IMPACT OF OVERWEIGHT TRUCKS ON SERVICE LIFE OF
BRIDGES

By

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Written under the direction of

Dr. Hani H. Nassif, Ph.D.

And approved by

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ABSTRACT OF THE DISSERTATION

IMPACT OF OVERWEIGHT TRUCKS ON SERVICE LIFE OF BRIDGES

by PENG LOU

Dissertation Director:
Dr. Hani H. Nassif, Ph.D.

Highway agencies are responsible for the optimal expenditure of taxpayer dollars allocated to highway infrastructure. Truck size and weight are regulated by federal legislation and every state highway agency has its own legal load limits. Over the last two decades, both the frequency and weight of overweight trucks has kept increasing. Although the AASHTO Load and Resistance Factored Design (LRFD) Bridge Design Specifications mandates a design life of 75 years, the actual service life of bridges is lower and varies from one bridge site to another. Additionally, state agencies issue permits for trucks with gross vehicle weights that are above legal load limits. However, the effect of overweight trucks on the service life of bridge components is not explicitly quantified.

This dissertation presents a rational approach to investigate the impact of truck loads on bridges in New Jersey through the utilization of bridge inspection reports, truck weight-in-motion (WIM) data, and the National Bridge Inventory (NBI) database. Actual bridge deterioration modes were identified from their respective inspection reports. Based on the condition ratings from NBI, the expected service life for each bridge component
on various highways were estimated. In addition, WIM data in New Jersey were used to extract the loading on bridges. For bridge decks and prestressed concrete (P/C) girders, the correlation between the expected service life and truck loadings was performed and prediction functions for service life were proposed. For steel bridge girder, predicted service life was calculated through the remaining fatigue life assessment. Lastly, Bridge Life Cycle Cost Analysis (BLCCA) was conducted using two contrasting scenarios, one with and the other without overweight trucks, to quantify economic impact of overweight trucks on bridges. The results show that deterioration mode of prestressed concrete (P/C) girders was the corrosion near the beam-ends induced by cracking and spalling while the deterioration mode of reinforced concrete deck was the punching shear failure. Overall, P/C girders have better performances than steel girders. During the lifetime of the bridge, the deterioration of P/C girders would be accelerated once cracking is initiated. The expected service life of P/C bridges was greatly affected by the condition of the bridge deck. Lastly, the results indicated that overweight trucks caused more damage on NJ state highways compared with interstate highways due to a larger proportion of overweight trucks, heavy wheel loads from overweight trucks, and fewer axles per truck.
DEDICATION

To My Parents, I love you.
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CHAPTER 1

INTRODUCTION

1.1. Background

Infrastructure systems, such as highway bridges, constitute a major part of the national investment and are critical for the mobility of our society as well as its economic growth and prosperity. The degree of bridge deterioration due to overweight trucks, as well as exposure to the natural elements, is relatively high. However, the impact of truck loads and frequency on the service life of bridge components are not explicitly quantified. In addition, the increasing number of permits issued for overweight trucks has had a significant impact on the highway infrastructure, most notably on bridges.

The Federal Bridge Formula (FHWA 2015), shown in Eq. 1.1, was enacted by Congress in 1975 and updated in 2006. It has been used by many local agencies to legislate and enforce the legal truck weight limits.

\[
W = 500 \times \left[ \frac{LN}{(N-1)} + 12N + 36 \right] \quad \text{Eq. 1.1}
\]

where

\( W \) = overall gross weight on any group of two or more consecutive axles to the nearest 500 pounds,
$L =$ distance in feet between the outer axles of any group of two or more consecutive axles,

$N =$ the number of axles in the group under consideration.

In addition to the Federal Bridge Formula, any single axle is limited to 20,000 pounds, a tandem is limited to 34,000 pounds, and the gross vehicle weight (GVW) is limited to 80,000 pounds.

Moreover, due to the increasing truck traffic, both in weight and in frequency on the highway network, there is a need to correlate the effect of overweight trucks with bridge deterioration. Figure 1.1 shows the changes of average daily truck traffic (ADTT) over the last twenty years on I-195 (Site 195) and I-287 (Site 287) in New Jersey as well as deck condition rating over time for a bridge near the WIM Site 195. The ADTT were collected from WIM sites and the deck condition ratings were taken from NBI database. It is clearly shown that while ADTT kept increasing over a period of 20 years, the bridge deck rating is decreasing over the same period.

![Figure 1.1. ADTT of interstate highways and deck deterioration](image-url)
1.2. Motivation

New Jersey Department of Transportation's Freight Services has undertaken a major study to collect and process data that is essential for monitoring large trucks (i.e., truck length greater than 102”) and their movement on side routes in various cities. To achieve this objective, data from various existing permanent fixed Weigh-In-Motion (WIM) stations were collected and processed to monitor the volume and pattern of large truck movements in NJ. In addition, NJDOT started collecting electronic data on special permits for overweight trucks. The collection of unbiased data and monitoring of truck movement, weight data, and axle configuration, provide the basis for understanding their impact on the State’s highway infrastructure. The implications on the cost of maintaining safe roads and their integrity are obvious. Accordingly, the State will expend significant effort and resources on the maintenance and repair of roadway infrastructure that will be adversely affected due to a growth in the frequency and an increase in the number of heavy and overweight trucks over the last decade. A great deal is known about the factors that causes damage in roads, and bridges, however, what is needed is to implement this and provide a reasonable quantification of damage cost to bridges in network to help maintain and manage NJ’s infrastructure systems. Moreover, due to the high bridge density in New Jersey highway network, a rational damage cost estimation approach would help local agency to establish a reasonable permit fee structure. Table 1.1 lists top ten bridge density in United States.

