<td>Scale error (E_scale)</td>
<td>1.30</td>
</tr>
<tr>
<td>Correlation coefficient (E_correlation)</td>
<td>0.9846</td>
</tr>
</tbody>
</table>
After satisfying the initial analysis results, BDI-WinGRF was used for graphing, data processing, and further comparison. With BDI-WinGRF, the raw test data can be viewed graphically and compared with the subsequent analysis results.

The mid-span strain response comparisons of the FE model and recorded observations are shown in Figure 5-22 and Figure 5-23 for span-1 and span-2, respectively. The closeness of the fit shows the reproducibility of the model under four different traveling paths. The figures also show the computed data (A1 to A4) and measured data at the sensor location for each path. The fit as indicated in the figures gives confidence that the calibrated model is adequate for use in future load rating and monitoring.

Figure 5-22 Mid-span strain comparison at span-1: girder 3

Figure 5-23 Mid-span strain comparison at span-2: girder 3
5.6 LOAD RATING

After the model calibration, an LRFR load rating was performed using the calibrated FE model for design trucks, legal trucks, and permit trucks. The dead load effects and element capacities were based on the AASHTOWare BrR model, which considers the construction sequence (composite versus non-composite effects). The critical flexure location is at girder 3, span-1 at about 40% of the span length. The rating summary is shown in Table 5-4.

<table>
<thead>
<tr>
<th>Live Load</th>
<th>Level</th>
<th>Rating Factor (RF)</th>
<th>Controls</th>
</tr>
</thead>
<tbody>
<tr>
<td>HL-93</td>
<td>Inventory</td>
<td>2.02</td>
<td>Service</td>
</tr>
<tr>
<td>HL-93</td>
<td>Inventory</td>
<td>3.17</td>
<td>Shear</td>
</tr>
<tr>
<td>HL-93</td>
<td>Inventory</td>
<td>2.78</td>
<td>Flexure</td>
</tr>
<tr>
<td>Legal</td>
<td>Legal</td>
<td>4.48</td>
<td>Flexure</td>
</tr>
<tr>
<td>Permit</td>
<td>Routine</td>
<td>4.43</td>
<td>Flexure</td>
</tr>
</tbody>
</table>

5.7 SUMMARY

In comparison to the theoretical rating presented in Chapter 4, the calibrated finite element model benefited from the NDT and the whole bridge behavior resulted in a more accurate bridge rating. The NDT rating results not only included the bridge system behavior beyond the design predictions, but also contained the construction and deterioration factors.

In taking advantage of the actual bridge 3D system behavior, the NDT rating method led to a load rating factor 40% higher than the as-designed (Level I) rating. The load rating factors from NDT are 25% higher than the FE method (Level II) as well, even though the FEM has a high degree of accuracy.

This result confirmed that the traditional live load distribution method is conservative for load rating and overload truck review for certain type of bridges.

The calibrated model and instrumentation have been used for long-term structural health monitoring for bridge management and for continuous load rating.
CHAPTER 6    LOAD RATING BASED ON WIM (LEVEL IV)

In bridge design and evaluation, the AASHTO code uses notional truck configurations as screening vehicles. Generally speaking, truck loads are strongly influenced by traffic volumes, axle weights, axle configurations, and local law enforcement effort. Traffic conditions are highly site specific and are the most variable element among the factors affecting load rating. To reduce the variability, the use of Weigh-In-Motion systems is one of the most utilized methods to collect long-term unbiased traffic data including volumes, classifications, traffic patterns, and truck configurations (gross weight, length, axle weight, axle spacing). In addition to the current traffic condition, the long-term WIM data can be used to project future site-specific live load model for the typical load rating period of two to five years.

Considering the maturity of the technology, cost of the WIM system, and close proximity of the bridge instrumentation, a pavement WIM system was selected over a bridge WIM. There are four commonly used high speed WIM systems: piezoelectric, piezoquartz, bending plate, and load cell with various accuracies and costs associated with each of these systems. After comparing the WIM systems, a piezoelectric system provided by International Road Dynamics Inc. (IRD) was chosen for this project.

As with all electronic signals, the WIM or bridge instrumentation may produce readings that are not consistent with the loads the bridge experiences due to environmental and other factors. Therefore, in addition to the WIM, a camera was also used at the US61 bridge site to confirm truck configurations. The camera automatically takes pictures when the strain response from the bridge exceeds a predetermined criterion. These photographs are then used to verify the readings from the WIM system as well as the strain and deformation readings from the bridge instrumentation.

6.1    WEIGH-IN-MOTION SYSTEM

The selected WIM system, including the IRD iAnalyze software, was provided by IRD. The piezoelectric sensors (Brass Linguini) WIM system is one of the most commonly used in the US. The piezoelectric sensors consist of a copper strand surrounded by piezoelectric material covered by a copper sheath (Figure 6-1). The sensors, when installed into a slot in the pavement, detect the changes in the deformation induced by tire loads on the pavement’s surface. This WIM system was selected due to the ease of installation, monitoring and removal, and minimal interruption to traffic. A properly installed and calibrated system can provide the gross vehicle weight within 10% of the actual vehicle weight.

The sensors were installed directly into a slot of the road as shown in Figure 6-2. To improve the accuracy, two piezoelectric sensors in series were placed in each lane, and the average readings were used for analysis. The WIM system was calibrated every 6 months to ensure proper operating. The IRD WIM was also synchronized with a bridge SHM system through a global positioning system (GPS) to validate the truck effects. The GPS provided consistent time stamps for the WIM and bridge instrumentation.
The WIM sensors were installed in both the driving lane and the passing lane of the two-lane, one-way state highway. The traffic data, collected continuously for over two years, captured over 200,000 trucks. Note that only one year’s worth of the data was used for this study due to the time required for the analysis.
Although the WIM system is considered as an advanced technique, as explained previously, many factors can produce unreasonable observations due to environmental effects and system limitations. Therefore, the first step is to clean up the data through data examination and filtering to remove the unreliable data. For example, slow moving traffic, and trucks with very large axle spacing can cause incorrect truck configurations.

Raw WIM data was scrubbed to ensure the quality of the data. The data scrubbing procedure was based on the WIM protocols developed in the NCHRP project 12-76 entitled “Protocol For Collecting and Using Traffic Data In Bridge Design” (Sivakumar et al., 2011) and Louisiana Regulations for Vehicles and Loads (LADOTD, 2013). To maintain the quality of WIM data, the unreliable data was filtered and eliminated using the following criteria:

- Records where speed < 10 mph or speed > 100 mph
- Records where truck length > 120 ft
- Records where total number of axles > 11
- Records where total number of axles < 3
- Records where the sum of axle spacing is greater than the length of the truck
- Records where gross vehicle weight (GVW) < 12 kips
- Records where an individual axle is > 70 kips
- Records where the steer axle < 6 kips
- Records where any axle spacing < 3.4 ft
- Records where any axle < 2 kips
- Records which have GVW +/- sum of the axle weights by more than 10% (this may indicate the axle records provided may not be complete or accurate)

This procedure filtered out calibration errors from the measured WIM histogram of the gross weight. The scrubbing removed around 20% of the records. A sampling of the eliminated data was also checked to make sure that real trucks, especially heavy ones, were not removed from the dataset.

The data recorded includes travel lane, time, speed, FHWA truck classifications, gross vehicle weight, number of axles, axle weights, and axle spacing. A sample of the WIM data is shown in Table 6-1. TRB (1990a, 1990b, 1997 and 2002) has published many truck studies. The FHWA truck classifications can be found in appendix A.1. The accuracy of the records is as follows: timestamp is to the hundredth of a second; speed is to the one tenth of a mph; weight is to the one tenth of a kip, and length is to one tenth of a foot.

The accurate timestamp (1/100 of a second) is important for estimating truck multiple presence probabilities. These timestamps allow the determination of headway separation of trucks in adjacent lanes or in the same lane.

The scrubbed dataset was then imported to SAS JMP for quality control check and statistical analysis.
Table 6-1 Recorded sample WIM data

<table>
<thead>
<tr>
<th>Timestamp</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>11/17/2013 5:50:34.47 AM</td>
<td>Recorded sample WIM data</td>
</tr>
<tr>
<td>Record</td>
<td>60352 Record number</td>
</tr>
<tr>
<td>Lane Number</td>
<td>4 Travel Lane: 4 – driving lane, 3- passing lane</td>
</tr>
<tr>
<td>Month</td>
<td>11</td>
</tr>
<tr>
<td>Day</td>
<td>17</td>
</tr>
<tr>
<td>Year</td>
<td>13</td>
</tr>
<tr>
<td>Hour</td>
<td>5</td>
</tr>
<tr>
<td>Minute</td>
<td>48</td>
</tr>
<tr>
<td>Second</td>
<td>37</td>
</tr>
<tr>
<td>HunSec</td>
<td>47</td>
</tr>
<tr>
<td>VehNum</td>
<td>58446 Vehicle Number</td>
</tr>
<tr>
<td>Number Of Axles</td>
<td>5</td>
</tr>
<tr>
<td>Class</td>
<td>9 FHWA truck class</td>
</tr>
<tr>
<td>Gross Weight</td>
<td>87000 GVW in pounds</td>
</tr>
<tr>
<td>Length</td>
<td>60.1 Total length (ft)</td>
</tr>
<tr>
<td>Speed</td>
<td>55.9 Speed in mph</td>
</tr>
<tr>
<td>Spc_1-2</td>
<td>13.8 Axle spacing 1-2 (ft)</td>
</tr>
<tr>
<td>Spc_2-3</td>
<td>4.4 Axle spacing 2-3 (ft)</td>
</tr>
<tr>
<td>Spc_3-4</td>
<td>28.1 Axle spacing 3-4 (ft)</td>
</tr>
<tr>
<td>Spc_4-5</td>
<td>4.1 Axle spacing 4-5 (ft)</td>
</tr>
<tr>
<td>Spc_5-6</td>
<td>0 Axle spacing 5-6 (ft)</td>
</tr>
<tr>
<td>Spc_6-7</td>
<td>0 Axle spacing 6-7 (ft)</td>
</tr>
<tr>
<td>Spc_7-8</td>
<td>0 Axle spacing 7-8 (ft)</td>
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<tr>
<td>Spc_8-9</td>
<td>0 Axle spacing 8-9 (ft)</td>
</tr>
<tr>
<td>Spc_total</td>
<td>50.4 Total spacing (ft)</td>
</tr>
<tr>
<td>Wt_1</td>
<td>9100 Axle weight axle 1 (lbs)</td>
</tr>
<tr>
<td>Wt_2</td>
<td>19600 Axle weight axle 2 (lbs)</td>
</tr>
<tr>
<td>Wt_3</td>
<td>18700 Axle weight axle 3 (lbs)</td>
</tr>
<tr>
<td>Wt_4</td>
<td>19300 Axle weight axle 4 (lbs)</td>
</tr>
<tr>
<td>Wt_5</td>
<td>20300 Axle weight axle 5 (lbs)</td>
</tr>
<tr>
<td>Wt_6</td>
<td>0 Axle weight axle 6 (lbs)</td>
</tr>
<tr>
<td>Wt_7</td>
<td>0 Axle weight axle 7 (lbs)</td>
</tr>
<tr>
<td>Wt_8</td>
<td>0 Axle weight axle 8 (lbs)</td>
</tr>
<tr>
<td>Wt_9</td>
<td>0 Axle weight axle 9 (lbs)</td>
</tr>
</tbody>
</table>
6.3 TRAFFIC CHARACTERISTIC

6.3.1 Traffic Pattern

An observation from the WIM data is that over 80% of the truck traffic was traveling on the driving lane (lane 4, sometime called “slow lane”), and less than 20% was on passing lane (lane 3), as shown in (Figure 6-3). The distributions of different number of truck axles are shown in Figure 6-4. Evidently, the most common vehicle configuration was the 5-axle truck with the tractor semi-trailer. The next most common vehicle was the four-axle truck.

The mean gross vehicle weight (GVW) statistics versus the truck axle configurations separated by lane designation are shown in Figure 6-5. Most of the heavier trucks travel on the driving lane, especially the heavy overload trucks. The range bar presents the GVW range.

![Figure 6-3 Traveling Lane Distribution (Lane 3 - Passing Lane; Lane 4 - Driving lane)](image)
Figure 6-4 Traffic distribution – axle distribution

Figure 6-5 Mean GVW verses axle configurations for Lane 3 and Lane 4
6.3.2 Site-specific Truck Model

The truck GVW statistics separated by different number of axles are shown in Table 6-2. The statistics include mean GVW, minimum GVW, maximum GVW, median GVW, and standard deviations. The coefficient of variation and quantile 95 are also shown here. The results provide a view of truck types operating on this bridge.

Obviously, trucks with 3 to 8 axles represented the majority of trucks since there were only 26 trucks with 9 or more axles. Thus, the chance for the side-by-side presence of the same vehicles with 9 or more axles was very unlikely. It is worth noting that the maximum GVW of the most popular 5-axle trucks is 158.0 kips, which is much higher than the Louisiana legal vehicle gross weight limit of 88 kips for non-interstate routes.

Table 6-2 GVW (kips) statistics by truck axles

<table>
<thead>
<tr>
<th>Axles</th>
<th>N</th>
<th>% of Total</th>
<th>Mean</th>
<th>Min</th>
<th>Max</th>
<th>Std Dev</th>
<th>CV*</th>
<th>Median</th>
<th>Quantiles 95*</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>2,153</td>
<td>1.48%</td>
<td>31.8</td>
<td>12.0</td>
<td>64.7</td>
<td>7.7</td>
<td>24.4</td>
<td>31.1</td>
<td>45.6</td>
</tr>
<tr>
<td>4</td>
<td>13,431</td>
<td>13.27%</td>
<td>45.7</td>
<td>12.0</td>
<td>157.1</td>
<td>17.7</td>
<td>38.6</td>
<td>41.9</td>
<td>76.3</td>
</tr>
<tr>
<td>5</td>
<td>59,099</td>
<td>76.62%</td>
<td>60.0</td>
<td>13.9</td>
<td>158.0</td>
<td>23.7</td>
<td>39.4</td>
<td>52.5</td>
<td>97.6</td>
</tr>
<tr>
<td>6</td>
<td>4,364</td>
<td>7.77%</td>
<td>82.4</td>
<td>19.4</td>
<td>197.1</td>
<td>25.9</td>
<td>31.5</td>
<td>87.5</td>
<td>121.6</td>
</tr>
<tr>
<td>7</td>
<td>220</td>
<td>0.45%</td>
<td>95.6</td>
<td>39.6</td>
<td>183.5</td>
<td>34.1</td>
<td>35.7</td>
<td>87.9</td>
<td>153.6</td>
</tr>
<tr>
<td>8</td>
<td>105</td>
<td>0.33%</td>
<td>144.3</td>
<td>43.2</td>
<td>205.9</td>
<td>35.6</td>
<td>24.7</td>
<td>151.4</td>
<td>188.8</td>
</tr>
<tr>
<td>9</td>
<td>25</td>
<td>0.09%</td>
<td>159.7</td>
<td>54.6</td>
<td>229.4</td>
<td>45.3</td>
<td>28.4</td>
<td>171.3</td>
<td>228.8</td>
</tr>
<tr>
<td>11</td>
<td>1</td>
<td>0.00%</td>
<td>183.2</td>
<td>183.2</td>
<td>183.2</td>
<td></td>
<td></td>
<td>183.2</td>
<td>183.2</td>
</tr>
</tbody>
</table>

*CV is coefficient of variation and Quantiles 95 is the 95th percentile value

Once the WIM data is checked, the next step is to establish a suit of site-specific truckload models based on the scrubbed data. The truckload models are established by grouping typical axle spacing and the distribution of the gross weight of each axle as shown in Table 6-3. The site-specific trucks are illustrated in Figure 6-6. The average axle spacing is used to derive the typical truck configurations.