In New Jersey, the weight regulation follows the Federal Bridge Formula. Although truck weight is regulated by legislature, permits are issued for trucks with
irregular dimensions and heavy loads. There is a need to evaluate the effect of loads exceeding legal limits on bridge components.

Table 1.1 Bridge density in United States

<table>
<thead>
<tr>
<th>No.</th>
<th>State</th>
<th>No. of Bridge (2014 NBI Data)</th>
<th>Area (mi²)</th>
<th># of bridge per mi²</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>District of Columbia</td>
<td>253</td>
<td>68</td>
<td>3.707</td>
</tr>
<tr>
<td>2</td>
<td>Connecticut</td>
<td>4218</td>
<td>5544</td>
<td>0.761</td>
</tr>
<tr>
<td>3</td>
<td>New Jersey</td>
<td>6609</td>
<td>8723</td>
<td>0.758</td>
</tr>
<tr>
<td>4</td>
<td>Puerto Rico</td>
<td>2304</td>
<td>3515</td>
<td>0.655</td>
</tr>
<tr>
<td>5</td>
<td>Ohio</td>
<td>26986</td>
<td>44825</td>
<td>0.602</td>
</tr>
<tr>
<td>6</td>
<td>Indiana</td>
<td>19019</td>
<td>36418</td>
<td>0.522</td>
</tr>
<tr>
<td>7</td>
<td>Rhode Island</td>
<td>766</td>
<td>1545</td>
<td>0.496</td>
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</tr>
<tr>
<td>10</td>
<td>Tennessee</td>
<td>20077</td>
<td>42144</td>
<td>0.476</td>
</tr>
</tbody>
</table>

1.3. Research Significance

Highway agencies are responsible for the optimal expenditure of taxpayer dollars allocated to highway infrastructure. Truck size and weight are regulated by federal legislation and every state highway agency has its own legal load limits. Additionally, state agencies issue permits for trucks with gross vehicle weights that are above legal load limits. However, the effect of overweight trucks on the service life of bridge structure, is not explicitly quantified. The developed methodology will allow allocating costs to different trucks in a manner that commensurate with their respective contributions to bridge damage and develop a “weight-distance” based damage cost for overweight trucks. It is expected from this dissertation, state agencies can be placed in a better position to optimize the exiting road use charging system, update permit fee
structures for overweight trucks, and ultimately, preserve investments in highway infrastructure without sacrificing the competitive position of the state.

1.4. Objectives

The primary objective of this dissertation is to develop deterioration models for various bridge components with the emphasize on the effect of truck loadings and quantify the damage cost due to overweight trucks and. Based on the research, damage cost to bridge structure caused by overweight trucks with known Gross Vehicle Weight (GVW), axle configuration, distance traveled can be quantified. The objectives of this dissertation can be classified into following:

1. Review the state of art and state of practice of study related to overweight trucks from various states and related methodologies regarding bridge deterioration;
2. Develop deterioration model for RC bridge decks;
3. Develop deterioration model for P/C girders;
4. Develop deterioration model for steel girders;
5. Perform Bridge Life Cycle Cost Analysis to obtain the economic impact of overweight trucks on bridges.

1.5. Organization of the Thesis

The dissertation contains seven chapters. This chapter serves as an introduction of the dissertation outlining the problem statement and statement of objectives.
Chapter two presents a comprehensive literature review of various issues and available research experiences in various related topics, including a brief introduction on highway truck regulations, study of overweight trucks from various states, structural reliability analysis, fatigue evaluation of bridges, deck deterioration models, deterioration models of P/C girder and Bridge Life Cycle Cost Analysis (BLCCA).

Chapter three provides a detailed implementation of Structural Health Monitoring (SHM) system and data collecting procedures. This will help validate and calibrate the analytical bridge models.

Chapter four describes finite element modeling (FEM) of selected bridges and the remaining fatigue life assessment for steel bridge girders. This will serve as an input for BLCCA.

Chapter five presents the deterioration models for bridge components, including RC decks and P/C girder. The deterioration models of girders will provide the estimated service life as another input for BLCCA.

Chapter six presents the results from BLCCA. Based on cost analysis, the economic impact of overweight trucks on bridge will be quantified. Damage cost of trucks with different GVW, and axle configuration will be quantified.

Chapter seven contains the summary and conclusions of this dissertation.
CHAPTER 2

LITERATURE REVIEW

2.1. Highway Truck Weight Regulations

Bridges on the Interstate Highway System were designed to carry a wide variety of vehicles and their expected loads. However, in 1950s and later, as the tradeoff between the concern over the deterioration of the existing bridge infrastructure and the pressure of increasing truck weight limitations from the trucking industry, the weight-to-length ratio of a vehicle crossing a bridge was limited by the Federal Bridge Formula shown in Eq. 1.1 which is enacted by Congress in 1975. This was introduced as a program called “Federal-Aid Highway Act” which restricted the gross vehicle weight and the weights of different axle types. This formula was calibrated to keep girder overstressing of HS20 bridges under 5 percent and of HS15 bridges under 30 percent.

Federal Highway Administration (FHWA) developed another weight formula shown in Figure 2.1, called TTI formula (Noel et al. 1985). The formula has the same criterion as Bridge Formula. The formula is given as:

\[
W = (34 + L)\ 1000 \text{ lbs} \quad \text{For } 8 \text{ ft} < L < 56 \text{ ft} \\
W = (62 + L/2)\ 1000 \text{ lbs} \quad \text{For } L > 56 \text{ ft}
\]

Eq. 2.1

Later, James et al. (James et al. 1986) proposed a modification on the TTI formula that reduced the limits on axle loads while allowing higher gross weights. This modified
TTI formula only limits the stress of HS20 bridges and not HS15 bridges. The formula is given as:

\[ W = 26000 + 2000L \quad \text{for } L < 23 \text{ft} \]
\[ W = 62000 + 500L \quad \text{for } L > 23 \text{ft} \]

Eq. 2.2

As demonstrated in Figure 2.1, the modified TTI formula allows higher weights for long vehicles, and tandem and tridem axle groups compared to the Bridge Formula. The modified TTI formula has higher limitations than TTI formula when vehicles or axle group longer than 8 ft.

![Comparison of different axle-group-weight formulas](image)

**Figure 2.1.** Comparison of different axle-group-weight formulas

In New Jersey, the vehicle dimensional and weight limitations is stated in New Jersey Statutes Annotated (N.J.S.A) 39:3-84. All oversize and overweight permits are governed by Rules and Regulations promulgated by the Chief Administrator, NJ Motor Vehicle Commission at N.J.A.C. 13:18-1.1, et seq. The weight limitations in New Jersey follows the Bridge Formula.
2.2. Studies on the Impact of Overweight Trucks

In 1993, Nowak et al. started the study on the effect of truck loads on bridges. Lately, the impact of overweight trucks on highway infrastructure mainly in bridges and pavements has caused lots of concern in North America. Various state agencies initiated studies on the impact of overweight trucks on infrastructure systems due to the increasing number of permit issued for overweight trucks. Simple analysis methods are not well established for local agency to estimate the impact on bridges subjected to overloads. Some state DOTs already initiated studies to help quantifying either structural or economic impact of overweight trucks. These states include Connecticut, Louisiana, Indiana, Ohio, Wisconsin, South Carolina, New York State, and New Jersey. A detailed literature review was conducted on the impact of overweight trucks, summarized in Table 2.1 for brevity. From the previous studies the reasons that estimated damage cost to bridges by overweight trucks varies from state to state could be mainly attributed to the following: 1) bridge components considered, 2) deterioration models of bridge components used, 3) regional loading and environmental circumstance, and 4) cost analysis method. One of the major shortcomings of the majority of these studies is that the damage was assessed based on assumed deterioration modes, which might not be the case in reality. For example, most of the previous studies regarded the fatigue of the prestressing tendon as the deterioration mode for P/C girder bridges. In addition, most of the previous studies focused only on the accumulated damage of girders without considering bridge decks. However, the effect of overweight trucks on deck might be unexpectedly high due to the highest rate of deterioration and high frequency of rehabilitation.
Table 2.1 Summary of studies on overweight trucks in US

<table>
<thead>
<tr>
<th>State</th>
<th>Authors and Year</th>
<th>Infrastructure considered</th>
<th>Bridge components</th>
<th>WIM</th>
<th>Cost Analysis</th>
<th>Life Cycle Cost Analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td>Connecticut</td>
<td>Culmo et al. 2004</td>
<td>Bridge</td>
<td>Girder</td>
<td>No</td>
<td>No</td>
<td>N.A.</td>
</tr>
<tr>
<td>Louisiana</td>
<td>Roberts et al. 2005</td>
<td>Bridge, Pavement</td>
<td>Girder, Deck</td>
<td>No</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Indiana</td>
<td>Reisert and Bowman 2006</td>
<td>Bridge</td>
<td>Girder</td>
<td>Yes</td>
<td>No</td>
<td>N.A.</td>
</tr>
<tr>
<td>Ohio</td>
<td>Swearingen et al. 2009</td>
<td>Bridge, Pavement</td>
<td>Girder</td>
<td>No</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td>Wisconsin</td>
<td>Bae and Oliva 2009 and 2012</td>
<td>Bridge</td>
<td>Girder, Deck</td>
<td>No</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td></td>
<td>Lin et al. 2012</td>
<td>Bridge</td>
<td>Deck</td>
<td>No</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td></td>
<td>Adams et al. 2013</td>
<td>Pavement</td>
<td>N.A.</td>
<td>No</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td>South Carolina</td>
<td>Chowdhury et al. 2013</td>
<td>Bridge, Pavement</td>
<td>Girder</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td>NYS</td>
<td>Ghosn et al. 2015</td>
<td>Bridge, Pavement</td>
<td>Girder</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td>NJ</td>
<td>Nassif et al. 2016</td>
<td>Bridge, Pavement</td>
<td>Girder, Deck</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
</tbody>
</table>

Canada

1995-Fatigue Based Methodology for Managing Impact of Heavy-Permit Trucks on Steel Highway Bridges (Canada)

As one of earliest study in this topic, Dicleli and Bruneau conducted a study on impact of heavy-permit trucks on steel highway bridges in 1995 (Dicleli, Bruneau, 1995). Both ultimate and cumulative effect of the overloads were investigated. It was found that the selected bridges had adequate ultimate capacity to accommodate the overweight. However, the cumulative fatigue damage was the concern for the large number of passing...
overweight trucks. In addition, the author stated that the concept of infinite fatigue life cannot be relied on due to the involvement of overweight trucks and a reasonably large number of special permits could only cause a small reductions in fatigue life.

**Connecticut State**

*2004-Behavior of Steel Bridges under Superload Permit Vehicles*

M.P. Culmo et al. presented a study on the behavior of selected steel bridges under specific superload permit trucks. Six specific superload trailer types were discussed in their effect on bridges in terms of bridge span length, lateral load distribution, and dynamic load allowance. Strain data from testing was obtained and compared to the response from structural analysis. The results showed that a conventional line girder analysis can be employed with minor adjustments in assumption for analyze the effect of superload has a highway bridge. Impact can be taken as zero for trucks crossing at walking speed.