The heaviest trucks that are at the upper tail of the truck gross weight histograms govern bridge load ratings. The average GVW of the top 20% trucks represents mostly fully loaded trucks, and the top 5% of the trucks reflects the more severe overloads. The results for the top 5% of trucks having 3 to 11 axles provide a rough view of truck types typically operating at the site. The top 5% average truck weight indicates that these routes are exposed to much heavier trucks than the state legal vehicle limits. These site-specific representative trucks are shown in Figure 6-6. These site-specific trucks were used for load rating later. There was only one truck recorded with more than 10-axles, no site-specific truck was developed for these trucks.
Table 6-3 Site-specific truck GVW statistics and configurations

<table>
<thead>
<tr>
<th>TRUCK CONFIGURATIONS</th>
<th>NO. OF AXLES</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
</tr>
</thead>
<tbody>
<tr>
<td>AVERAGE GVW (KIPS)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>100%</td>
<td>31.76</td>
<td>45.74</td>
<td>60.04</td>
<td>82.40</td>
<td>95.56</td>
<td>144.26</td>
<td>159.73</td>
<td></td>
</tr>
<tr>
<td>TOP 20%</td>
<td>43.10</td>
<td>72.44</td>
<td>94.56</td>
<td>115.48</td>
<td>145.83</td>
<td>183.65</td>
<td>212.43</td>
<td></td>
</tr>
<tr>
<td>TOP 5%</td>
<td>49.39</td>
<td>84.81</td>
<td>104.25</td>
<td>132.33</td>
<td>159.84</td>
<td>196.96</td>
<td>228.45</td>
<td></td>
</tr>
<tr>
<td>AXLE WEIGHT (PERCENT)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wt_1</td>
<td>27%</td>
<td>21%</td>
<td>12%</td>
<td>11%</td>
<td>10%</td>
<td>9%</td>
<td>6%</td>
<td></td>
</tr>
<tr>
<td>Wt_2</td>
<td>40%</td>
<td>21%</td>
<td>21%</td>
<td>19%</td>
<td>17%</td>
<td>9%</td>
<td>8%</td>
<td></td>
</tr>
<tr>
<td>Wt_3</td>
<td>34%</td>
<td>29%</td>
<td>21%</td>
<td>19%</td>
<td>17%</td>
<td>15%</td>
<td>12%</td>
<td></td>
</tr>
<tr>
<td>Wt_4</td>
<td>29%</td>
<td>23%</td>
<td>17%</td>
<td>14%</td>
<td>15%</td>
<td>12%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wt_5</td>
<td>33%</td>
<td>23%</td>
<td>17%</td>
<td>14%</td>
<td>13%</td>
<td>13%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wt_6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>Wt_7</td>
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<td></td>
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<td></td>
<td></td>
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<td></td>
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<tr>
<td>Wt_8</td>
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<td>Wt_9</td>
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<tr>
<td>AXLE SPACING (FEET)</td>
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<td></td>
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<td>Spc_1-2</td>
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<td>16</td>
<td>16</td>
<td>16</td>
<td>16</td>
<td>16</td>
<td>12</td>
<td></td>
</tr>
<tr>
<td>Spc_2-3</td>
<td>22</td>
<td>4</td>
<td>4</td>
<td>4</td>
<td>4</td>
<td>4</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>Spc_3-4</td>
<td>4</td>
<td>28</td>
<td>28</td>
<td>26</td>
<td>4</td>
<td>8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Spc_4-5</td>
<td>4</td>
<td>4</td>
<td>12</td>
<td>34</td>
<td>8</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Spc_5-6</td>
<td>4</td>
<td>8</td>
<td>4</td>
<td>20</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Spc_6-7</td>
<td>4</td>
<td>4</td>
<td>8</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Spc_7-8</td>
<td>4</td>
<td>8</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Spc_8-9</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total (AL)</td>
<td>34</td>
<td>34</td>
<td>52</td>
<td>56</td>
<td>70</td>
<td>70</td>
<td>76</td>
<td></td>
</tr>
</tbody>
</table>

79
Figure 6-6 Site-specific representative truck configurations
6.3.3 Traffic Stream Variability

6.3.3.1 Seasonal Variation

Traffic can be seasonal and varies with time. The one-year long collection period of this study can track this variation and reduce the seasonal bias. The observed low GVWs occurred in April, May, and September (Figure 6-7), while the high truck traffic volumes occurred in the second half of the years from June to December (Figure 6-8). Note that, the data of the entire month of March and a portion of February and April were not collected due to issues with the backup battery of the data acquisition system. Aside from the missing data from February to April, the traffic is fairly consistent throughout the year. Even with the maximum monthly average GVW observed in February (61.5 kips), the difference between the average (58.32 kips) and monthly maximum GVWs is less than 5%.

Figure 6-7 Monthly average GVW
The monthly maximum truck weights are relatively consistent throughout the year, with somewhat heavier weights observed in November (Figure 6-9), which coincides with the typical period that industrial plants perform maintenance. The GVW cumulative distributions (Figure 6-10) show very little deviation in the GVW distributions from month to month. Therefore, the traffic pattern was consistent throughout the year. It can be concluded that it would be reasonable to use short period data to predict long period traffic pattern.
The average hourly truck traffic volumes vary from 761 at 1:00 AM to 7782 at 10:00 AM (Figure 6-11). The majority of the traffic is traveling from 6:00 AM to 5:00 PM. However, the heavier trucks tend to travel during late night and early morning hours (10:00 PM to 6:00 AM), opposite to the traffic flow volumes. The average GVWs at 4:00 PM is 53.8 kips while the average GVW at 3:00 AM is 66.8 kips, which is 24% heavier (Figure 6-12).
The heaviest overloads typically move during the daytime (Figure 6-13), likely due to the vehicle permit regulations and safety concerns. Certain types of permitted vehicles are prohibited from traveling at night by Louisiana regulation. Although the distribution shapes are similar, the separation of the GVW CDFs clearly shows differences of truck weights by hour in a typical day (Figure 6-14). Therefore, in order to use short period data to predict long period traffic, the record shall cover every hour of the day.
6.3.3.2 Lane-by-lane Variation

As shown previously, the majority of the trucks (82%+) travel in the driving lane (Table 6-4). The histograms of the GVW for lane-3 and lane-4 are shown in Figure 6-15 and Figure 6-16, respectively. The GVW distribution characteristics between the driving lane and the passing lane are similar, but the trucks that used the driving lane are heavier (10%).

Among the hundreds of nationwide WIM stations, the majority of them are single-lane WIM stations. Because of the similarity of the truck GVW distribution, the multiple presence factors can be simulated based on the single-lane records.

<table>
<thead>
<tr>
<th>Lane Number</th>
<th>N</th>
<th>% of Total</th>
<th>Mean</th>
<th>Min</th>
<th>Max</th>
<th>Std Dev</th>
<th>CV</th>
<th>Quantiles95</th>
</tr>
</thead>
<tbody>
<tr>
<td>Passing Lane-3</td>
<td>15,088</td>
<td>17.58%</td>
<td>53.97</td>
<td>12.00</td>
<td>197.10</td>
<td>22.27</td>
<td>41.27</td>
<td>90.70</td>
</tr>
<tr>
<td>Driving Lane-4</td>
<td>64,310</td>
<td>82.42%</td>
<td>59.34</td>
<td>12.00</td>
<td>229.40</td>
<td>25.15</td>
<td>42.38</td>
<td>100.20</td>
</tr>
</tbody>
</table>
6.3.3.3 Multiple-Presence (MP) Probabilities

The maximum lifetime load effect can come from more than one vehicle travelling on the same span of the bridge at the same time. There are three types of MPs: following, staggered, and side-by-side. The up-to-date WIM system recorded time to the nearest hundredth of a second and can accurately determine the headway separations. With a light truck volume for this study (ADTT<1,000), the multiple-presence probability is also very small, MP=0.23%. This MP includes all of the legal and overload trucks. This is much smaller than the 1.5% presented in the NCHRP project 12-76 (Sivakumar et al. 2008) and the 6.67% indicated in the NCHRP 368 (Nowak, 1999).

The MP probability for permit trucks is even smaller for the legal trucks due to the low permit truck volume and the span length. For the 80’ long permit trucks traveling on the 77’ long span bridge at 55 miles per hour, there could be no true following events. Only the two-lane side-by-side and staggered loading patterns were investigated. As can be expected, there is only one permit vehicle multiple presence, i.e., MP=0.001%. Among all the side-by-side cases, only two overload truck side-by-side with legal truck cases (MP =0.0025%) were recorded.
6.4 LEGAL TRUCKS (STRENGTH I) AND PERMIT TRUCKS (STRENGTH II)

In order to develop the legal trucks and permit trucks for load rating, the total 79,397 truck records were separated into two groups: Strength I and Strength II. The Strength I group includes all of the normal legal vehicles and some “illegally” overloaded vehicles; the Strength II group contains owner-specified permit vehicles. It is important to separate the very heavy loads from the routine trucks so that they do not control all of the upper tail of the normal legal traffic distribution.

The LADOTD legal truck gross weight limits vary from 80,000 pounds to 88,000 pounds for interstate highways and non-interstate highways. The limit for the strength I trucks was set to be 100,000 pounds for this study, considering the 10% equipment error and the exceptions for some overloaded legal trucks. Louisiana truck regulations require all “superloads”, those having gross weight exceeding 254 kips, to be escorted and no other heavy vehicle is allowed to cross the same bridge on the same span at the same time. A LADOTD rating engineer evaluates all superloads individually. As such, superloads were excluded from this study. Based on the above, the Strength II (permit) trucks were selected based on either trucks with 7 axles or more, or trucks with 254 kips > GVW> 100 kips. All others trucks were grouped into Strength I (legal truck).

Another very important factor for bridge rating is the weight ratio (GVW/AL where AL is the distance between the outer axles). A large weight ratio represents compact trucks, such as cranes and other special hauling vehicles. These compact trucks have more significant impact on short to medium span bridges. The statistics for GVW and GVW/Al are shown in Table 6-5. The GVW and GVW/AL generally have a linear relationship, as shown in Figure 6-17.

<table>
<thead>
<tr>
<th>Group</th>
<th>N</th>
<th>Mean</th>
<th>Std Dev</th>
<th>Min</th>
<th>Max</th>
<th>Std Err</th>
<th>CV</th>
<th>Quantile 95</th>
</tr>
</thead>
<tbody>
<tr>
<td>GVW</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Legal</td>
<td>75824</td>
<td>55.9</td>
<td>22.3</td>
<td>12.0</td>
<td>100.0</td>
<td>0.081</td>
<td>39.903</td>
<td>93.2</td>
</tr>
<tr>
<td>Permit</td>
<td>3573</td>
<td>109.5</td>
<td>16.4</td>
<td>39.6</td>
<td>229.4</td>
<td>0.275</td>
<td>14.991</td>
<td>138.9</td>
</tr>
<tr>
<td>GVW/AL</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Legal</td>
<td>75824</td>
<td>1.168</td>
<td>0.656</td>
<td>0.232</td>
<td>7.346</td>
<td>0.002</td>
<td>56.2</td>
<td>2.469</td>
</tr>
<tr>
<td>Permit</td>
<td>3573</td>
<td>2.016</td>
<td>0.599</td>
<td>0.537</td>
<td>9.029</td>
<td>0.01</td>
<td>29.71</td>
<td>2.653</td>
</tr>
</tbody>
</table>
\[ \text{GVW} = -82.37 + 112.6 \left( \text{GVW}/\text{AL} \right) \]

Figure 6-17 Average gross weight (GVW) and weight ratio (GVW/AL) relationship

6.5 LEGAL TRUCK STATISTICS

The legal truck group contains trucks having a maximum GVW of 100.0 kips and a maximum weight ratio (GVW/AL) of 7.34 kip/ft. The legal truck traffic stream is similar to the whole truck database.

Figure 6-18 shows the legal truck distribution by number of axles. Two examples of typical heavy legal truck photos are shown in Figure 6-19. The gross weight and weight ratio statistical summary for legal trucks are shown in Table 6-6.
Table 6-6 Legal truck GVW statistics by lanes

<table>
<thead>
<tr>
<th>Legal Trucks</th>
<th>Lane</th>
<th>N</th>
<th>Mean</th>
<th>Std Dev</th>
<th>Min</th>
<th>Max</th>
<th>CV</th>
<th>Quantiles95</th>
</tr>
</thead>
<tbody>
<tr>
<td>GVW (kip)</td>
<td>3</td>
<td>14894</td>
<td>53.29</td>
<td>21.50</td>
<td>12.00</td>
<td>100.00</td>
<td>40.35</td>
<td>89.00</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>60930</td>
<td>56.55</td>
<td>22.46</td>
<td>12.00</td>
<td>100.00</td>
<td>39.71</td>
<td>93.80</td>
</tr>
<tr>
<td>GVW/AL (Kip/ft)</td>
<td>3</td>
<td>14894</td>
<td>1.134</td>
<td>0.647</td>
<td>0.245</td>
<td>6.573</td>
<td>57.052</td>
<td>2.230</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>60930</td>
<td>1.176</td>
<td>0.658</td>
<td>0.232</td>
<td>7.346</td>
<td>55.977</td>
<td>2.569</td>
</tr>
</tbody>
</table>

6.5.1 One-Lane Load GVW Statistical Analysis

Different researchers have used many GVW distribution models, such as the Beta distribution (Bailey, 1996), semi-parametric (Enright and O’Brien, 2012), and bi-modal or tri-modal Normal distribution (Caprani et al. 2002). A couple of different statistical analysis methods were performed for this study to select the most suitable statistical model.

The probability plots of the gross weights and gross weight ratios from this site are shown in Figure 6-20 and Figure 6-21. The figures show the frequency histogram and a normal probability plot for lane 3 and 4. If the gross weights or weight ratios are normally distributed, the normal quantile plot approximates a diagonal straight line, similar to the red lines shown in the figures. Clearly, the PDFs do not follow any obvious probability distribution type, and the histograms have the bi-modal or tri-modal distribution shapes. However, since we are only interested in the heavy loads, the upper tail ends are more important for this study. Close observations find that the upper tails are closer to forming a straight line at tail ends of the normal probability distributions.
Figure 6-20 Legal load GVW histogram and normal probability plot – Lane 3
Two-Lane Load GVW Statistical Analysis

There were a total of 184 multiple presence events including the permit trucks within the year of study with a MP=0.23%. The average GVW distributions are shown in Figure 6-22 and Table 6-7. The red line indicates the fitted normal quantile line with the confidence bounds. All of the two-lane GVWs generally conform to the Normal distribution ($R^2 = 97.1\%$). Therefore, all of the trucks were used for the analyses.
Three multiple presence patterns, staggered, side-by-side (Figure 6-23) and following (Figure 6-24), were considered for this study. Out of the total 184 MP cases, there were only two side-by-side permit truck cases. A few following cases inferred from the WIM system data were actual one long truck-trailer separated into two vehicles based on the algorithm used in the WIM to identify truck separations. Therefore, there were even less multiple presence cases than the WIM data indicated. For this study, these long truck-trailer cases were still considered as multiple presence cases.
6.5.3 Maximum Load Projection

Typical bridge design life is 75 years, and typical bridge rating period is 5 years. Only one year of the traffic data was collected for this project. As such, using statistical methods to project the traffic loads to the design life or rating period is necessary.

One commonly used method to extend the statistics from a small size sample to a population is the Monte Carlo simulation; this method assumes that the population has similar distribution characteristics of a smaller sample. Projection using the Monte Carlo method is computation intensive and may not be practical when an extremely large sample size is needed. Other methods have been shown to produce comparable results with much less computation demand. Two prediction methods were compared for this study. One is the tail Normal
distribution method used by Nowak (1993) for LRFD calibration; the other one is the simplified Extreme Distribution method recommended by Sivakumar (2008) for LRFR calibration. Both methods assume that the WIM data is sufficiently long enough to encompass the seasonal variations.

The one-lane gross weight (GVW) and weight ratio (GVW/AL) were selected to represent the live loads. The top 5% GVW statistics were used for the projection.

First, assemble the histograms and the normal probability plots of weight ratio for each lane. Then, linearly fit the upper 5% of the tail ends of the normal probability plots. The Y-axis is the normal quantile value, which is computed as

\[ N_{\text{Quantile}} = \Phi^{-1}\left(\frac{r_i}{N + 1}\right) \]  

(6.1)

where \( r_i \) is the rank of the ith observation, \( N \) is the number of observation, and \( \Phi() \) is the cumulative probability distribution function for the Normal distribution. If the plot appears to follow a straight line, then it is reasonable to conclude that the data can be modeled using a Normal distribution. The fitted lines with a \( R^2 = 90\% \) are shown in Figure 6-25 and 6-26. The comparison of fitted CDF with actual event CDF are shown in Figure 6-27 to Figure 6-29.