**Indiana State**

*2005-Fatigue of older bridges due to overweight and oversized loads (Indiana)*

In 2005, J.A. Reisert et al. reported a study on the fatigue of older bridges in Northern Indiana due to overweight and oversized loads. Field measurements of truck axle loads spectrum and bridge response were collected. Both two dimensional and three dimensional models were built to predict the structural response under identified truck loads from WIM data. Based on WIM database, new 3-axle and 4-axle fatigue trucks were developed. Moreover, a statistical databased of resistance parameters was incorporated. Then fatigue evaluation regarding estimating remaining fatigue life of selected bridge was obtained on the heavy weight corridor.
Louisiana State

2005-Effects of Hauling Timber, Lignite Coal, and Coke Fuel on Louisiana Highways and Bridges (Louisiana)

F.L. Roberts et al. presented a study on the effect of overweight permitted vehicle hauling timber, lignite coal, and coke fuel on highway pavements and bridges. Three loading scenarios were used in the analysis, 80,000 lbs (interstate weight limit), 86,600 lbs or 86,600 lbs (permit practice of the time), 100,000 lbs (proposed permit weight limit). The analysis was performed on the identified highway routes and bridges that commodities hauling was on, including approximately 1,400 control sections and 2,800 bridges. Results indicate that permit fees paid by timber trucks should increase from the current $10 per year to around $346/year/truck for a GVW of 86,600 lb. when axles are equally loaded and $4,377/year/truck if 48-kip axle load are permitted. The current permit fee for lignite coal should remain at current levels. The legislature should not consider raising the GVW level to 100,000 lb. because the pavement overlay costs double over those at 86,600 lb. GVW and the bridge repair costs become significant. In many cases, the bridge costs per passage of a loaded truck amount to $8.90 meaning that the cost of bridge damage per truck per year can easily exceed $3,560.

Ohio State

2009-Impacts of Permitted Trucking on Ohio’s Transportation System and Economy (Ohio)

In 2009, Ohio Department of Transportation performed a study of the impacts of permitted trucking on Ohio’s transportation system and economy. In the study, a three tiered approach was employed for the pavements cost. The basic cost is shared by all
users. Structural costs are shared by all trucks in accordance with their impact and overweight costs were entirely attributed to permitted vehicles. The resulting allocations employing this method results in a $122 million allocation to overweight vehicles annually. For bridge, the study used the incremental method to quantify the damage directly in dollar terms. The bridge impact costs total $22 million annually. The annual trip and trip length were estimated as 24.8 annual trips of an average length of 98.8 miles. The Calculated unit costs are $0.05 per ESAL-mile plus $0.008 per mile as shown in Table 2.2 also lists the breakdown of each cost category considered. Both unit costs, ESAL-mile and Mile cost, allocate three categories. For a permit cost calculation example, when looking specifically at the 5-axle trucks with the GVW was 113,006 pounds which would produce nearly 17 ESALs and the trip length was 152 miles, the permit cost was $129.20 (17 ESALs multiplied by 152 Miles multiplied by $0.05 per ESAL Mile) plus $1.22 (152 miles multiplied by $0.008 per mile) which is equal to $130.42.
Table 2.2 Unit Cost Summary from Ohio DOT

<table>
<thead>
<tr>
<th>Cost Categories Allocated</th>
<th>ESAL-mile</th>
<th>Mile</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1) Load Bearing Damage (Millions)</td>
<td>$89.35</td>
<td>/</td>
</tr>
<tr>
<td>(2) 1'' for OW (Millions)</td>
<td>$29.27</td>
<td>/</td>
</tr>
<tr>
<td>(3) Bridge Asset Consumption (Millions)</td>
<td>$21.20</td>
<td>/</td>
</tr>
<tr>
<td>(5)+(6) Pavement (Millions)</td>
<td>/</td>
<td>$3.19</td>
</tr>
<tr>
<td>(4) Bridge (Millions)</td>
<td>/</td>
<td>$0.86</td>
</tr>
<tr>
<td>Total Annual (Millions)</td>
<td>$139.82</td>
<td>$4.05</td>
</tr>
<tr>
<td>Total ESAL Miles/Miles (Millions)</td>
<td>2785</td>
<td>523</td>
</tr>
<tr>
<td>Unit Cost</td>
<td>$0.05</td>
<td>$0.01</td>
</tr>
</tbody>
</table>

Expenses Considered

- (1) Load bearing structural pavement thickness
- (2) Additional 1.0'' pavement thickness due to overweight trucks
- (3) Bridge Asset Consumption
- (4) Bridge Preservation Cost
- (5) 3.00'' minimum pavement thickness and 6'00 aggregate
- (6) 2.0'' overlays

Wisconsin State

2009 and 2012-Bridge Analysis and Evaluation of Effects under Overload Vehicles-Phase I and II

H. Bae and M. Oliva initiated the evaluation of bridge under overload vehicle in 2009, the first phase was mainly focus on the structural analysis of bridges under the overweight trucks. Finite element models of 118 multi-girder bridges were developed and 16 load cases of overload vehicles for each multi-girder bridge were performed. The girder distribution factor equations for multi-girder bridges under overload vehicles were proposed afterward. It was found that intermediate diaphragms under overload vehicles is
not of a concern from the investigation. As an extension of first phase, the authors aimed in evaluating the long term cost impact of vehicles on bridges with life cycle cost analysis. Finally, long term behavior of concrete decks and steel girder bridges was investigated and a means to assign cost to the overloads was developed.

**2012-Impact of Overweight Vehicles on Bridge Deck Deterioration**

Many researchers has stated that the deterioration of bridge decks are a complicated results of different failure modes, such as corrosion, fatigue, global or local flexural crack and so on. Z. Lin et al. from University of Wisconsin-Milwaukee performed an investigation of the impact of overweight trucks on bridge deck deterioration using laboratory tests and numerical simulations. The laboratory tests simulated the combined effect of mechanical stresses and freeze-thaw cycles on concrete cylinders and the results confirmed that the mechanical loading combined with freeze-thaw cycles significantly increased the permeability of air-entrained concrete and may accelerate the deterioration of concrete elements such as bridge decks. The numerical simulation of bridge deck analyzed the stress level in both transverse and longitudinal direction. In addition, empirical equations were proposed to predict the stress under heavy wheel load.

**2013-Aligning Oversize/Overweight Permit Fees with Agency Costs: Critical Issues**

T. Adams et al. performed a review of current permitting practice from different states, and fee structure. The preliminary trends for overweight and oversize demand in the foreseeable future was also outlined. In addition, the different infrastructure impacts of OSOW loads were documented, such as pavement, bridge, safety, congestion, and
environment. At last, a methodology was proposed to quantify the cost but without validation.

**South Carolina State**

*2013-Rate of Deterioration of Bridges and Pavements as Affected by Trucks*

The Clemson University group analyzed the rate of deterioration of bridges and pavements as affected by trucks for the South Carolina Department of Transportation. The effect of overweight trucks and super-load trucks were evaluated. The cost for bridge is consisted with annual bridge fatigue damage cost for superstructure (girders only) and annual bridge maintenance cost (data obtained from DOT). The unit cost directly from the report is shown in Table 2.3 shows the transformed unit cost in dollar per mile per ton.
<table>
<thead>
<tr>
<th>Truck Type</th>
<th>Bridge (Per Mile)</th>
<th>Combine Pavement and Bridge (Per Mile)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
<td>B</td>
</tr>
<tr>
<td>2-axle, 35-40 kips</td>
<td>$0.0040</td>
<td>$0.0058</td>
</tr>
<tr>
<td>3-axle, single unit, 46-50 kips</td>
<td>$0.0062</td>
<td>$0.0075</td>
</tr>
<tr>
<td>3-axle, combination, 50-55 kips</td>
<td>$0.0075</td>
<td>$0.0094</td>
</tr>
<tr>
<td>4-axle, single unit, 63.5-65 kips</td>
<td>$0.0061</td>
<td>$0.0067</td>
</tr>
<tr>
<td>4-axle, combination, 65-70 kips</td>
<td>$0.0067</td>
<td>$0.0092</td>
</tr>
<tr>
<td>5-axle, 80-90 kips</td>
<td>$0.0074</td>
<td>$0.0111</td>
</tr>
<tr>
<td>6-axle, 80-90 kips</td>
<td>$0.0101</td>
<td>$0.0139</td>
</tr>
<tr>
<td>6-axle, 90-100 kips</td>
<td>$0.0101</td>
<td>$0.0191</td>
</tr>
<tr>
<td>6-axle, 100-110 kips</td>
<td>$0.0101</td>
<td>$0.0262</td>
</tr>
<tr>
<td>7-axle, 80-90 kips</td>
<td>$0.0115</td>
<td>$0.0152</td>
</tr>
<tr>
<td>7-axle, 90-100 kips</td>
<td>$0.0115</td>
<td>$0.0201</td>
</tr>
<tr>
<td>7-axle, 100-110 kips</td>
<td>$0.0115</td>
<td>$0.0267</td>
</tr>
<tr>
<td>7-axle, 110-120 kips</td>
<td>$0.0115</td>
<td>$0.0354</td>
</tr>
<tr>
<td>7-axle, 120-130 kips</td>
<td>$0.0115</td>
<td>$0.0469</td>
</tr>
<tr>
<td>8-axle, 80-90 kips</td>
<td>$0.0121</td>
<td>$0.0157</td>
</tr>
<tr>
<td>8-axle, 90-100 kips</td>
<td>$0.0121</td>
<td>$0.0204</td>
</tr>
<tr>
<td>8-axle, 100-110 kips</td>
<td>$0.0121</td>
<td>$0.0265</td>
</tr>
<tr>
<td>8-axle, 110-120 kips</td>
<td>$0.0121</td>
<td>$0.0344</td>
</tr>
<tr>
<td>8-axle, 120-130 kips</td>
<td>$0.0121</td>
<td>$0.0446</td>
</tr>
</tbody>
</table>

**Note:** A: Per Mile Damage for a Truck Loaded at the Legal Weight Limit, B: Per Mile Damage for an Overweight Truck Loaded up to the Maximum Overweight Limit. C: Additional per Mile Damage for an Overweight Truck above the Legal Weight limit up to the Maximum Overweight Limit.
2.3. Structural Reliability Analysis

Reliability analysis is based on making a distinction between the “success” and the “failure” of a structure’s ability to accomplish its intended purpose. The limit state function is used in developing this boundary. It is defined as a boundary between the desired and undesired performance of a structure, and it is mathematically represented. There are three main categories of limit state functions: ultimate, serviceability, and fatigue.

Strength limit state functions deal mainly with the ultimate capacity of the structure, in terms of flexure, shear, torsion, or buckling. Thus, modes of failure may include: exceeding the moment carrying capacity of the structure, crushing of concrete in compression, or buckling of the web. Serviceability limit state (SLS) functions deal with the lifetime performance of the structure in terms of deterioration and the user's comfort. Modes of failure include: excessive deflection or vibration, concrete deck cracking, and permanent deformations. Finally, fatigue limit states (FLS) are related to the deterioration and damage of the structure under repeated loading. Modes of failure include: formation of fatigue cracks, high stresses in secondary members, and damage to welded connections.

The structural reliability analysis begins from establishing the limit state function, which can be expressed as following:

$$g(R, Q) = R - Q$$  \hspace{1cm} \text{Eq. 2.3}

where the R is the capacity and Q is the load effects.

For each limit state, the probability of failure can be expressed as:
\[ P_f = P(R - Q < 0) = P(g < 0) \]  
Eq. 2.4

Figure 2.2 presents the probability density function (PDF) of load and resistance and probability of failure is corresponding to the shaded area in Figure 2.3.