![Figure 6-25 Top 5% of the GVW Normal probability plot – one-lane](image-url)
Figure 6-26 GVW Normal probability plot – Side-by-Side

Figure 6-27 Compare Normal Fitted CDF with Event CDF – Lane 3
Figure 6-28 Compare Normal Fitted CDF with Event CDF – Lane 4

Figure 6-29 Compare Normal Fitted CDF with Event CDF – Side-by-Side
The fitted CDF line can also be expressed as shown in Eq. (6.2):

\[ N_{\text{Quantile}} = n + m(GVW) \]  

(6.2)

The slope \( m \) and intercept \( n \) of the best-fit Normal distribution may be calculated from the Normal plot. For the lane 3 case, the slope \( m = 0.1864 \) and \( n = -14.6484 \).

For comparison, first use the tail Normal plot method to fit the upper tails. The normal quantile or normal inverse of the CDF can be calculated using Equation (6.1).

To project the summary statistics for 5 future years, the total number of trucks, \( N = (n_{d\text{ay}})(365)(\text{years}) \). The projected number of trucks for a 5-year period is 74,470.

\[ n_{d\text{ay}} = \text{total number of trucks per day} = \text{ADTT} = 41. \]

\[ N_{\text{Quantile}} = \phi^{-1}\left(\frac{r_i}{N+1}\right) = \phi^{-1}\left(\frac{1}{74470 + 1}\right) = 4.199 \]

The maximum 5-year GVW \( (L_{\text{max}}) \) corresponding to the probability of occurrence is:

\[ L_{\text{max}} = \frac{(N_{\text{Quantile}}) - n}{m} = 101.1 \text{ kips} \]

Then, using the simplified Extreme Distribution method (Gumbel distribution), the mean and standard deviation of the equivalent Normal distribution that best fit the tail end of WIM data can be calculated per Eqs. (6.3) and (6.4) as:

\[ \mu_{\text{event}} = -\frac{n}{m} = 78.577 \]  

(6.3)

\[ \sigma_{\text{event}} = \frac{1}{m} = 5.364 \]  

(6.4)

The mean \( (L_{\text{max}}) \) and standard deviation \( (\sigma_{L_{\text{max}}}) \) of the Gumbel distribution that best models the maximum load effects for the 5-year rating period are calculated using Eqs. (2.23) to (2.26) as following:

\[ \alpha_N = \sqrt{\frac{2\ln(N)}{\sigma_{\text{event}}}} = 0.883 \]

\[ u_N = \mu_{\text{event}} + \sigma_{\text{event}} \left[ \sqrt{2\ln(N)} - \frac{\ln(\ln(N)) + \ln(4\pi)}{2\sqrt{2\ln(N)}} \right] = 101.2 \]

\[ \overline{L_{\text{max}}} = \mu_{\text{max}} = u_N + \frac{\gamma}{\alpha_N} = 101.8 \text{ kips} \]
\[ \sigma_{L_{\text{max}}} = \frac{\pi}{\sqrt{6} \alpha_N} = 1.45 \text{ kips} \]

The results for these two methods match very well (101.1 kips versus 101.8 kips) for the 5-year prediction (99.9%). The methods also show consistent results even for the 75-year project. Therefore, the tail Normal plot method was chosen for future analysis.

Based on the WIM data from this study, the projected future truck gross weight for the period of 5 years, 20 years, and 75 years are shown in Table 6-8 using the Normal tail distribution method. The five years maximum GVW is about 10% higher than the arbitrary legal limit set for this study. The increasing ratio of 1.1, which is called the projection factor for this study, is selected.

<table>
<thead>
<tr>
<th></th>
<th>WIM_recorded</th>
<th>5 years</th>
<th>75 years</th>
</tr>
</thead>
<tbody>
<tr>
<td>One Lane</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lane 3</td>
<td>100</td>
<td>101.1</td>
<td>104.2</td>
</tr>
<tr>
<td>Lane 4</td>
<td>100</td>
<td>108.9</td>
<td>110.2</td>
</tr>
<tr>
<td>Two Lane</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td>96.2</td>
<td>109.7</td>
<td>122.9</td>
</tr>
</tbody>
</table>

The R-squared, coefficient of determination, is used to measure the goodness of the fit in the regression analysis. R-squared is calculated as shown in Equation (6.5)

\[ R^2 = \frac{\text{Sum of Squares (Model)}}{\text{Sum of Squares (C Total)}} \]  \hspace{1cm} (6.5)

where,

The Sum of Square (C. Total) is the sum of the squared difference between the response values and the sample mean.

The Sum of Square (Model) is the difference between the C Total and Error.

The Sum of Square Error is the sum of the squared difference between the fitted values and the actual values.

R\(^2\) = 100% indicates that the regression model can be used to describe all of the response data points while R\(^2\)= 0% indicates that there is no correlation between the model and measured abridge responses.

As shown in Figure 6-25, the calculated \( R^2 \) for single lane and two-lane regression line fitting is 89\%, and 97\%, respectively. The high coefficient indicates a reasonably good fit.

### 6.6 PERMIT TRUCK STATISTICS

In general, there are two types of overweight permits: annual permits and single trip permits. The approved permit trucks are allowed to travel on pre-approved truck routes. The common heavy annual permits are for cranes and tractors, etc.
There are two loading scenarios that need to be considered: a permit vehicle alone and a permit vehicle alongside a random vehicle. As discussed previously, side-by-side permit vehicles did not exist based on the observed WIM records. There is a limitation for the WIM system. If the total truck length is too long, it could be considered as two separate trucks. Therefore, these cases were considered as two trucks for this study. There were two cases of MPs of normal trucks traveling in the adjacent lane.

![Routine permit truck and Single trip permit truck](image)

**Figure 6-30 A routine permit truck and a single trip permit truck**

### 6.6.1 Traffic Stream Variability

Similar to regular traffic, the frequency of permit trucks can be seasonal and varies over time. The highest permit truck average GVW was from October to December (Figure 6-31), and the truck traffic volume is similarly relatively high from July to December (Figure 6-32). The majority of the heavier permit trucks (Figure 6-33) were traveling during the daytime from 6:00 AM to 5:00 PM (Figure 6-34). The low observed monthly difference enabled this study to use data from less than one year for long-term vehicle weight projection as described previously.

![Monthly permit truck average GVW](image)

**Figure 6-31 Monthly permit truck average GVW**
Figure 6-32 Monthly permit truck volume

Figure 6-33 Hourly permit truck average GVW

Figure 6-34 Hourly permit truck volume
6.6.2 Maximum Load Projection

The probability plots of the gross weights and gross weight ratios of the permit trucks are shown in Figure 6-35. The figures show both a percent frequency histogram and a normal probability plot. As seen, the PDF shapes do not follow any obvious probability distribution type. However, since we are only interested in the heavy loads, the tail ends are more important for this study. With careful observations, the tail ends of the WIM data (both GVW and GVW ratio) match the tail ends of normal probability distribution plot. Figure 6-36 and Figure 6-37 show the Normal prediction regression line and the comparison of prediction CDFs.

![Figure 6-35 Permit truck GVW and GVW ratio histogram and normal probability plot](image-url)
The WIM future average maximum GVWs for 5, and 75 years is shown in Table 6-9. The projected 5-year permit load is 220.8 kips, which is close to the measured one-year maximum GVW 229.4 kips (within 3%). The 75-year GVW is 7% more than the recorded maximum weight. The 5-year projected unit gross weight (GVW/AL) is 10% less than the recorded one-year maximum GVW/AL, which indicated that the recorded permit loads consisted of a very compacted overload truck. The record shows that the single axle load is 43 kips, which is much
greater than the regulation (LADOTD, 2013) allowed (30 kips). The recorded heaviest permit truck with GVW=229.4 kips is show in Figure 6-38.

<table>
<thead>
<tr>
<th></th>
<th>Max_recorded</th>
<th>5 years</th>
<th>75 years</th>
</tr>
</thead>
<tbody>
<tr>
<td>GVW</td>
<td>229.4</td>
<td>220.8</td>
<td>246.2</td>
</tr>
<tr>
<td>GVW/AL</td>
<td>9.03</td>
<td>8.11</td>
<td>13.08</td>
</tr>
</tbody>
</table>

Figure 6-38 Maximum load effect permit truck

6.7 SENSITIVITY ANALYSIS

6.7.1 Sensitivity to Sample Size

To evaluate the sensitivity of projections to sample size, the CDFs various number of month’s data were used for the traffic projection using the Normal distribution method.

Following the Normal distribution procedure, one month to nine months of data were randomly selected to calculate the longer term live loads. The projected maximum GVWs were compared to the baseline projection using one-year data as shown in Table 6-10.

The predicted permit truck GVW results are all within 2% of the baseline projection. This result also agrees with previous findings that there were no obvious seasonal differences in GVW distribution. In conclusion, the prediction method is not sensitive to the amount of the data used for the projection if there are no obvious seasonal traffic differences, even with low ADTT.
The consistent results also validated the accuracy of the projection methods, since all the projected one-year maximum GVW are very comparable with the recorded one-year maximum GVW.

Table 6-10 Maximum projected GVW ratio for increased data collecting time

<table>
<thead>
<tr>
<th>Number of Months</th>
<th>Projection Ratio (Months/One-year)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1 year</td>
</tr>
<tr>
<td>1</td>
<td>1.00</td>
</tr>
<tr>
<td>2</td>
<td>0.99</td>
</tr>
<tr>
<td>3</td>
<td>0.98</td>
</tr>
<tr>
<td>4</td>
<td>0.98</td>
</tr>
<tr>
<td>5</td>
<td>1.00</td>
</tr>
<tr>
<td>6</td>
<td>1.00</td>
</tr>
<tr>
<td>12</td>
<td>1.00</td>
</tr>
</tbody>
</table>

6.7.2 Sensitivity to ADTT

The traffic volume can be grouped into four levels:

- Light Volume: \( ADTT \leq 1,000 \)
- Average Volume: \( 1,000 < ADTT \leq 2,500 \)
- Heavy Volume: \( 2,500 < ADTT \leq 5,000 \)
- Very Heavy Volume: \( ADTT > 5,000 \)

The maximum projected permit GVW also varies with the different ADTT volumes (Table 6-11). Increasing the ADTT from 275 to 5,000 results in the increase in the projected maximum permit GVW of about 12% for a 5-year projection period. Therefore, the GVW projection is very sensitive to the ADTT.

Table 6-11 Maximum projected GVW ratio verses ADTT

<table>
<thead>
<tr>
<th>ADTT</th>
<th>GVW Ratio</th>
<th>1-year</th>
<th>2-year</th>
<th>5-year</th>
<th>20-year</th>
<th>75-year</th>
</tr>
</thead>
<tbody>
<tr>
<td>275</td>
<td>0.94</td>
<td>0.97</td>
<td>1.04</td>
<td>1.15</td>
<td>1.29</td>
<td></td>
</tr>
<tr>
<td>1000</td>
<td>1.00</td>
<td>1.03</td>
<td>1.09</td>
<td>1.20</td>
<td>1.33</td>
<td></td>
</tr>
<tr>
<td>2500</td>
<td>1.04</td>
<td>1.07</td>
<td>1.13</td>
<td>1.23</td>
<td>1.36</td>
<td></td>
</tr>
<tr>
<td>5000</td>
<td>1.07</td>
<td>1.09</td>
<td>1.15</td>
<td>1.25</td>
<td>1.38</td>
<td></td>
</tr>
</tbody>
</table>

6.8 SITE-SPECIFIC LIVE LOAD CALIBRATION AND LOAD RATING

6.8.1 Live Load Factor Calibration

The last step before load rating is to calibrate the live load factors to satisfy the target beta value. Moss (2001) recommended a simplified rating procedure for deriving the live load factor based on the site-specific WIM for load rating. This method is based on the assumption
that the target reliability index is the same (β=2.5) for legal load, permit load, and the design load at operating level. Use the site-specific WIM truck weight, volume, and side-by-side occurrences to project the 5-year maximum truck weight and compare the result with the LRFD truck weight to adjust the live load factor.

For determining site-specific live load factors, the WIM-based maximum load will be compared with the load used in the calibration of the AASHTO LRFD live load factors. This was achieved by determining the live load ratio, r, which is the ratio of the five years maximum WIM-based load to the AASHTO LRFD calibration load effect (75 years) at the operating level (β=2.5). The live load ratio can be calculated in Eq. (6.6).

\[
r = \frac{5 \text{ year maximum projected live load effect}}{(HL - 93) \text{ load effect at operating level}}
\]  

(6.6)

Derived the load rating Equation (2.39) into following equation:

\[
\gamma_{LL} = \frac{C - \gamma_D DL}{(RF)LL}
\]

Where, C is the capacity of the bridge, and DL is the factored dead load effect, and \(\gamma_{LL}\) = generalized live load factor from MBE.

The adjusted site-specific live load factor can be calculated based on the rating factors of the site-specific trucks as expressed in Eq. (6.7).

\[
r = \frac{L_{L,WIM}}{L_{L HL93}} = \frac{(\gamma_{L,opr})RF_{HL93}}{(\gamma_{L,wim})RF_{wim,projected}}
\]

(6.7)

where,

\(\gamma_{L,opr}\) = generalized HL93 live load factor for operating level from the MBE

\(\gamma_{L,site}\) = site-specific live load factor based on WIM data

A level I rating was performed for these site-specific trucks. Next, the site-specific rating factors were compared with the HL-93 load rating factor at operating level to calibrate the live load factor adjustment ratio. The calculated live load adjustment ratio “r” is shown in Table 6-12. Therefore, the calibrated live load factor including the 5-year projection factor is \(\gamma_{site} = 1.16\) for both legal loads and permit loads.

<table>
<thead>
<tr>
<th>Load Type</th>
<th>r</th>
<th>MBE Load Factor</th>
<th>RF_Flexure</th>
<th>RF_Shear</th>
<th>Projection Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Legal Load</td>
<td>0.89</td>
<td>1.30</td>
<td>3.23</td>
<td>2.85</td>
<td>1.1</td>
</tr>
<tr>
<td>Permit Load</td>
<td>0.96</td>
<td>1.20</td>
<td>3.64</td>
<td>2.34</td>
<td>1.0</td>
</tr>
<tr>
<td>HL-93_opr</td>
<td>1.00</td>
<td>1.35</td>
<td>2.52</td>
<td>2.00</td>
<td>1.0</td>
</tr>
</tbody>
</table>
6.8.2 Load Rating

Instead of the Louisiana standard trucks, the site-specific trucks as shown in Figure 6-6 and Table 6-3 also were used for the Level I load rating, namely AASHTO live load distribution method. As previously mentioned, the maximum gross weight limit for any type of legal trucks is 88 kips per LA permit regulation. For this study, 100 kips is used as maximum legal load limit to consider the 10% of WIM system and some exempted overload vehicles. Therefore, the top 20% average GVW and with maximum limit of 100 kips are used to represent the truck that operate within the weight regulation. The average of the top 5% was used to reflect the overloads.

The Level I load rating results are shown in Table 6-13. The rating factors for Louisiana general legal trucks and permit trucks are also presented for comparison. Note the site-specific trucks lead to higher load rating factors when compared with the Louisiana typical rating trucks, except the 4-axle legal trucks’. Therefore, although the LA routine trucks can envelope these site-specific trucks, using the site-specific trucks can lead to better rating for this bridge.

<table>
<thead>
<tr>
<th>Truck Type</th>
<th>Number of Axes</th>
<th>RF-Flexure</th>
<th>RF-Shear</th>
</tr>
</thead>
<tbody>
<tr>
<td>Legal Load</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Site-Specific</td>
<td>3</td>
<td>6.37</td>
<td>6.01</td>
</tr>
<tr>
<td>Site-Specific</td>
<td>4</td>
<td>3.23</td>
<td>2.85</td>
</tr>
<tr>
<td>Site-Specific</td>
<td>5</td>
<td>3.84</td>
<td>3.28</td>
</tr>
<tr>
<td>Site-Specific</td>
<td>6</td>
<td>3.72</td>
<td>3.20</td>
</tr>
<tr>
<td>LA Legal</td>
<td></td>
<td>3.14</td>
<td>2.91</td>
</tr>
<tr>
<td>Permit Load</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Site-Specific</td>
<td>3</td>
<td>10.08</td>
<td>6.85</td>
</tr>
<tr>
<td>Site-Specific</td>
<td>4</td>
<td>5.01</td>
<td>3.27</td>
</tr>
<tr>
<td>Site-Specific</td>
<td>5</td>
<td>6.33</td>
<td>3.85</td>
</tr>
<tr>
<td>Site-Specific</td>
<td>6</td>
<td>5.11</td>
<td>3.13</td>
</tr>
<tr>
<td>Site-Specific</td>
<td>7</td>
<td>5.06</td>
<td>3.19</td>
</tr>
<tr>
<td>Site-Specific</td>
<td>8</td>
<td>3.87</td>
<td>2.41</td>
</tr>
<tr>
<td>Site-Specific</td>
<td>9</td>
<td>3.64</td>
<td>2.34</td>
</tr>
<tr>
<td>LA Permit</td>
<td></td>
<td>3.11</td>
<td>2.20</td>
</tr>
</tbody>
</table>

Next, these site-specific trucks shown in Table 6-13 were rated by using the calibrated FE model (Level IV). The critical site-specific legal truck (4-axle) rating factor was increased from 3.23 (Level I) to 4.61 (Level IV) and the site-specific permit load rating factor was increased to 5.18.