![PDF of Load and Resistance](image)

Figure 2.2. PDF of Load and Resistance

![Illustration of Probability of Failure](image)

Figure 2.3. Illustration of Probability of Failure

Once the statistical information such as distribution type, mean and standard deviation of R and Q has been obtained from the experimental or simulation techniques, the safety level of the structural member can be evaluated using reliability index \( \beta \):
\[ \beta = \frac{\mu_R - \mu_Q}{\sqrt{\sigma_R^2 + \sigma_Q^2}} \]  

Eq. 2.5

where,

\( \beta \) = reliability Index,

\( \mu_R \) = mean value of resistance moment,

\( \mu_Q \) = mean value of the moment due to applied loads,

\( \sigma_R \) = standard deviation of the resistance moment,

\( \sigma_Q \) = standard deviation of the moment due to applied loads.

The reliability index \( \beta \) is correlated with the probability of failure and can be converted to it by looking up the tabulated values for normal probability distribution.

2.3.1. Target Reliability Index

The reliability index considers only two parameters of the random variable (mean and standard deviation) when describing its statistical variation, hence the term “second moment”. This information is used to specify a boundary between safe and unsafe conditions. The limit state function also has its own statistical variability, with a mean and standard deviation based on the input random variables. Since this boundary is specified when the limit state function is equal to zero, the reliability index measures how far the mean of the limit state function is from failure in terms of number of standard deviations. Thus the further the central tendency of the limit state function is away from zero or the failure boundary, the lower the probability of failure and the greater the reliability of the structure. This relation is shown in Figure 2.3. Cornell (1969) defines the reliability index in the following way:
The probability of failure, Pf, is given by

\[ P_f = \Phi(-\beta) \quad \text{Eq. 2.7} \]

where

\[ \Phi(\cdot) \] is the cumulative standard normal distribution function. It follows that the reliability index can be also given by

\[ P_f = \Phi(-\beta) \quad \text{Eq. 2.8} \]

where

\[ \Phi^{-1}(\cdot) \] is the inverse cumulative standard normal distribution function.

Table 2.4 provides the relation between the probability of failure and the reliability index. For example, the probability that one out of a million units will fail, \(10^{-6}\), corresponds to a reliability index, \(\beta\), of 4.75.

<table>
<thead>
<tr>
<th>Probability of failure (P_f)</th>
<th>Reliability Index (\beta)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(10^{-1})</td>
<td>1.28</td>
</tr>
<tr>
<td>(10^{-2})</td>
<td>2.33</td>
</tr>
<tr>
<td>(6.31 \times 10^{-3})</td>
<td>2.5</td>
</tr>
<tr>
<td>(10^{-3})</td>
<td>3.09</td>
</tr>
<tr>
<td>(2.51 \times 10^{-4})</td>
<td>3.5</td>
</tr>
<tr>
<td>(10^{-4})</td>
<td>3.71</td>
</tr>
<tr>
<td>(10^{-5})</td>
<td>4.26</td>
</tr>
<tr>
<td>(10^{-6})</td>
<td>4.75</td>
</tr>
<tr>
<td>(10^{-7})</td>
<td>5.19</td>
</tr>
<tr>
<td>(10^{-8})</td>
<td>5.62</td>
</tr>
<tr>
<td>(10^{-10})</td>
<td>5.99</td>
</tr>
</tbody>
</table>
2.3.2. Random Variables

In order to perform the reliability analysis for the different limit states, the probabilistic distribution and statistical parameters (μ and σ) of various random variables are needed. Table 2.5 shows a summary of the information based on previous research studies by Siriaksorn and Naaman (1980) and Nowak et al. (2008).

Table 2.5 Summary of typical statistical information for various variables from previous research (Siriaksorn and Naaman 1980, Nowak et al. 2008)

<table>
<thead>
<tr>
<th>Variables</th>
<th>Distribution</th>
<th>Mean, m</th>
<th>COV., Ω</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>$b$, $b_1$, $b_w$</td>
<td>normal</td>
<td>$b_n$</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>$h$, $h_{f1}$, $h_{f2}$</td>
<td>normal</td>
<td>$h_n$</td>
<td>$1/6.4 \mu$</td>
<td></td>
</tr>
<tr>
<td>$d_p$, $d_1$, $e_1$</td>
<td>normal</td>
<td>$d_{pn}$, $d_{an}$, $e_{in}$</td>
<td>0.04-0.68/$h_n$</td>
<td></td>
</tr>
<tr>
<td>$l$, $a$</td>
<td>normal</td>
<td>$l_a$, $a_a$</td>
<td>$11/(32 \mu)$</td>
<td></td>
</tr>
</tbody>
</table>
| $C_{E_c}$ | normal | 33.6 | 0.1217 | nominal=33 $E_c = C_{E_c} \cdot \gamma_c^{1.5} \cdot \sqrt{f_c'}$
| $C_{f_r}$ | normal | 9.374 | 0.0938 | nominal=7.5 & $f_i = C_{f_r} \sqrt{f_c'}$
| $f_c'$ | lognormal | 1.11 $f_{cn}'$ | 0.11 | |
| $\gamma_c$ | normal | $\gamma_{cn} = 150$ | 0.03 | |
| $C_{f_{si}}$ | normal | 0.6445 | 0.073 | nominal=0.8 & $f_{si} = C_{f_{si}} \cdot f_c'$
| $f_y$ | lognormal | 1.13 $f_{yn}$ | 0.03 | |
| $A_{ps}$ | normal | 1.01176 $A_{pun}$ | 0.0125 | $A_{pun} = 0.153 \text{ in}^2$
| $f_{py}$ | lognormal | 1.027 $f_{pyn}$ | 0.022 | $f_{pyn} = 240 \text{ ksi}$
| $E_{ps}$ | normal | 1.011 $E_{pun}$ | 0.01 | $E_{pun} = 29000 \text{ ksi}$
| $C_{f_{su}}$ | normal | $C_{f_{sus}}$ | 0.08 | $C_{f_{sus}} = 0.7$
| $f_{si} = C_{f_{sus}} \cdot f_{pu}$ |
| $C_{f_{se}}$ | normal | $C_{f_{sus}}$ | 0.08 | $C_{f_{sus}} = 0.83$
| $f_{se} = C_{f_{se}} \cdot f_{si}$ |
2.3.3. Monte Carlo Simulation

Estimation techniques are sometimes necessary and useful in determining the reliability index of the more complicated, nonlinear limit state functions. The methods include: Monte Carlo simulation, Rosenblueth's $2n+l$ method, Latin Hypercube Sampling, and integration methods.