Table 6-14 shows the critical truck rating summary. The flexure rating for legal load and permit load is 47% and 67% higher than the Level I – live load distribution method results (Table 4-3).
Table 6-14 Site-specific trucks rating summary based on the WIM (Level IV)

<table>
<thead>
<tr>
<th>Live Load</th>
<th>Level</th>
<th>Rating Factor (RF)</th>
<th>Controls</th>
</tr>
</thead>
<tbody>
<tr>
<td>HL-93 Inventory</td>
<td>2.02</td>
<td>Service</td>
<td></td>
</tr>
<tr>
<td>HL-93 Inventory</td>
<td>3.17</td>
<td>Shear</td>
<td></td>
</tr>
<tr>
<td>HL-93 Inventory</td>
<td>2.78</td>
<td>Flexure</td>
<td></td>
</tr>
<tr>
<td>Site-Spec. Legal</td>
<td>4.61</td>
<td>Flexure</td>
<td></td>
</tr>
<tr>
<td>Site-Spec. Permit</td>
<td>5.18</td>
<td>Flexure</td>
<td></td>
</tr>
</tbody>
</table>

6.9 SUMMARY

Instead of using notional pre-defined trucks, the site-specific trucks based on WIM data were used for the US61 bridge load rating.

After filtering, sorting, and checking, the WIM data were analyzed statistically for both one-lane and two-lane cases to project the 5-year maximum load.

Two statistical projection methods, Normal parent tail distribution method and the simplified extreme value method (Gumbel distribution), were compared and led to consistent results. The consistent results also validated the accuracy of the projection methods, since all the projected one-year maximum GVW are very comparable with the recorded one-year maximum GVW within 2%. The Normal parent tail distribution method was selected for this study. For the permit trucks, the five-year maximum GVW is close to the recorded one-year maximum value. The seventy-five year maximum GVW is about 8% higher than recorded one-year maximum value.

This site shows little seasonal variation, and the projection result is insensitive to the amount of data. One month of data is sufficient to produce consistent projection results. However, the projection method is sensitive to the traffic volume (ADTT).

In order to meet the target reliability index (β=2.5) for evaluation, the live load factors were re-calibrated based on the comparison of site statistical data with the data used for developing the LRFD specifications. The site-specific live load factor for both legal loads and permit loads is $\gamma_{site} = 1.16$. The adjusted live load factors were developed for future load rating and overload truck permit reviewing.

The critical site-specific load rating results (Table 6-14) are 47% and 67% higher than the Level I rating (AASHTO live load distribution method with predefined trucks, (Table 4-3) for site-specific legal loads and permit loads, respectively.
CHAPTER 7 LOAD RATING THROUGH SHM (LEVEL IV)

When the bridge performance becomes critical due to traffic or structure condition changes, a bridge structural health monitoring system is usually installed to observe the bridge behavior or behavior changes over time. SHM and load testing have been widely used for bridge evaluation and load rating in addition to visual inspection and theoretical analysis. The SHM system has the benefit of identifying vehicle position, speed, dynamic loading, and load distribution that WIM cannot provide. This study utilizes the site-specific traffic and bridge response data to directly rate the bridge and project the future bridge rating based on the reliability analysis.

The typical method for bridge rating employs pre-defined rating trucks and compares the calculated load effects to the assumed resistances of the bridge components based on the as-built plans and up-to-date bridge conditions. In lieu of the calculated load effects (moment, shear or serviceability), which are subject to various boundary constraints and other condition factors, the rating model for this study will be based on the actual SHM measurements (strain, rotation, displacement, and dynamic characteristics). The long term structural health monitoring can provide an abundance of information on the actual bridge performance, which represents the current bridge conditions and behaviors. No assumption of boundary conditions and environmental factors are needed. Rating the bridge using the actual bridge in service measurements and the site-specific traffic can remove the design conservatism and uncertainties, such as load distribution factors, dynamic impact, and the secondary and non-structural element effects.

7.1 INTRODUCTION

7.1.1 Strain Measurements and Load Rating

When designing bridges, conservative load factors are used to cover the uncertainties associated with the effects of many factors. When evaluating bridges, some of those uncertainties can be removed using site-specific data. If the bridge itself is used as the true model by monitoring the bridge behavior under in-service traffic, a more accurate and realistic rating can be achieved. Some of the site-specific factors that affect the bridge rating include the site-specific load, load distribution, and dynamic load; the unintended composite action continuity/fixity; and the participation of secondary members and non-structural members.

To eliminate or reduce certain uncertainties and consider all of the aforementioned factors, the ideal condition is to incorporate the direct measurements of the structural responses using a SHM system and the in-service life traffic condition using WIM. If the rating factor has always been greater than 1.0, the bridge is safe under the current site-specific service load. Furthermore, through the Extreme Value Theory, we can develop a continuous bridge rating method by applying the reliability theory to predict the future bridge rating. Any sudden load pattern or bridge response variation might indicate a bridge condition change or traffic change. Therefore, load rating should promptly reflect the actual condition accordingly. This rating
method is based on the assumption that the structure response is elastic. For inelastic behavior, further study will be needed to set the limit states.

The benefits of rating bridges based on the bridge responses are:

- The rating is for the actual observed loads and load distribution.
- The rating accounts for the in-situ truck multi-presence and live load impact implicitly.
- The rating is based on the actual bridge behavior, including the secondary member and nonstructural element effects.
- The rating considers the actual bridge conditions, such as boundary conditions, the actual damage conditions and structural damage if it exists, such as concrete cracks and section loss.
- The method can reduce the effort and assumptions needed for finite element modeling.

The most commonly used SHM data analysis methodology is based on the statistical pattern recognition approach using the extreme value theory. This method employs the concept that when the number of SHM data is large enough, the extreme value distribution of the SHM data will asymptotically approach one of the extreme distributions: Gumbel, Fisher-Tippett or Weibull.

The LRFR component rating is for both the strength limit states and service limit states. As all the live-load effects were measured in terms of strain (microstrain \(10^{-6}\)), the strength or service limit states are also expressed in terms of strains in this study. The load rating equation (MBE 6a.4.2.1-1) can be expressed in strain as in Eq. (7.1):

\[
RF = \frac{\varepsilon_R - Y_{DL}\varepsilon_{DL}}{Y_{LL}\varepsilon_{LL,\text{max}}} = \frac{\varepsilon_{LL,\text{all}}}{Y_{LL}\varepsilon_{LL,\text{max}}} \tag{7.1}
\]

where \(\varepsilon_R\) is the strain capacity, \(\varepsilon_{LL,\text{all}} = \varepsilon_R - Y_{DL}\varepsilon_{DL}\) is the maximum allowable live load strain, \(\varepsilon_{LL,\text{max}}\) is the maximum live load strain, and \(\gamma\) is the load factor. The subscripts DL and LL denote dead load and live load, respectively.

The section capacity and dead load strain can be calculated from as-built moment capacity and dead load moment. The flexure strain can be calculated from

\[
\varepsilon = \frac{M}{ES} \tag{7.2}
\]

where, \(M\) is moment, \(E\) is elastic modulus of material and \(S\) is section modulus respectively.

The maximum live load strains can be measured by the structural health monitoring system directly. Normally the monitoring period is shorter than the typical load rating period of 5 years. Therefore, some statistical prediction methods have to be used to predict maximum live load strain to the desired rating period. A strain prediction factor can be calculated as
\[ \lambda_{LL}(T) = \frac{\varepsilon_{LL,\text{max}}(t = T)}{\max(\varepsilon_1, \varepsilon_2, \ldots, \varepsilon_n)} \geq 1.0 \]  

(7.3)

where \( \varepsilon_{LL,\text{max}}(t=T) \) is the projected maximum mean strain for a return period \( T \) and \( \max(\varepsilon_1, \varepsilon_2, \ldots, \varepsilon_n) \) are the observed strains.

The live load distribution factor also varies with different bridges and loads. The site-specific live load distribution factors can present both the bridge behavior and the traffic pattern. Therefore, the site-specific live load distribution factors were developed for the future rating.

### 7.1.2 Reliability Analysis

In bridge evaluation, member safety margin may be described as the situation where the resistance (\( R \)) exceeds load effect (\( S \)). The safe limit state function can be expressed as \( G(t) = \text{Resistance-load effect} = R - S = R - DL - LL \). The load effect includes live load effect, \( LL \), and dead load effect, \( DL \). Obviously, both capacity and load effect are variables and can vary with time. This study concentrates on the largest variable in the load rating, which is the live load. It is assumed that the capacity is determinate at the time of load rating. The limit state function can be expressed as:

\[ G(t) = R - DL - LL_{\text{max}}(t) \]  

(7.4)

Probability of failure, i.e., the probability that the capacity is less than the applied load effects, may be formally calculated; however, its accuracy depends upon the probability distributions of both the load and resistance variables. The probability of failure, \( P_f \), may be expressed by integrating over the load frequency distribution curve as:

\[ P_f = P[R < S] = P[R - DL - LL(t) < 0] = \int P[R < S] f_l dt \]  

(7.5)

where \( t \) is the time and \( LL_{\text{max}}(t) \) is the maximum live load effect for the bridge evaluation period, normally 5 years; \( P[\cdot] \) is the probability; \( f_l \) is the load probability density curve.

In structural reliability theory, safety is measured in terms of the reliability index \( \beta \). The target reliability indices are 3.5 and 2.5 for the AASHTO LRFD bridge design, and the MBE for bridge evaluation, respectively.

Incorporating the concept of bridge load rating factor into the limit state function (Equation (7.4)), the bridge safety margin can be expressed as \( g(R, DL, LL) = R - DL - (RF)(LL) \). For an ideal case, a rating factor of 1.0 indicates that the reliability index equals to the target value 2.5. As such, one can include the load and resistance factors in the limit state function to achieve the target reliability index. The limit state function considering the resistance and load factors is

\[ \Phi R - \gamma_{DL}(DL) - (RF)\gamma_{LD}(LL) = 0 \]  

(7.6)
To be consistent with the LRFD, the reliability index calculation follows the process used in the calibration of design specification. Nowak assumed that the total load, \( S \), is a normal random variable and that the resistance, \( R \), is a lognormal random variable. The modified first-order second-moment reliability index for this combination is expressed in Equation (7.7) (Nowak and Collins 2000):

\[
\beta = \frac{R_n \lambda_R (1 - 2 \nu_R) [1 - \ln(1 - 2 \nu_R)] - \mu_S}{\sqrt{[R_n \nu_R \lambda_R (1 - 2 \nu_R)]^2 + \sigma_S^2}}
\]  

(7.7)

where

- \( \nu_R \) = coefficient of variation of the resistance
- \( \mu_S \) = mean of the total applied load
- \( \lambda_R R_n \) = mean unfactored resistance (actual)
- \( \sigma_S \) = standard deviation of the total applied load

Even though the live load uncertainties are tremendously reduced with the benefit of site SHM data, there are still uncertainties in modeling and data processing, such as:

- Measurement uncertainties due to gage location, data-acquisition, sensor resolution, and temperature effect.
- Strain to moment calculation uncertainties due to variability of concrete strength, section dimensions, and modulus.
- Modeling and projection uncertainties due to the statistical projecting.

The site-specific live load factors can be re-calibrated based on the calculated reliability index for future load rating.

### 7.2 INSTRUMENTATION PLAN

As discussed previously, there are several factors contribute to the measurement uncertainties from a SHM system. Therefore, it is very important that a well-planned instrumentation scheme be applied.

The long-term instrumentation for the US61 bridge includes 28 dynamic strain transducers and two data loggers. The data used for this study consists of continuously measured peak live-load strains over a period of one year. A thorough initial load test and calibration (Chapter 5), followed by periodical re-calibrations were made to ensure data quality. The instrumentation plan is illustrated in Figure 7-11 to Figure 7-13.

All twelve girders of Span-1 and Span-2 were instrumented with BDI strain gages. The strain transducers on Span-1 were located at around 40% of the span length based on the results of finite element model analysis. The strain transducers on Span-2 were located 3’-4” away from the mid-span. All strain transducers were installed with 24” gage extensions to reduce the possible erroneous measurement from potential cracking of concrete.
Figure 7-1 Instrumentation Plan
SHM measurements consist of all in-service routine traffic, which is the mixture of legal trucks and permit trucks. Since we are more interested in the bridge response under ambient traffic condition instead of the loads, there is no need to separate them. Consequently, the peak strain (S) caused by a single heavy truck or two side-by-side trucks, is all considered as one load event.

7.3 STRAIN DATA ACQUISITION AND FILTERING

Even with advanced techniques and periodic calibration, there are still some questionable observations that may affect the accuracy of the load effect modeling results. Data sorting and filtering is performed to reduce such errors.

Strain readings were collected under ambient traffic. To filter out data noise and inconsequential data, a threshold was set as a filtering criterion, and only the data exceeding the threshold were considered in this study. This system was designed to capture events based on the trigger limit of 10 microstrains (µε) on two mid-span strain transducers. Whenever the triggering limit is exceeded, the monitoring system will record a block of data at a rate of 0.02 second, only
100 records before the events and 150 records after the trigger for a 5 second event are stored. For the study period, there were over 6 million records captured. The typical peak strain ($S_{\text{max}}$) records are shown in Figure 7-4.

![Figure 7-4 Peak strains for live load events](image)

For bridge evaluation, the main concern is the peak load effects. To further simplify the recorded data, the measured maximum load effect during each loading event was identified. Furthermore, the peaks of these events were separated into different reference duration or blocks, such as an hour, a day, a week, and a month.

In summary, to maintain the quality of the SHM data and remove the unreliable observations, the data was examined and certain data was excluded using the following rules:

- All records of the mid-span where strain reading < 10 µε
- All records with the total of 6 sensors at mid-span strains < 50 µε
- All readings without WIM truck records

The procedure filtered out the calibration errors. After the scrubbing, a statistical analysis software, SAS JMP, was selected for the statistical analysis.

### 7.4 STRAIN STATISTICAL PATTERN ANALYSIS

The summary statistics of the strain readings are shown in Table 7-1. The maximum recorded peak strain is 134.30 µε at the interior girder 3 of Span-1. The records indicate the structural responses are essentially symmetrical. The two middle interior girders experienced maximum and mean strains higher than other two interior girders. The mean strains at the exterior span (Span-1) are around 20% higher than the interior span. Those results are consistent to the finite element analysis.

As the truck traffic varies with time, live load strains also vary with time. To predict the strain for a long return period, the variability of the strain pattern is first studied.
Table 7-1 Event strain summary (µε)

<table>
<thead>
<tr>
<th>Span</th>
<th>Girder</th>
<th>Gage</th>
<th>Max</th>
<th>Mean</th>
<th>Standard Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>G1</td>
<td>Strain15</td>
<td>53.88</td>
<td>14.44</td>
<td>6.20</td>
</tr>
<tr>
<td></td>
<td>G2</td>
<td>Strain16</td>
<td>98.80</td>
<td>28.41</td>
<td>9.82</td>
</tr>
<tr>
<td></td>
<td>G3</td>
<td>Strain18</td>
<td>134.30</td>
<td>38.34</td>
<td>11.32</td>
</tr>
<tr>
<td></td>
<td>G4</td>
<td>Strain20</td>
<td>131.60</td>
<td>27.40</td>
<td>8.05</td>
</tr>
<tr>
<td></td>
<td>G5</td>
<td>Strain21</td>
<td>72.39</td>
<td>13.84</td>
<td>9.71</td>
</tr>
<tr>
<td></td>
<td>G6</td>
<td>Strain22</td>
<td>44.01</td>
<td>4.03</td>
<td>7.20</td>
</tr>
<tr>
<td>2</td>
<td>G1</td>
<td>Strain01</td>
<td>6.11</td>
<td>-2.80</td>
<td>1.60</td>
</tr>
<tr>
<td></td>
<td>G2</td>
<td>Strain04</td>
<td>106.20</td>
<td>23.74</td>
<td>8.64</td>
</tr>
<tr>
<td></td>
<td>G3</td>
<td>Strain06</td>
<td>129.90</td>
<td>32.62</td>
<td>10.25</td>
</tr>
<tr>
<td></td>
<td>G4</td>
<td>Strain08</td>
<td>129.0</td>
<td>21.86</td>
<td>7.59</td>
</tr>
<tr>
<td></td>
<td>G5</td>
<td>Strain09</td>
<td>111.70</td>
<td>11.76</td>
<td>8.55</td>
</tr>
<tr>
<td></td>
<td>G6</td>
<td>Strain10</td>
<td>46.61</td>
<td>2.81</td>
<td>5.71</td>
</tr>
</tbody>
</table>

7.4.1 Seasonal Variation Factor

The seasonal variation of the traffic has been discussed in the preceding Chapter 6 on WIM study. The traffic study indicated very small GVW deviation from month to month throughout the studied period. This section contains the study of the seasonal variation of the peak strains.

The interior girder 3 (strain18) is used here to compare the strain gage records as shown in Table 7-2. The average peak strain spread out fairly throughout the year (Figure 7-5 and Figure 7-6 for monthly and hourly strains, respectively). The mean peak strains range from 28.1 µε to 30.1µε. All strains but those recorded in August are within 2% of the mean value (Table 7-2) based on a 95% confidence interval. The maximum peaks observed were in the month of June. A comparison of the monthly probability density distributions shows only minor variations (Figure 7-7).

In conclusion, this site does not display obvious seasonal difference based on the peak strain observations, which mirrors the WIM study results. The seasonal difference factor can be set as 1.0 for this site.
Table 7-2 Monthly strain statistics comparison

<table>
<thead>
<tr>
<th>Month</th>
<th>Number</th>
<th>Mean</th>
<th>Standard Deviation</th>
<th>Lower 95%</th>
<th>Upper 95%</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1295</td>
<td>28.17</td>
<td>7.48</td>
<td>27.76</td>
<td>28.58</td>
</tr>
<tr>
<td>2</td>
<td>1400</td>
<td>28.28</td>
<td>7.85</td>
<td>27.87</td>
<td>28.69</td>
</tr>
<tr>
<td>3</td>
<td>1278</td>
<td>29.06</td>
<td>8.09</td>
<td>28.62</td>
<td>29.51</td>
</tr>
<tr>
<td>4</td>
<td>1652</td>
<td>29.34</td>
<td>8.54</td>
<td>28.92</td>
<td>29.75</td>
</tr>
<tr>
<td>5</td>
<td>1246</td>
<td>29.45</td>
<td>9.04</td>
<td>28.95</td>
<td>29.96</td>
</tr>
<tr>
<td>6</td>
<td>2176</td>
<td>29.55</td>
<td>8.44</td>
<td>29.20</td>
<td>29.91</td>
</tr>
<tr>
<td>7</td>
<td>2078</td>
<td>29.34</td>
<td>8.52</td>
<td>28.97</td>
<td>29.70</td>
</tr>
<tr>
<td>8</td>
<td>4407</td>
<td>24.28</td>
<td>9.67</td>
<td>23.99</td>
<td>24.56</td>
</tr>
<tr>
<td>9</td>
<td>2821</td>
<td>30.11</td>
<td>9.23</td>
<td>29.77</td>
<td>30.45</td>
</tr>
<tr>
<td>10</td>
<td>2202</td>
<td>28.74</td>
<td>8.53</td>
<td>28.39</td>
<td>29.10</td>
</tr>
<tr>
<td>11</td>
<td>1748</td>
<td>28.76</td>
<td>8.34</td>
<td>28.37</td>
<td>29.15</td>
</tr>
<tr>
<td>12</td>
<td>1296</td>
<td>28.13</td>
<td>7.59</td>
<td>27.72</td>
<td>28.55</td>
</tr>
</tbody>
</table>

Figure 7-5 Monthly strain (με) comparison
7.4.2 Girder Strain Distribution

The interior girder mid-span strains, including Strain 16, 18, 20 and 21, are compared to ensure the data quality. Figure 7-8 shows a time series plot of strain measurements of four strain transducers and Figure 7-9 shows all the interior girder strain correlations. The consistency of patterns further reinforces the quality of the SHM system. The linearly correlation shows how
strongly these of strains are related. Since there are two traveling lanes, the strain data forms two correlation lines.

Figure 7-8 Span-1 interior girder strain comparison

Figure 7-9 Interior girder strain correlations
The girder strain histogram of strain 18 is shown in Figure 7-10. Figure 7-11 and Figure 7-12 show the distribution of the sum of the girder strains at the mid-span of Span-1 and Span-2, respectively. See Appendix B for girder strain histograms of all other girders. Although the distributions do not follow any of the common statistical distributions, the Span-1 and Span-2 patterns are similar. The maximum mean strains were observed at the 2\textsuperscript{nd} interior girder of the driving lane (strain 18). As expected, the exterior girder (strain 21) at passing lane experienced the lowest mean strains among all the girders.

![Figure 7-10 Span-1 interior girder strain histogram - Strain 18](image)

![Figure 7-11 Span-1 girder strain histogram - sum at mid-span](image)
Figure 7-12 Span-2 girder strain histogram - sum at mid-span

7.5 PEAK STRAIN STATISTICAL DISTRIBUTION AND PROJECTION

Statistically, there is a great probability of occurrence of extreme value when inferring from a small sample data set to the larger population. The same idea also applies to the projection of short duration data to longer periods such as the case for using one-year data to project to a five-year bridge evaluation period. To project the maximal load effect to the next 5-year period, it is common to apply extreme value distributions projection to the observations.

The underlying assumption of this method is that each peak strain measurement is independent and identically distributed (Catbas, 2009). Since a vehicle travels at an average speed of 55 mph, it will take less than one second to cross the 77-ft long span and less than three seconds to pass the entire 3-span unit. The 5-second block should be sufficiently long enough to catch the entire responses of each independent event. Thus, the independent event assumption can be accepted as valid. In addition, the threshold strain limits the measurement noise to help ensure the independence of the measurements (Bhattacharya, et al., 2005), as described in Chapter 7.3. Without knowing the peak strain statistical distribution, the sample data mean and standard deviation for the observation set \((x_1, x_2, \ldots x_n)\) are calculated as

\[
\bar{x} = \frac{1}{n} \sum_{i=1}^{n} x_i
\]

\[
\sigma = \sqrt{\frac{1}{n} \sum_{i=1}^{n} (x_i - \bar{x})^2}
\]

The cumulative probability \(p_i\) is equal to

\[
p_i = \frac{i}{n + 1}
\]
The extreme value projection techniques are used to project for long return periods using limited data. Many methods have been developed for estimating the distribution parameters. Three methods were used for this study: the upper tail fitting method, the block maxima method and the tail fitted block maxima method. The Gumbel tail fitting method is proven to provide the best fit for the strain distribution and projection.

7.5.1 Parent Distribution Power Rule (PD)

As we discussed in the previous chapters, if \( n \) independent events \( x_1, x_2, \ldots, x_n \) follow the same probability distribution, one can obtain the maximum value distribution by raising the parent distribution \( F(x) \) to the \( n \)th power \( \{F(x)\}^n \) directly (Equations 2.18 to 2.20). The new CDF can be used to estimate the mean and the standard deviation of the expected maximum peak strains for the projected period.

This prediction method is straightforward. It requires high accuracy of \( F(x) \) to approximate the upper tail of \( F(x)^n \). When the future number of events \( n \) becomes large, this method is no longer accurate.

For example, if the site ADTT = 275, the parent distribution of the daily maximum need to be raised to a power of 275. For 5-year maximum strain, the number of truck loading events becomes \( n = (275 \text{ trucks/day}) \times (365 \text{ days}) \times (5 \text{ years}) = 500,000. \) Obviously, the original data is insufficient to obtain an accurate distribution for the maximum monthly or yearly strain directly (Figure 7-13). A statistical projection is necessary for calculating the 5-year maximum strain.

![Figure 7-13 Parent distribution and projected cumulative distribution of one-day, one-week, one-month, and one-year maximum strains](image-url)
7.5.2 Parent Tail Distribution Method (PTD) – Normal Distribution

Nowak (1999) used the parent tail distribution method to develop the live load model for the AASHTO LRFD Specifications to predict the mean 75-year maximum load effect. Due to the limitation of the two-week only samples, instead of fitting the entire set of data of the normal histogram, fitting of the upper tail of the normal probability plot was used. The upper tail of the distribution of the measured data was assumed to distribute similarly to the upper tail of a Normal distribution. When plotted on a normal probability paper, the upper tail should form a straight line. The future load effect can be read from the fitted plot directly.

One interior girder (strain 18) was selected here to demonstrate the tail fitting method. Similar to the gross weight distribution, the strain distribution does not follow any of the typical probability distribution types based on the histogram (Figure 7-14).

![Figure 7-14 Histogram of strain 18](image)

A normal probability plot is executed by taking normal inverse or normal quintile ($\Phi^{-1}[F(x)]$) of the cumulative distribution of the strain $F(x)$, as the Y axis (N-quantile) and take the strain in microstrain as the X axis. The plot would produce a straight line if the data set follows a Normal distribution. The empirical cumulative strain distributions of the strain measurements of strain gage 18 are presented on the normal probability plot in Figure 7-15. Figure 7-15 (a) shows the cumulative plot for the entire data set. Observation from the plot indicated that while the plot shows significant deviation from the Normal distribution, the upper tail clearly shows a straight line pattern with the exception of two extreme high strains. The upper 5% of the strain readings were re-plotted in Figure 7-15(b). The regression analysis of the linear correlation shows the top 5% (683 data points) data forms a strong correlation to a straight line, $N_{\text{quantile}}=-0.41791+0.0381595(\mu\varepsilon)$ with a regression efficient $R^2=0.973$. The WIM system recorded 100,000 truck events over a year. For a 5-year projected period, the total number of truck events will be $100,000 \times 5 = 500,000$. The cumulative probability is $p=1/(500,000+1) = 2e^{-6}$. The return level (N-quantile) for $\Phi^{-1}(1-p)$ is 4.61, and the strain is 135.1 $\mu\varepsilon$, according to Figure 7-15(b). It is observed that the 5-year projected maximum value, 135.1 $\mu\varepsilon$ is close to the one-year maximum value observed from the SHM 134.3 $\mu\varepsilon$. The predicted
maximum 75-year maxima leads to 148.77\(\mu\varepsilon\), which exceeds the recorded SHM maximum by 11\%.

![Graphs showing CDF and upper 5% tail CDF fitting](image)

(a) CDF
(b) Upper 5% tail CDF fitting

\[ Y = -0.41791 + 0.0381595X \]
\[ R^2 = 0.973 \]

Figure 7-15 Normal plot of strain 18 and upper 5% tail fitting

7.5.3 Parent Tail Distribution Method - Gumbel

Sivakumar et al. (2011) proposed an alternative method to analyze longer return periods for a WIM study. Since the tail end of the data approaches a Normal distribution, this allows the application of the extreme value theory to obtain the maximum strain, called simplified Tail Gumbel Distribution method.

On the Normal plot, the standard Normal distribution CDF and inverse of the CDF could be expressed as equations (7.11) and (7.12).

\[ F(x) = \Phi \left( \frac{x - \mu_x}{\sigma_x} \right) \quad (7.11) \]

\[ \Phi^{-1}(F(x)) = z = \left( \frac{1}{\sigma_x} \right) x + \left( \frac{-\mu_x}{\sigma_x} \right) = mx + n \quad (7.12) \]

Similar to the previous tail fitting method in Figure 7-15, the upper 5% of the data approaches a straight line with a slope of \(m=0.038\) and an intercept of \(n=-0.418\). The mean of the equivalent Normal distribution that best fits the tail end mean is \(\mu_{\text{event}} = -n/m=10.952\) and the standard deviation is \(\sigma_{\text{event}}=1/m=26.205\). Figure 7-16 and Figure 7-17 show the cumulative distribution and probability distribution plots of the fitted Normal distribution and the observed data. They clearly indicated a close fit at the upper tail.
Figure 7-16 The Fitted Hypothetical Normal CDF and observed CDF

Figure 7-17 The Fitted Hypothetical Normal PDF and observed PDF
Applying the Equation (2.23) to (2.26), the maximum 5-year and 75-year mean strains are 135.10 με and 148.77 με, respectively. The deviation between the prediction and the observed one year SHM strains for the 5-year and 75-year return periods are 1% and 11%, respectively. The result matches the normal plot method very well.

The 5% upper tail is also fitted to the Gumbel distribution directly (Figure 7-18). The projected 5-year maxima is 134.5 με, which also matches other methods well.

![Gumbel Distribution](image1)
![Gumbel CDF (upper 5%)](image2)

(a) Gumbel Distribution  
(b) Gumbel CDF (upper 5%)

Figure 7-18 Gumbel Distribution Fit

### 7.5.4 Block Maxima Method (BM)

This method groups the n independent maximum strain observations, \(x_1, x_2, ..., x_n\), into m blocks of a sequence of observations in a loading event or within a reference period of time, such as a day or a week, and generate a series of block maxima, \(M_{n1}, M_{n2}, ..., M_{nm}\)

\[
M_{nl} = \max(x_{l1}, ..., x_{ln})
\]  
(7.13)

Where \(x_{l1}..x_{ln}\) is the independent observations during each reference period of time (block).

These block maxima \(M_{nl}\) can then fit to one of the types of extreme value distributions (Gumbel, Frechet and Weibull) for future maxima.
The SHM data is separated into a couple of subsets of dates with different block lengths; including maxima per day, per week, and per month. The parameters of the distributions are estimated using the maximum likelihood estimation method. The estimated quantiles are plotted on the extreme value distribution paper to verify the quality of the fit.

For the daily maxima, Gumbel distribution (also named LEV in SAP software) provides the best fit per the maximum likelihoods method. For the 5-year maxima, the characteristic value Lmax=138.8 µε for 1-p = 1-1/(365x5)= 0.99945 can be read from the Figure 7-19 directly. The Frechet distribution fit is also shown in Figure 7-20, which indicates the Lmax value of 140.0 µε.

For the weekly maxima data, both Frechet and Gumbel fit the data well (Figure 7-21 and Figure 7-22). The probability is calculated as 1-p = 1-1/(52x5) = 0.9962. The 5-year maxima Lmax are 197.5 µε and 142.8 µε, respectively.

In the upper tail method, most of the data does not follow the extreme value distribution except the upper tails. Therefore, it is common to use the upper tail point only to generate the distribution, called the Tail Block Maxima (TBM) method in this study. The number of points have been selected between $2\sqrt{n}, 3\sqrt{n}$, or 30% (Castillo et al. 2004, O’Brain et al. 2003, O’Connor 2001, and Enright 2010). This study selected 30% of the block tail and the projection results are shown in Table 7-3.

![Gumbel Plot](image1.png)  
![Gumbel CDF](image2.png)

Figure 7-19 Daily maxima fitted to Gumbel distribution
Figure 7-20 Daily maxima fitted to Frechet distribution

Figure 7-21 Weekly maxima fitted to Frechet distribution
7.5.5 Projection Comparison and Conclusion

The performances of the prediction methods used to estimate the maximum strain are compared in Table 7-3. The results of the 5-year projections is closed to the maximum value observed from the one-year SHM record set. The 75-year projections lead to an average of about 12% beyond the SHM observed maximum value. The one-year projection value is also calculated to verify the accuracy of the projection methods.

The best three tail distribution fitting methods (Normal, Gumbel and Frechet) result in very similar projections. The deviation for the 5-year predictions are within 2% of the project average and for the 75-year projects are within 4% of the projected average.

The two best-fit statistical models using the Block Maxima methods are the Gumbel and the Frechet distributions. The projections using daily maxima, weekly maxima, and monthly maxima are performed for the two distribution methods. The monthly maxima data fitting is not as good as the daily and weekly data due to the limited amount of data (12 points only). The Gumbel distribution produces the closest estimates to the recorded maxima for both the daily and weekly maxima for the 5-year projection. As observed, the larger the sample block size, the higher the estimated maximum value. Generally, load rating only considers the live load and bridge conditions for the next five years. Therefore, all the following methods are suitable for the live load projection: Parent Tail Distribution method (5%), Block Maxima Method and Tail Block Maxima Method (TBM) (30%).
In this case, the recorded maximum peak strain during the monitoring period appears to be close to the 5-year projection and therefore is suitable for bridge evaluation. It should be noted that all projections are based on the assumption that the future traffic pattern and ADTT will remain the same during the projection period.