The Monte Carlo simulation is one of the most fundamental estimation techniques. It involves using a random number generator to generate values for each random variable, based on their corresponding probability distribution, and calculating the limit state function. Values of the limit state function are thus statistically variable with a mean and standard deviation. To obtain a certain level of accuracy, a high number of simulations must be carried out. The number of simulations will depend on the complexity of the structure and the reliabilities of the various components. Elements with a high reliability will require a greater number of simulations as opposed to those with a lower reliability. With the availability of high-speed computers, however, this becomes of little concern.

In complex engineering problems, it becomes necessary to simulate distributions based on a fewer number of tests, as it is often impractical or not economical to run tests multiple times in order to establish probability distributions for each random variable. Once these probability distributions are constructed, parameters such as the mean and the standard deviation may be obtained for each random variable. The limit state function is then evaluated at each set of random values for the random variables. A CDF for the limit state function is then constructed and the probability of failure can be determined.
If \( N \) is the total number of simulations and \( k \) is the number of simulations in which \( g < 0 \), then the probability of failure is

\[
P_f = \Pr(g < 0) = \lim_{N \to \infty} \frac{k}{N}
\]  

Eq. 2.9

The simulated values are plotted on normal probability paper (NPP), for which the vertical axis is the cumulative probability associated with the corresponding value of the random variable on the horizontal axis. If for \( N \) simulations, the limit state function does not fall below zero, that is \( k = 0 \), then the cumulative distribution function of the simulated \( g \) values can be extended until it intersects the vertical axis. The probability of failure is then the ordinate of the intersection point.

The basis of the Monte Carlo simulation is the generation of random numbers, \( u \), that are uniformly distributed between 0 and 1. This is often contained in computer software as a built-in function or subroutine. Values of each random variable are then simulated based on their respective probability distribution.

### 2.4. Fatigue Evaluation of Steel Girders

Fatigue is a mode of failure whereby a crack develops and propagates within metal under loads that are less than the design ultimate strength of the structure. The ASTM definition: “The process of progressive localized permanent structural change occurring in a material subjected to conditions which produce fluctuating stress and strains at some point or points and which may culminate in cracks or complete fracture after a sufficient number of fluctuations” (ASTM E206-62T). As a progressive nature, most of the fatigue failures are initiated from small cracks at relatively low nominal stresses. These cracks usually start at small flaws or stress concentrations, and
propagated slowly. However, such cracks may lead to catastrophic brittle failure when they propagate to a critical size.

2.4.1. Historical Overview of Fatigue

To gain a general understanding, it is best to start with a brief historical review of fatigue developments. This shows a few basic ideas and indicates very briefly how they were developed by the efforts of many people. Engineers noted fatigue failures as early as 1829 (Munse 1990). Albert studied this phenomenon in conveyor chains used in coalmines in 1837 (Schütz 1996). In 1840s, railroad axles failed regularly at shoulders due to repeated stresses, and the failure was different from the normal ruptures. An erroneous concept of “crystallization” was brought out at that time. Until 1850s, the word “fatigue” was introduced to describe the failure from repeated stresses. In Germany during 1850s and 1860s, August Wohler, who was called “father” of systematic fatigue testing, performed lots of laboratory fatigue test concerned with railway axle failures. He first proposed the concept of Stress-Life (S-N) diagrams.

2.4.2. Rainflow Cycle Counting

There are a variety of cycle counting methods available. The goal of each method is to best describe the effects of variable amplitude loading in terms of discrete cycles, which can be compared to constant amplitude test data (Bannantine 1990). Rainflow cycle counting, as specified in ASTM, identifies closed hysteresis loops from the loading spectrum. Matsuishi and Endo first described the original Rainflow method in 1968. Cycles are defined the same way that rain falls from pagoda roofs. The stress history is
rotated vertically, such that time increases downward. The primary cycles are extracted and the process is repeated for the minor cycles.

1. In order to eliminate the counting of half cycles, the strain-time history is rearranged to begin at the largest strain value. More complex procedures have been developed to eliminate this requirement (Downing 1982).

2. A flow of rain is begun at each strain reversal in the history and is allowed to continue to flow unless: (a) the rain began at a local maximum point (peak) and falls opposite a local minimum point greater than that from which it came; (b) The rain began at a local minimum point (valley) and falls opposite a local minimum point greater (in magnitude) than that from which it came; (c) It encounters a previous rainflow.

The following example illustrates the rainflow counting method for a sample complex loading as shown in Figure 2.4. The procedure is started at each reversal:

1. Rain flows from point A over points B and D and continues to the end of the history, since none of the conditions for stopping rainflow are satisfied.

2. Rain flows from point B over point C and stops opposite point D, since both B and D are local maximums and the magnitude of D is greater than B (rule 2a).

3. Rain flows from point C and must stop upon meeting the rain flow from point A (rule 2c).

4. Rain flows from point D over points E and G and continues to the end of the history, since none of the conditions for stopping are satisfied.