Table 7-3 Statistical projection comparison

<table>
<thead>
<tr>
<th>Statistical Distribution Method</th>
<th>Projected Maximum Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1-year</td>
</tr>
<tr>
<td><strong>Parent Tail Distribution (PA) (5%)</strong></td>
<td>Normal Tail Plot</td>
</tr>
<tr>
<td></td>
<td>Simplified Gumbel</td>
</tr>
<tr>
<td></td>
<td>Gumbel</td>
</tr>
<tr>
<td></td>
<td>Frechet</td>
</tr>
<tr>
<td><strong>Block Maxima (BM)</strong></td>
<td>Daily Maxima Gumbel</td>
</tr>
<tr>
<td></td>
<td>Daily Maxima Frechet</td>
</tr>
<tr>
<td></td>
<td>Weekly Maxima Gumbel</td>
</tr>
<tr>
<td></td>
<td>Weekly Maxima Frechet</td>
</tr>
<tr>
<td></td>
<td>Monthly Maxima Gumbel</td>
</tr>
<tr>
<td></td>
<td>Monthly Maxima Frechet</td>
</tr>
<tr>
<td><strong>Tail Block Maxima (TBM) (30%)</strong></td>
<td>Daily Maxima Frechet</td>
</tr>
<tr>
<td></td>
<td>Daily Maxima Gumbel</td>
</tr>
<tr>
<td></td>
<td>Weekly Maxima Gumbel</td>
</tr>
<tr>
<td></td>
<td>Weekly Maxima Frechet</td>
</tr>
<tr>
<td></td>
<td>Hourly Maxima Frechet</td>
</tr>
<tr>
<td></td>
<td>Hourly Maxima Gumbel</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td>123.3</td>
</tr>
</tbody>
</table>

*Percentage = projected/maximum recorded strain (134.3 με)

Among the three projection methods, the TBM and the parent tail method provide more consistent results than the BM method (Figure 7-23). The percentages of the data used to fit the
distribution do not result in significant differences (Figure 7-24). Although the Frechet distribution provides a better fit with some data, the Gumbel distribution provides more consistent results throughout (Figure 7-25). For the BM or the TBM method, the block size of daily or weekly provides results closest to the whole data set observations (Figure 7-26). The weekly block predicted a somewhat larger maximum value.

Figure 7-23 Prediction type comparison

Figure 7-24 Sample size comparison
Based on the above analyses, fitting the tail of block maximum data to Gumbel or Normal distributions is the most suitable method for the projection of live load strain.

The block size can be 5% of the one-year event data, daily-maxima, or weekly-maxima blocks. The weekly maxima block projects higher strain values.

Based on the closeness of the fit, the Gumbel distribution using 5% tail fitting is used for the rating analysis. The Gumbel distribution statistics are shown in Table 7-4. The projected 5-year maximum strains are shown in Table 7-5.
Table 7-4 Strain Gumbel distribution statistics

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Estimate</th>
<th>Std Error</th>
<th>Lower 95%</th>
<th>Upper 95%</th>
</tr>
</thead>
<tbody>
<tr>
<td>location</td>
<td>60.02</td>
<td>0.22982290</td>
<td>59.566885</td>
<td>60.469530</td>
</tr>
<tr>
<td>scale</td>
<td>5.763946</td>
<td>0.18917914</td>
<td>5.409104</td>
<td>6.151926</td>
</tr>
<tr>
<td>Mean</td>
<td>63.342627</td>
<td>0.28081161</td>
<td>62.792246</td>
<td>63.893008</td>
</tr>
</tbody>
</table>

The cumulative probability for the 5-year rating period is calculated as

\[ 1 - p = 1 - \frac{1}{100,000 \times 5} = 0.999998 \]

Table 7-5 5-year maximum projection for strain

<table>
<thead>
<tr>
<th>1-p</th>
<th>Strain</th>
<th>Lower 95%</th>
<th>Upper 95%</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.999998</td>
<td>135.65</td>
<td>130.64</td>
<td>140.66</td>
</tr>
</tbody>
</table>

To be conservative, the upper 95% maximum expected 5-year live load strain is selected and the mean and standard deviations are \( \mu_{L_{max}} = 140.66 \ \mu \varepsilon \), \( \sigma_{L_{max}} = 6.15 \mu \varepsilon \), respectively.

The coefficient of variation \( V = \frac{\sigma_{L_{max}}}{\mu_{L_{max}}} = \frac{6.15}{140.66} = 4.4\% \). The projection factor can be calculated from Equation (7.3) as:

\[ \lambda_{LL(5\text{-year})} = \frac{140.66}{134.3} = 1.05 \]

Only the analyses associated with gage 18 (girder 3 Span-1) are presented in the main text of this dissertation. The results of the analyses of other girders are presented in Appendix B.

### 7.5.6 Convergence Check

To check the convergence of the data set, the Strain 18 data were analyzed for using incremental time periods from 3 months to 12 months. Three random sets of data are selected for each time period. The average projected maximum strains are compared with the 12-month measurements. Although the 3-month data prediction result is within 6\%, 6-month to 12-month data prediction are more accurate. The differences between the predicted strains is within only 3\% (Table 7-6). Hence, it can be concluded that six months or longer will satisfy the minimum requirement for the analysis.

The consistent results can also validate the accuracy of the statistical projection methods. The projected one-year maximum strains are very comparable with the recorded one-year maximum strain within 3\% for the dataset from 6-month to 12-month. Therefore, the Parent Tail Gumbel Distribution method is the recommended statistical analysis method.
<table>
<thead>
<tr>
<th>Time Period</th>
<th>Set 1</th>
<th>Set 2</th>
<th>Set 3</th>
<th>Average</th>
<th>Average/Recorded</th>
</tr>
</thead>
<tbody>
<tr>
<td>3 Months</td>
<td>140.33</td>
<td>141.44</td>
<td>147.14</td>
<td>142.97</td>
<td>1.06</td>
</tr>
<tr>
<td>6 Month</td>
<td>145.65</td>
<td>132.69</td>
<td>136.67</td>
<td>138.34</td>
<td>1.03</td>
</tr>
<tr>
<td>9 Month</td>
<td>133.27</td>
<td>137.08</td>
<td>132.20</td>
<td>134.18</td>
<td>0.99</td>
</tr>
<tr>
<td>11 Month</td>
<td>135.93</td>
<td>136.35</td>
<td>138.12</td>
<td>136.80</td>
<td>1.02</td>
</tr>
<tr>
<td>Recorded</td>
<td></td>
<td></td>
<td></td>
<td>134.3</td>
<td>1.00</td>
</tr>
</tbody>
</table>

7.6 **LOAD RATING FROM THE SHM MEASUREMENT DIRECTLY**

As calculated previously, the 5-year maximum live load strain was found to be $\varepsilon_{\text{LL}\text{ max}} = 140.66 \mu\varepsilon$ and $\sigma = 6.15 \mu\varepsilon$.

The observed maximum live load strains for the exterior girder are smaller with a maximum of $46.61 \mu\varepsilon$, about 33% of interior girder 3 Span-1. This is due to the lane location and the contribution of the concrete barrier stiffness. Since the girders are all designed the same, the exterior girders do not control the load rating for this bridge.

The maximum allowable live load strain derived from MBE 6a.4.2.1-1 is

$$LL_{\text{alt}} = C - (\gamma_{DC})(DC)$$  \hspace{1cm} (7.14)

$$\varepsilon_{LL_{\text{alt}}} = \frac{C - (\gamma_{DC})(DC)}{ES}$$  \hspace{1cm} (7.15)

Where, $C$ is the strain capacity and $\varepsilon_{LL_{\text{alt}}}$ is the allowable live load strain. Since the live load impact is included in the measurement, IM can be eliminated from the calculation. Using the same live load factor, the flexure strength limit rating can be calculated as following:

$$\varepsilon\varepsilon_{LL_{\text{alt}}} = \frac{1.0M_n - 1.25M_{DL}}{ES}$$

$$= \frac{4598.48 - 1.25 \times 919.27}{4792.82 \times 35190.07}$$

$$= 813.4 \times 10^{-6}$$

$$RF = \frac{\varepsilon_{LL_{\text{alt}}}}{(Y_{LL})\varepsilon_{LL_{\text{max}}}} = \frac{813.4 \times 10^{-6}}{1.30 \times 140.67 \times 10^{-6}} = 4.40$$

The service limit state rating factor is 2.40 using the allowable stress limit at 475 psi.
7.7 LIVE LOAD DISTRIBUTION FACTORS (LDF)

The LRFD live load effects can be calculated using the live load distribution factors directly. The site-specific live load distribution factors will help for future load rating, especially for permit load reviewing.

For typical pre-stressed concrete girder bridges, the distribution factors for interior girders are given in Table 4-1.

The multi-presence factor MP=1.2 is included in the single lane distribution factor which accounts for the higher probability of having one heavy truck in one lane as compared to the probability of having two side-by-side heavy trucks in two adjacent lanes. For this study, the live load distribution factors are calculated directly from the strain measurements. To compare the results, MP=1.2 shall be removed from the equation. To obtain the mean value of the load distribution factors, Nowak (1999) assumed that the AASHTO LRFD distribution factor (DF) is the actual mean value of the distribution factor.

The calculated distribution factors for this bridge based on the LRFD Specifications are DF\textsubscript{multi} = 0.652 and DF\textsubscript{single} = 0.472 for multi-lane and single-lane, respectively. The live load moment effect can be calculated from $M_{\text{max}} = L_{\text{max}} \times IM \times DF$, where IM is the dynamic load allowance = 1.33, DF is the live load distribution factor, and $L_{\text{max}}$ is the maximum live load effect.

The load carried by a girder can be calculated from the measured strains across the cross section since the strain gages were installed at the bottom of each girder across the bridge at the mid-span and ends of the span. Regardless of the number of loaded lanes, the total load effects (moment) can be expressed as:

$$M_{\text{Total}} = \sum_{i=1}^{n} M_i = \sum_{i=1}^{n} (\varepsilon E S)_i$$

(7.16)

Where,

$n$ = total number of girders,
$E$ = Young’s modulus and
$S$ = the section modulus.

The girder load distribution factor (LDF) can be derived as:

$$LDF = \frac{N (\varepsilon E S)_j}{\sum_{i=1}^{n} (\varepsilon E S)_i}$$

(7.17)

Where N is the number of loaded lanes.

The LDF reflects a relative distribution of load effects among the girders. If the stiffness of each girder is the same, the LDF would be the percentage of measured girder strain over the
total girder strain. The prior load test and the calibrated FEM model indicate that the exterior girder stiffness is obviously much greater than the interior girder stiffness.

In order to use strain rating directly, we assume all the interior girder stiffness \((EI)_{int}\) remain the same. The equivalent exterior girder is expressed as

\[
\varepsilon_{ext}(ES)_{ext} = \varepsilon_{ext}(ES)_{int} \frac{(ES)_{ext}}{(ES)_{int}} = k \varepsilon_{ext}(ES)_{int}
\]  

(7.18)

where \(k = \frac{(ES)_{ext}}{(ES)_{int}}\)

Equation (7.17) can be converted into the load distribution factor as the following equations.

\[
LDF_{int} = \frac{(\varepsilon)_{int}}{\sum \varepsilon_{int} + \sum k \varepsilon_{ext}}
\]  

(7.19)

\[
LDF_{ext} = \frac{k(\varepsilon)_{ext}}{\sum \varepsilon_{int} + \sum k \varepsilon_{ext}}
\]  

(7.20)

\[
LL_{max} = (LDF) (L_{max})
\]  

(7.21)

The calibrated girder stiffness is obtained from prior calibrated FE model. The girder effective modulus of elasticity of concrete, \(E\), and section modulus, \(S\), can be derived from girder effective flexural stiffness \((EI)_{eff}\) and center of gravity \((CG)_{eff}\) based on the load test results as follows:

\[
(ES)_{eff} = \frac{E_{eff} I_{eff}}{(CG)_{eff}}
\]  

(7.22)

The optimized FEM developed based on the load test results indicated that the effective exterior girder ES is 2.514 times more than the interior girder ES. Therefore, all exterior strains are increased by \(k = 2.514\) to calculate the total live load distribution factor.

The total live load moment effect (in effective strains) fitted using the Gumbel distribution results in, location \(\xi=138.598\), scale \(\sigma=32.11\) and mean \(\mu=157.13\) as shown in Figure 7-27. This distribution is similar to each girder strain distribution.
The interior girder live load distribution factors for moment were calculated following Equation (7.19). The interior girder LDF follows a Normal distribution as shown in Figure 7-28 with mean = 0.289 and standard deviation = 0.028 for strain 18. The live load moment distribution factor histogram for other girders are shown in Reference B.3.

The mean LDF for interior girders ranges from 0.081 for the passing lane to 0.315 for the driving lane. Most of the higher distribution factors are due to the effects of very light trucks. The lower total strain will cause higher errors in the distribution result. Since we are more
interested in the heavy truck effects, only the top 30% of the highest loads are considered, as was done in the Block Maxima Tail method. The girder distribution factor statistics are shown in Table 7-7. The average interior girder LDF is 0.184 and COV=18%. Even the maximum LDF under the driving lane, LDF_{int,max}=0.366 is still much less than the LRFD Specification of 0.652 for multi-lane and 0.472 for single lane. Note that the live load impact is included in the girder SHM strain readings. The multiple presence factors 1.2 for single lane is included in the AASHTO Spec live load distribution factors, and the 1.33 impact factor is not included. For comparison, the live load effect including the IM should be DF_{single}= 0.472×1.33÷1.2=0.5231, and DF_{multi}=0.652×1.33=0.868. The results are at least 40% higher than the maxima measured results. The average interior girder distribution factor has a COV of 18%.

The LFD for exterior girders is higher than the interior girders due to the stiffness of the concrete barrier; and the maximum strain is low also because of the barrier section. The maximum exterior girder LDF= 0.505 which is also less than the LRFD DF (0.654 for one-lane and 0.684 for multi-lane). Although the exterior girder LDF is much higher compared with the interior LDF, the maximum strain reading is only 46.61με, which is only 1/3 of maximum interior girder strain reading. Therefore, the exterior girder is seldom the critical member in load rating for this type of structure.

Table 7-7 Live load distribution factor statistics

<table>
<thead>
<tr>
<th>Span</th>
<th>Girder</th>
<th>Strain</th>
<th>N</th>
<th>Mean</th>
<th>Std Dev</th>
<th>Max</th>
<th>COV</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>Strain15</td>
<td>17885</td>
<td>0.236</td>
<td>0.084</td>
<td>0.505</td>
<td>35.670</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>Strain16</td>
<td>17885</td>
<td>0.183</td>
<td>0.049</td>
<td>0.272</td>
<td>26.990</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>Strain18</td>
<td>17885</td>
<td>0.243</td>
<td>0.053</td>
<td>0.355</td>
<td>21.896</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>Strain20</td>
<td>17885</td>
<td>0.173</td>
<td>0.033</td>
<td>0.366</td>
<td>19.313</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>Strain21</td>
<td>17885</td>
<td>0.091</td>
<td>0.051</td>
<td>0.278</td>
<td>55.861</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>Strain22</td>
<td>17885</td>
<td>0.075</td>
<td>0.103</td>
<td>0.487</td>
<td>137.200</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>Strain01</td>
<td>2348</td>
<td>0.024</td>
<td>0.003</td>
<td>0.034</td>
<td>10.293</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>Strain04</td>
<td>2348</td>
<td>0.198</td>
<td>0.020</td>
<td>0.232</td>
<td>9.986</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>Strain06</td>
<td>2348</td>
<td>0.262</td>
<td>0.027</td>
<td>0.312</td>
<td>10.475</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>Strain08</td>
<td>2348</td>
<td>0.188</td>
<td>0.009</td>
<td>0.311</td>
<td>4.530</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>Strain09</td>
<td>2348</td>
<td>0.136</td>
<td>0.022</td>
<td>0.281</td>
<td>15.875</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>Strain10</td>
<td>2348</td>
<td>0.193</td>
<td>0.032</td>
<td>0.433</td>
<td>16.358</td>
</tr>
</tbody>
</table>
The AASHTO-LRFD girder distribution factor was developed using mean + one standard deviation (Zokaie, 2000). Thus, the recommended site-specific LDF can be calculated as 

\[
\mu + \sigma = (1 + V)\mu = \frac{0.184(1+18\%)x1.2}{1.33} = 0.196 \text{ of total load for both single and multilane trucks.}
\]

To be conservative and consider the multiple presence factor of 1.2 and the impact factor variance, \(LDF_{\text{single}} = 0.25\) and \(LDF_{\text{multi}} = 0.45\) are recommended for the interior girder moment calculation for this site (Figure 7-29). The shear and deflection LDF is not part of this study.

### 7.8 RELIABILITY ANALYSIS

The goal of structural reliability analysis is to account for uncertainties while evaluating the safety of the structure. The uncertainties associated with the bridge evaluation include the capacity of the structure and the loads.

The AASHTO design and evaluation codes use nominal mean and bias values for the variables used in design equations. The probability distribution of the random variable must also be specified.

To be consistent with the current bridge design process, the same statistical data for member resistance and dead load statistics used by Nowak (1999) during the calibration of the AASHTO LRFD are used for this study. It is assumed that the total load is a normal random variable and the resistance is considered as a lognormal random variable.
As described in Chapter 2, the uncertainties of the resistance for bridge component are: material (M), fabrication (F), and analysis (P). Material uncertainties include strength of material, modulus of elasticity, and cracking stress (Table 2-4). The fabrication uncertainties are caused by geometry, dimension and section modulus. The analysis uncertainties are due to the approximation of analysis methods. Since this study will use the in-service bridge response directly, the analysis varieties can be reduced. From Table 2-4, the moment resistance follows a lognormal probability distribution with statistic bias $\lambda_{FM} = 1.04$ and $V_{FM} = 0.045$. Considering $\lambda_P = 1.0$ and $V_P = 0$, the resistance statistical data is $\lambda_R = 1.04$ and $V_R = 0.045$.

The dead load effects are assumed to follow the Normal probability distribution; and the mean and COV can be found in Table 2-1. For the prestressed girder bridge, the bias for prefabricated girder element (DC1) and cast-in-place element (DC2) are 1.03 and 1.05, respectively, and the COVs are 8% and 10%, respectively. Therefore, the mean of dead load without the wearing surface is calculated as:

$$\mu_{DL} = \mu_{DC1} + \mu_{DC2} + \mu_{DW} = 1.03DC1_n + 1.05DC2_n + 0 = 387.5 \mu\varepsilon$$

The standard deviation of the total dead load is:

$$\sigma_{DL} = \sqrt{\sigma_{DC1}^2 + \sigma_{DC2}^2} = \sqrt{(V_{DC1}\mu_{DC1})^2 + (V_{DC2}\mu_{DC2})^2} = 25.7 \mu\varepsilon$$

$$V_{DL} = \frac{\sigma_{DL}}{\mu_{DL}} = 6.6\%$$

The live load uncertainty expressed in the COV is 18% to 20% from the LRFD specification calibration. Those uncertainties include variability within a site, site-to-site and sample data limitation. During the AASHTO LRFR calibration, two more uncertainty factors had been added: the dynamic amplification and the load distribution factor. For the in-service rating process, many of those uncertainties can be reduced, such as site-to-site, load distribution, and dynamic amplification uncertainties. On the other hand, there are also some additional uncertainties due to the SHM needing to be included for the analysis.

The live load model is based on the in-service strain data for this study. The COV of the maximum live load effect on a single girder should account for the following variations:

1. Measurement uncertainties ($V_{measure}$) due to gage installation, data-acquisition, sensor resolution, and temperature effect. The BDI strain transducer has a gage accuracy of 2%. Installation errors due to misalignment and gage location are assumed to be 5%. Thus, an estimation of errors for a properly calibrated system with temperature compensation can be estimated as $V_{measure} = \sqrt{(2\%)^2 + (5\%)^2} = 5.4\%$.

2. Strains to moments calculation uncertainties due to variability of concrete strength and modulus ($V_{conversion}$). As we discussed in Chapter 4, the actual girder concrete strength and plastic modulus are higher than the designed nominal values. Since the strength and moduli are estimated based on the laboratory test results, the statistical
parameters can be estimated as \( V = \sigma / \mu = 0.45 / 6.84 = 6\% \) (18\% for the LRFD calibration). The bias is \( \lambda = \mu / \mu_n = 6.84 / 5.0 = 1.36 \). (0.84 for the LRFD calibration). These values are much more representative of the actual material at this site than the values LRFD selected for calibration. However, the higher concrete strength has minor effects on the prestressed concrete capacity variability. Therefore, the higher concrete strength is ignored for the reliability study. Considering that the concrete is non-homogeneous, a\(, \) additional \( V_{\text{conversion}} = 5\% \) is estimated for the conversion of moments to strains.

3. Modeling and projection uncertainties (\( V_{\text{projection}} \)) including distribution modeling and projection uncertainties. For the 5-year rating period, \( V_{\text{projection}} = 4.5\% \) is obtained from prior calculation (Chapter 7.5.6).

4. Sample size uncertainties (\( V_{\text{data}} \)) associated with the use of one-year worth of data. The project of maximum live load is performed from data collected for 3 months, 6-months, 9 months, 11 months and 12 months. The convergence can be verified for the time period greater or equal to six months. The projection result difference from 6-month to 12 months resulted in difference of less than 2\%. Therefore, \( V_{\text{data}} = 2\% \) is selected for the sample size uncertainties.

In summary, the COV of mean applied live load can be calculated as:

\[
V_{LL} = \sqrt{V_{\text{measure}}^2 + V_{\text{conversion}}^2 + V_{\text{projection}}^2 + V_{\text{data}}^2}
\]

\[
= \sqrt{(5.4\%)^2 + (5\%)^2 + (4.5\%)^2 + (2\%)^2}
\]

\[
= 8.9\%
\]

To be conservative, the total live load \( V_{LL} \) is increased to 9\% for the reliability calculation. This is still much less than the 20\% used for the LRFD and LRFR calibrations.

Statistical data summary for moments are shown in Table 7-8.

<table>
<thead>
<tr>
<th>Variable</th>
<th>Mean ( \mu )</th>
<th>Standard Deviation ( \sigma )</th>
<th>Coefficient of Variation ( \text{COV} )</th>
<th>Bias ( \lambda )</th>
<th>Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Resistance - R</td>
<td>1127.36</td>
<td>50.73</td>
<td>4.50%</td>
<td>1.04</td>
<td>lognormal</td>
</tr>
<tr>
<td>Dead Load - DL</td>
<td>387.50</td>
<td>25.70</td>
<td>6.60%</td>
<td>1.04</td>
<td>Normal</td>
</tr>
<tr>
<td>Live Load - LL</td>
<td>140.67</td>
<td>10.55</td>
<td>9.00%</td>
<td>1.05</td>
<td>Gumbel</td>
</tr>
</tbody>
</table>

The total load mean, standard deviation, and COV are: \( \mu_S = 528.1\mu \epsilon \), \( \sigma_S = 28.6\mu \epsilon \), and \( \text{COV} = 5.4\% \) respectively.
To be consistent with the LRFD, reliability index calculation follows the process used in calibrating the design specification. Nowak assumed that the total load, \( S \), is a normal random variable and the resistance, \( R \), is a lognormal random variable. The modified first-order second-moment reliability index for this combination is expressed in the following equation:

\[
\beta = \frac{R_n \lambda_R (1 - 2V_R)[1 - \ln(1 - 2V_R)] - \mu_S}{\sqrt{[R_n V_R \lambda_R (1 - 2V_R)]^2 + \sigma_S^2}} = 10.9
\]

(7.23)

The calculated beta is significantly higher than the target \( \beta = 3.5 \) for inventory rating or \( \beta = 2.5 \) for operating rating. To meet the LRFD uniform reliability index concept, the live load factor or the rating factor can be adjusted.

### 7.9 IN-SERVICE TRAFFIC LOAD RATING AND LIVE LOAD FACTOR CALIBRATION

#### 7.9.1 In-service load (Legal and Routine Permit Load)

The target reliability index of 3.5 was set for the design, and 2.5 for the bridge evaluation. In an ideal case, a rating factor of 1.0 indicates the reliability equal to the target value 2.5. Otherwise, the live load factor should be adjusted to meet the target.

During calibration, a new live load factor based on the in-service traffic will be revised so that the bridge sections have a rating factor \( RF = 1.0 \) and will meet the reliability index \( \beta = 2.5 \) for a 5-year period. The load factor is then calculated as the function of the bias factor and the coefficient of variation. The safe margin expression becomes:

\[
Z = R - DL - (RF)LL
\]

\[
\beta = \frac{R_n \lambda_R (1 - 2V_R)[1 - \ln(1 - 2V_R)] - (\mu_{DL} + (RF)\mu_{LL})}{\sqrt{[R_n V_R \lambda_R (1 - 2V_R)]^2 + \sigma_S^2}}
\]

(7.25)

\[
RF = \frac{R_n \lambda_R (1 - 2V_R)[1 - \ln(1 - 2V_R)] - \beta \sqrt{[R_n V_R \lambda_R (1 - 2V_R)]^2 + \sigma_S^2} - \mu_{DL}}{\mu_{LL}}
\]

(7.26)

We can set the target reliability index \( \beta \) and back calculate the rating factor using Equation (7.26). This results in \( RF = 4.26 \) for \( \beta = 2.5 \) and 3.88 for \( \beta = 3.5 \), respectively. In comparison to the prior directory rating method of \( RF = 4.40 \) for the in-service loads, the two methods match well.

The live load factor can be adjusted by setting load rating factor equals to 1.0. The live load factor can be calculated as Eq. (7.27)
\[ \gamma_{LL} = \frac{\varphi_c \varphi_s \varphi_m R_n - \gamma_{DC} (D_{C1} + D_{C2})}{(RF)(L_L)} = 1.10 > 1.0 \] (7.27)

Therefore, \( \gamma_{LL} = 1.10 \) is recommended as the load factor for legal loads at this site to meet the target reliability index of 2.5.

### 7.9.2 Design Load

Although the permit loads are not separated from the legal load from the SHM data set for this study, future permits can use the results of this analysis, such as using the revised load distribution factor (LDF) directly.

The design live load model HL-93 reliability is calculated to direct the design load rating factors. To analyze the safety of a single member under the design load, the site-specific live load distribution factor LDF=0.25(s) and 0.45(m) are used. Although the LDF are not the actual mean value of the distribution factors, it is used directly to be on the conservative side. The revised load distribution factor is a normal distributed random variable with \( \mu=0.185 \) and \( V_{LDF}=18\% \) as discussed previously. To use the LDF directly, it is assumed that all the other statistical factors remain the same, the live load COV shall be revised to \( V_{LL}=20\% \) to include the live load distribution variance.

Following Eq. (7.23), the calculated \( \beta \) is 9.7 for the HL-93 truck. Therefore, the inventory and operating load rating factor for the HL-93 truck is 3.58 and 3.99, respectively. The calculated live load factors are 1.0 for both inventory and operating level. For the design loads, the design live load factor can be set as 1.05 conservatively.

### 7.9.3 Future Permit Load

Three Louisiana routine permit loads OFRD #1, 2, and 3 were included in the reliability analysis. The permit-truck load effects are less than HL-93 effect at this bridge site. Therefore, HL-93 operating is selected for the permit truck reliability analysis. To meet the target \( \beta \) at 2.5, the calculated live load calibrated factor is 1.0 based on the Eq.(7.27). For future permit loads, the live load factor can be set at 1.05 conservatively.

### 7.10 WIM AND SHM COMPARISON

The WIM data can be used to verify the truck configuration, speed, side-by-side possibility, and future rating predictions. The weekly means of GVW, GVW/AL, and GVW/# of axles (average axle weight) were compared to the measured strains as shown in Figure 7-30. GVW/AL and GVW/# of axles are closely correlated. Although it is unapparent, the degree of correlation between the strains and GVW/ALs is higher than the correlation with GVW, which indicates that the single tonnage type of load limitation may not be an efficient bridge posting method. A tonnage posting combined with either the truck length or number of axles is more beneficial.
7.11 BRIDGE POSTING

Bridges that do not have sufficient capacity under the design-load shall be load rated for legal loads to establish the need for load posting. Bridge posting involves the consideration of safety, economy, and public interests. A load-posted bridge shall be restricted for overload trucks. See Appendix A.5 for Louisiana legal load posting signs.

By the FHWA category classification, all three-axle vehicles are single unit vehicles. At this site, the majority of the four axles are the class 7 type single unit vehicles. See Appendix A.1 for FHWA Vehicle Classifications. These vehicles are compact and produce greater load effects on bridges. Therefore, three and four axle trucks are used to set the single unit limit, and five and more axle vehicles are used for setting the multi-unit limits.

The current Louisiana single tonnage posting method is a somewhat conservative and simple approach. The limits on the truck gross weight plus the number of axles will represent the live load effects more accurately. The proposed weight limit sign shall be as shown in Figure 7-31.

![Figure 7-30 Strain, GVW, GVW/AL and GVW/# of axles correlation](image)

![Figure 7-31 Recommended weight limit sign](image)
7.12 SUMMARY

This is an in-service response based rating method that utilized the SHM system. The US61 bridge was first rated directly based on long term SHM strain measurements and statistical projection. Next, the reliability methodology was used to derive the site-specific load rating factors and live load factors.

After filtering, sorting, and checking, the strain data were analyzed statistically to project the 5-year maximum load effects.

Three statistical projection methods were compared: Parent Tail Distribution method, Block Maxima method, and Block Maxima Tail method. The Normal, Simplified Gumbel, Gumbel and Frechet distributions were included for each projection method based on the maximum likelihood method. Gumbel parent tail Distribution method performed the best fit and was selected for this study. The 5-year projected load is 5% beyond the recorded one-year maximum value. The consistent results from the convergence check also validated the accuracy of the statistical projection methods.

This site shows little seasonal variation, and the projection result is insensitive to the amount of data. After six-month monitoring period, the projection results are consistent and the difference is within 2%.

The site-specific live Load Distribution Factor was developed based on the strain data. A revised flexure load distribution factor LDF = 0.25 (s) and 0.45 (m) is recommended to both one-lane and multi-lane traffic, which is much less than the AASHTO live load distribution factor.

The in-service live load reliability index is β=10.9, which is much higher than the target β =2.5. The bridge was first rated using the strain data directly. Then, re-rated based on the reliability analysis with a target β=2.5 directly. The design load was also rated based on the site LDF. See Table 7-9 for the load rating results.

The live load factors are recalibrated to meet the target β for future rating. The recommended live load factors for design load (inventory), legal loads, and permit loads are 1.05, 1.10 and 1.05, respectively.

The site-specific load rating result is 36% higher than the Level I rating (AASHTO live load distribution method with predefined trucks) for in-service loads. Note that the in-service loads include both the legal and permit loads. Therefore, the rating factor is the same for legal and permit loads.

After evaluating the correlation between strains and truck configurations, a new posting sign was recommended to include the truck gross weights and number of axles.
Table 7-9 Load rating summary based on the SHM (Level IV)

<table>
<thead>
<tr>
<th>Live Load</th>
<th>Level</th>
<th>Rating Factor</th>
<th>Controls</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Strain Directly</td>
<td>Reliability Directly</td>
</tr>
<tr>
<td>HL-93</td>
<td>Inventory</td>
<td>2.02*</td>
<td>N/A</td>
</tr>
<tr>
<td>HL-93</td>
<td>Inventory</td>
<td>3.17*</td>
<td>N/A</td>
</tr>
<tr>
<td>HL-93</td>
<td>Inventory</td>
<td>2.78*</td>
<td>3.58</td>
</tr>
<tr>
<td>Legal</td>
<td>Legal</td>
<td>4.40</td>
<td>4.26</td>
</tr>
<tr>
<td>Permit</td>
<td>Routine</td>
<td>4.40</td>
<td>4.26</td>
</tr>
</tbody>
</table>

*Rating Factor is based on the NDT result
CHAPTER 8  SUMMARY AND CONCLUSIONS

A pre-stressed 3-span continuous concrete girder bridge was selected to be load rated by four different levels of rating methods in order to increase the accuracy of load rating. At the beginning, the selected bridge was rated with the traditional Level I method - the LRFD approximation method for the as-designed and the as-built rating; followed by the Level II - finite element rating and Level III - nondestructive live load testing and FE model calibration. At the end, the Level IV - in-service rating based on WIM and SHM data and reliability analysis were conducted.

Integrating WIM, SHM, and component reliability analysis is an important topic for data interpretation and advanced load rating. In this context, it is also important to consider the uncertainties in data analysis and prediction of future performance. With accurate predictions, it would be possible to more accurately evaluate future overload permit, establish bridge load posting and provide a better assistant for bridge management. In order to achieve this, the integration of novel techniques offered by SHM and analytical and numerical methods is required. After the reliability analysis, the bridge load factors for each limit state were adjusted to meet the target $\beta_T$ of 3.5 and 2.5 for inventory and operating evaluation levels, respectively, and the new critical rating factor can be established.

The SHM data is a continuous random variable. Based on the observed response, the original statistical model can be established. With time, the new data can be incorporated into the models to reduce epistemic uncertainty to better describe the current structural performance.

8.1 RATING FACTOR SUMMARY

Table 8-1 summarizes the load rating factors for different rating levels. The as-designed rating was calculated as the baseline for comparison. All other rating factors were compared with the as-designed rating and the results are shown in Table 8-2.

Level I to Level III ratings are developed for pre-defined trucks. The Level IV rating is based on the in-service traffic; therefore, the design inventory and operating rating based on WIM and Strain method that are using the calibrated FEM based on NDT.

Compare with Level I as-designed rating, the Legal flexure ratings are increased by 10% for Level II, 43% for Level III and 36% for Level IV. Three different methods are experienced for Level IV rating. First, the bridge was rated based on WIM gross weight, which resulted in the highest increase of 65%. Next, the bridge was rated based on in-service SHM strain data. The Level IV ratings results either based on strains directly or based on reliability analysis are increased by 40% and 36%, respectively.

The NDT method, WIM method, and SHM method provided very consistent results. These methods are recommended for future bridge evaluation and health monitoring.

The WIM method is recommended for truck mapping and truck routs study.
<table>
<thead>
<tr>
<th>Load Type</th>
<th>Rating Level</th>
<th>State Limits</th>
<th>Level I</th>
<th>Level II</th>
<th>Level III</th>
<th>Level IV</th>
</tr>
</thead>
<tbody>
<tr>
<td>HL-93</td>
<td>Inventory</td>
<td>Service-III Tensile</td>
<td>1.34</td>
<td>1.49</td>
<td>2.12</td>
<td>2.02</td>
</tr>
<tr>
<td>HL-93</td>
<td>Inventory</td>
<td>Strength-I Shear</td>
<td>1.42</td>
<td>1.54</td>
<td>1.78</td>
<td>3.17</td>
</tr>
<tr>
<td>HL-93</td>
<td>Inventory</td>
<td>Strength-I Flexure</td>
<td>1.96</td>
<td>1.96</td>
<td>2.18</td>
<td>2.78</td>
</tr>
<tr>
<td>HL-93</td>
<td>Operating</td>
<td>Strength-I Flexure</td>
<td>2.53</td>
<td>2.53</td>
<td>2.83</td>
<td>3.61</td>
</tr>
<tr>
<td>Legal</td>
<td>Legal</td>
<td>Strength-I Flexure</td>
<td>3.14</td>
<td>3.14</td>
<td>3.45</td>
<td>4.49</td>
</tr>
<tr>
<td>Permit</td>
<td>Routine</td>
<td>Strength-I Flexure</td>
<td>3.11</td>
<td>3.11</td>
<td>3.41</td>
<td>4.43</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Rating Level</th>
<th>State Limits</th>
<th>Level I</th>
<th>Level II</th>
<th>Level III</th>
<th>Level IV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Distribution Factors</td>
<td>FEM</td>
<td>NDT</td>
<td>WIM</td>
<td>SHM</td>
<td>Strain</td>
<td>β</td>
</tr>
<tr>
<td>5.0 ksi</td>
<td>6.25 ksi</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Rating factor is based on NDT
Table 8-2 Load rating factors compared with Level I as-designed rating

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Rating Level</th>
<th>State Limits</th>
<th>Level I</th>
<th>Level II</th>
<th>Level III</th>
<th>Level IV</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Distrib</td>
<td>FEM</td>
<td>NDT</td>
<td>WIM</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Strain</td>
<td></td>
<td></td>
<td>*β</td>
</tr>
<tr>
<td>HL-93</td>
<td>Inventory</td>
<td>Service-III Tensile</td>
<td>1.00</td>
<td>1.11</td>
<td>1.58</td>
<td>1.51</td>
</tr>
<tr>
<td>HL-93</td>
<td>Inventory</td>
<td>Strength-I Shear</td>
<td>1.00</td>
<td>1.08</td>
<td>1.25</td>
<td>2.23</td>
</tr>
<tr>
<td>HL-93</td>
<td>Inventory</td>
<td>Strength-I Flexure</td>
<td>1.00</td>
<td>1.00</td>
<td>1.11</td>
<td>1.42</td>
</tr>
<tr>
<td>HL-93</td>
<td>Operating</td>
<td>Strength-I Flexure</td>
<td>1.00</td>
<td>1.00</td>
<td>1.12</td>
<td>1.43</td>
</tr>
<tr>
<td>Legal</td>
<td>Legal</td>
<td>Strength-I Flexure</td>
<td>1.00</td>
<td>1.00</td>
<td>1.10</td>
<td>1.43</td>
</tr>
<tr>
<td>Permit</td>
<td>Routine</td>
<td>Strength-I Flexure</td>
<td>1.00</td>
<td>1.00</td>
<td>1.10</td>
<td>1.43</td>
</tr>
</tbody>
</table>

*Rating factor is based on NDT
8.2 SITE-SPECIFIC LIVE LOAD BASED ON WIM

Truck models can vary from site to site. The site-specific truck models were developed instead of the national pre-defined trucks to represent the local trucks. The projection factor were also developed using the statistical projection method. The 5-year maximum GVW is close to the one year recorded maximum value. The 75-year projection GVW is 8% higher than the one year recorded maximum value.

The site-specific live load factors was re-calibrated as $\gamma_{\text{site}} = 1.16$ for legal load and permit load based on the target reliability index of $\beta = 2.5$.

8.3 RATING BASED ON SHM

The site-specific live load Distribution Factor was developed based on the SHM strain data. The recommended site-specific live load flexure distribution is 0.25 of total load for single lane, which is 50% of the AASHTO LRFD recommended distribution factor. The multi-lane live load flexure distribution is 0.45, which is 70% of the AASHTO LFD.

The live load factors were re-calibrated based on the target reliability. The recommended live load factors for design load (inventory), legal loads, and permit loads are 1.05, 1.10 and 1.05, respectively.

8.4 STATISTICAL PROJECTIONS

The bridge evaluation period is generally longer than the observation period. The extreme value theory is used to project the future load effects.

The Parent Tail Distribution method, the Block Maxima method, and the Tail Block Maxima method all can provide close projection results. The Gumbel and Normal distribution fit the best for the peak strains based maximum likelihood analysis. Therefore, the Parent Tail Gumbel Distribution method was approved to be suitable for this study.

8.5 CONCLUSIONS

The AASHTO specifications have been developed for structure design, not for bridge assessment. It is apparent that considerable conservatism exists in the traditional load design and rating methods. This study indicates that the application of WIM, SHM and probabilistic assessment of individual structures provides more accurate bridge evaluations which can extend the bridge service life and reduce the posting cases.

The traditional approximate live load distribution method is a simplified evaluation method for commonly used bridge types. The result can provide a baseline for bridge evaluation.

The refined analysis method (FEM) provides more accurate bridge modeling, but still cannot fully represent the bridge condition and specific live loads.
A field-testing based rating can more accurately represent the true behavior and load distribution of the structure, but this method still could not count for the site-specific loads.

WIM can provide site-specific live load model for bridge evaluation, but by itself it cannot represent the bridge behavior.

The SHM probability based rating with the WIM validation can provide the most accurate evaluation method. The SHM in-situ rating is more accurate and can reduce the bridge posting cases and allow for more overload permit trucks.

The response-based reliability method selects the actual bridge as the evaluation model and in-service traffic as live load. This systematic rating method improves the load ratings through significantly reducing the live load uncertainties, such as load distribution, dynamic impact, and multiple presences. As such, this in-service rating method evaluates the bridge in a fully probabilistic manner and maintains all the bridges at the same reliability level.

### 8.6 FUTURE RESEARCH

This study only considered the element reliability for bridge load rating. A system reliability analysis accounts for the consequences of failure, load sharing, and load redistribution will provide more accurate bridge behavior and load rating for the future.

The in-service rating method is limited to flexure for this study. This method can be extended to other limit states, such as shear and service limit states.

Instead of using the load factor rating method for important bridges, with enough instrumentation data, the reliability index (β) can be used directly for future bridge evaluation and monitoring.

Bridge instrumentation has become more common in recent decades. With enough instrumentation, we can extend this more accurate rating method to a region of bridges or even to all bridges.

The reliability index is a time variant. If we consider the bridge strength lost as a time dependent variable, a continuous bridge evaluation can be implemented to extend the bridge’s service life.
REFERENCES


## APPENDIX A  WEIGH-IN-MOTION DATA AND STATISTICAL ANALYSIS

### A.1  FHWA Vehicle Classifications

<table>
<thead>
<tr>
<th>Class</th>
<th>Description</th>
<th>Illustration</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Motorcycles</td>
<td><img src="image1.png" alt="Motorcycle" /></td>
</tr>
<tr>
<td>2</td>
<td>Passenger cars</td>
<td><img src="image2.png" alt="Passenger car" /></td>
</tr>
<tr>
<td>3</td>
<td>Four tire, single unit</td>
<td><img src="image3.png" alt="Four tire" /></td>
</tr>
<tr>
<td>4</td>
<td>Buses</td>
<td><img src="image4.png" alt="Buses" /></td>
</tr>
<tr>
<td>5</td>
<td>Two axle, six tire, single unit</td>
<td><img src="image5.png" alt="Two axle" /></td>
</tr>
<tr>
<td>6</td>
<td>Three axle, single unit</td>
<td><img src="image6.png" alt="Three axle" /></td>
</tr>
<tr>
<td>7</td>
<td>Four or more axle, single unit</td>
<td><img src="image7.png" alt="Four or more" /></td>
</tr>
<tr>
<td>8</td>
<td>Four or less axle, single trailer</td>
<td><img src="image8.png" alt="Four or less" /></td>
</tr>
<tr>
<td>9</td>
<td>5-Axle tractor semitrailer</td>
<td><img src="image9.png" alt="5-Axle" /></td>
</tr>
<tr>
<td>10</td>
<td>Six or more axle, single trailer</td>
<td><img src="image10.png" alt="Six or more" /></td>
</tr>
<tr>
<td>11</td>
<td>Five or less axle, multi trailer</td>
<td><img src="image11.png" alt="Five or less" /></td>
</tr>
<tr>
<td>12</td>
<td>Six axle, multi-trailer</td>
<td><img src="image12.png" alt="Six axle" /></td>
</tr>
<tr>
<td>13</td>
<td>Seven or more axle, multi-trailer</td>
<td><img src="image13.png" alt="Seven or more" /></td>
</tr>
</tbody>
</table>
A.2 Louisiana Legal Vehicles

LA Type 3 \[ \text{GVW} = 41 \text{ kips} \]

LA Type 3-S2 \[ \text{GVW} = 73 \text{ kips} \]

AASHTO Type 3 \[ \text{GVW} = 80 \text{ kips} \]

LA Type 6 \[ \text{GVW} = 80 \text{ kips} \]

LA Type 8 \[ \text{GVW} = 88 \text{ kips} \]
Figure B.6 - Bridge Posting Loads for Single Unit Trucks That Meet Federal Bridge Formula B.
A.3 Louisiana Annul Permit Vehicles

- OFRD #1 Unit Weight = 132.7 kips
- OFRD #2 Unit Weight = 142.1 kips

A.4 Louisiana Single Trip Permit Vehicles

- OVL#1 Unit Weight = 200.0 kips
- OVL#2 Unit Weight = 260.0 kips
- OVL#3 Unit Weight = 240.0 kips
A.5 Louisiana Legal Load Posting Signs

The weight limit sign (R12-5) limits the gross weight of all single truck vehicles to the displayed first number and the gross weight of all vehicle combinations to the displayed second number.
A.6  GVW Histogram for Vehicles with Different Number of Axles

Figure A-1 GVW histogram for 3-axle trucks

Figure A-2 GVW histogram for 4-axle trucks
Figure A-3 GVW histogram for 5-axle trucks

Figure A-4 GVW histogram for 6-axle trucks
Figure A-5 GVW histogram for 7-axle trucks

Figure A-6 GVW histogram for 8-axle trucks
Figure A-7 GVW histogram for 9-axle trucks

Figure A-8 GVW histogram for 11-axle trucks
APPENDIX B   SHM DATA AND STATISTICAL DISTRIBUTION

B.1  Strain Histogram

Figure B-1 Strain 15 Histogram

Figure B-2 Strain 16 Histogram
Figure B-3 Strain 18 Histogram

Figure B-4 Strain 20 Histogram
Figure B-5 Strain 21 Histogram

Figure B-6 Strain 22 Histogram
Figure B-7 Strain 01 Histogram

Figure B-8 Strain 04 Histogram
Figure B-9 Strain 06 Histogram

Figure B-10 Strain 08 Histogram
B.2 Strain Statistical Distribution and Projection

The extreme values theory is used for the extrapolation of data into the required return period. After comparison, the Block Maxima method and Gumbel distribution were adopted for this study. This appendix presents all of the girder strain statistical distributions and the five-year return period projections based on the weekly maximum strains.

The weekly maximum strains are plotted on the Gumbel plot. The shaded area indicates the 95% confidence interval regions of the cumulative distribution estimate.

Most of the girder strains follow the Gumbel distribution with one exception, strain 21. Strain 21 is at span-1 and on girder number 5 under the passing lane. The strain 21 statistical
distribution follows the Normal distribution much better. For consistence, only the Gumbel plots are shown here.

(a) Gumbel Plot

(b) CDF

Figure B-13 Strain 15 Gumbel Plot and CDF

(a) Gumbel Plot

(b) CDF
Figure B-14 Strain 16 Gumbel Plot and CDF

(a) Gumbel Plot

(b) CDF

Figure B-15 Strain 18 Gumbel Plot and CDF

(a) Gumbel Plot

(b) CDF
Figure B-16 Strain 20 Gumbel Plot and CDF

(a) Gumbel Plot

(b) CDF

Figure B-17 Strain 21 Gumbel Plot and CDF

(a) Gumbel Plot

(b) CDF

Figure B-18 Strain 22 Gumbel Plot and CDF
Figure B-19 Strain 01 Gumbel Plot and CDF

(a) Gumbel Plot

(b) CDF

Figure B-20 Strain 04 Gumbel Plot and CDF
(a) Gumbel Plot

(b) CDF

Figure B-21 Strain 06 Gumbel Plot and CDF

(a) Gumbel Plot

(b) CDF

Figure B-22 Strain 08 Gumbel Plot and CDF
Figure B-23 Strain 09 Gumbel Plot and CDF

(a) Gumbel Plot

(b) CDF

Figure B-24 Strain 10 Gumbel Plot and CDF

(a) Gumbel Plot

(b) CDF
B.3 Moment Live Load Distribution Factors (LDF) Histogram

Figure B-25 Strain 15 Moment LDF Histogram

Figure B-26 Strain 16 Moment LDF Histogram
Figure B-27 Strain 18 Moment LDF Histogram

Figure B-28 Strain 20 Moment LDF Histogram
Figure B-29 Strain 21 Moment LDF Histogram

Figure B-30 Strain 22 Moment LDF Histogram
Figure B-31 Strain 1 Moment LDF Histogram

Figure B-32 Strain 4 Moment LDF Histogram
Figure B-33 Strain 6 Moment LDF Histogram

Figure B-34 Strain 8 Moment LDF Histogram
Figure B-35 Strain 9 Moment LDF Histogram

Figure B-36 Strain 10 Moment LDF Histogram
VITA

Dana Feng was born in the Harbin, China; a city that is notable for its beautiful ice sculpture festivals. She received her Bachelor of Science degree in Civil Engineering from Harbin Institute of Technology, China and her Master of Science degree in Civil Engineering from McNeese State University, Louisiana. Ms. Feng is currently working as the Louisiana State Bridge Load Rating Engineer for Louisiana Department of Transportation and Development. Ms. Feng joined the Department of Civil and Environmental Engineering in its Ph.D. program at Louisiana State University in 2008. She anticipates graduating with her Ph.D. degree in December 2016.