5. Rain flows from point E over point F and stops opposite point G, since both E and G are local minimums and the magnitude of G is greater than E (rule 2b).
6. Rain flows from point F and must stop upon meeting the flow from point D (rule 2c).

7. Rain flows from point G over point H and stops opposite point A, since both G and A are local minimums and the magnitude of A is greater than G (rule 2b).

8. Rain flows from point H and must stop upon meeting the rainflow from point D (rule 2c).

Figure 2.4. Rainflow counting example (Bannantine 1990)

The following closed hysteresis loops are computed from Figure 2.4: A-D-A, B-C-B, E-F and G-H. The resulting rainflow table is compact compared to the much larger stress history. Thus, a very lengthy time history is equivalently described in a matrix of values.
Rainflow counting can be accomplished in a variety of forms, some examples include: range only, range-mean, and to-from. In range only counting, only the range of the cycle is kept. The range-mean method contains the basic range of the cycle in addition to the mean value of the min and max. Finally, the To-From rainflow matrix, contains the starting and ending point of every cycle. Therefore, with a To-From matrix, information about a cycle’s origin and terminus are maintained. Information about load switching from tension to compression is also preserved.

2.4.3. Cumulative Damage Estimation

Multiple laboratory tests of specimens subject to repeated loading cycles at constant amplitudes are used to generate these S-N curves. The loading patterns of actual structures, however, contain random variable amplitude stress cycles. Therefore, a means to find an equivalent damage accumulation is needed. The linear cumulative damage rule, or the Palmgren-Miner Rule, herein referred to as Miner’s Rule, is used to relate variable amplitude behavior to constant amplitude behavior (Miner 1945). When the damage reaches unity, this can be defined as the failure criterion. Miner’s Rule, in its simplest form, is given as:

$$\sum \frac{n_i}{N_i} = \frac{n_1}{N_1} + \frac{n_2}{N_2} + \ldots + \frac{n_n}{N_n} = 1$$

Eq. 2.10

where

\( n_i \) = number of stress cycles at level \( \sigma_i \),

\( N_i \) = number of stress cycles to produce failure at \( \sigma_i \).

The damage caused by a load history is not immediately clear from the number of cycles or the maximum stress range. In other words, the most damaging load history is
not necessarily the one with the highest number of cycles. The most damaging load history is most likely the history that contains a large number of mid-to-high range cycles (Socie and Pompetzki 2004). Therefore, it is critical that the cumulative damage method be applied to normalize each load history for comparison.

2.4.4. Evaluation of Bridge Fatigue Life with AASHTO the Manual for Bridge Evaluation (MBE)

The current evaluation provisions for highway bridges are set forth in the AASHTO the MBE (AASHTO 2011). The principle inputs for fatigue design are the average daily truck traffic, percentage of truck traffic, and connection detail category. The manual identifies two levels of fatigue evaluation: the infinite-life check and the finite-life calculations. The bridge details are subject to the more complex finite-life fatigue evaluation when they fail the infinite-life check. The steps of the evaluation is summarized and shown in Figure 2.5.

The current evaluation, and the American Welding Society (AWS) code, specify detail categories for welded and bolted connections. The categories are denoted by letter and include: A, B, B’, C, D, D’, E, and E’. Category A details include rolled beam sections and are considered the most fatigue resistant details. Category E’, however, are the most susceptible to fatigue damage and include longitudinally loaded fillet-welded attachments. The Stress Range Versus Number of Cycles used in AASHTO is shown in Figure 2.6. The fatigue design is based on a single lane loaded. Therefore, the average daily truck traffic is used for determination of fatigue loading. Multiple truck loading is considered rare (Moses 1987). Special provisions are given for cases when multiple truck situations may occur. For example, bunching of trucks may occur on a bridge near traffic signals or uphill
on a two or more lane bridge. For these cases, a 15 percent increase in fatigue truck weight is prescribed.

Figure 2.5. Flowchart for Fatigue Evaluation in AASHTO the MBE (AASHTO 2011)
2.4.1. Evaluation of Bridge Fatigue Life with Linear Elastic Fracture Mechanics (LEFM) Method

Upon above discussion of fatigue assessment, the S-N curve-based stress-life approach was presented. This method has the advantage of being simple and widely adopted by the bridge design and evaluation. However, the stress-life approach has the disadvantages that it cannot be used to provide information about crack size and anticipated crack growth rates at various stages of the service life of a bridge. The fracture mechanics approach for fatigue crack growth can be useful where information about crack size is either known or needed. This will be the case where a fatigue crack has been detected, and the information about the remaining fatigue life is needed. The application in fatigue involves the crack growth process, covering the range from a detectable crack or flaw to final fracture. One of the common methods used in this process is Linear Elastic Fracture Mechanics (LEFM) with the assumption that material conditions are predominantly linear elastic during fatigue process. When the assumption is violated, elastic-plastic fracture mechanics approaches are applied. Figure 2.7 describes the basic procedures of LEFM.
The above discussion gives the basis for conducting fracture mechanics calculations for a wide range of cases, including fatigue susceptible details on steel bridge structures. A fundamental assumption of LEFM fatigue calculations is that prior to loading, small initial cracks or defects are already present. Previous research (Smith 1982) has shown that this is (for practical purposes) an appropriate assumption for welded civil engineering structures. Modifications of above three equations are available from previous study. Figure 2.8 shows that on a log-log plot of $da/dN$ versus $\Delta K$, Paris law of stable crack growth results in a straight-line relationship, with a slope of $m$. In reality, fatigue cracks in steel specimens are known to exhibit also non-linear behavior, that is, as $\Delta K$ decreases it
will approach a threshold value wherein the crack growth rate will decrease rapidly to zero. Ignoring this effect will generally lead to the underestimation of fatigue life (which is conservative), with the degree of underestimation depending on the number of stress cycles occurring near or below this threshold. In order to consider threshold effects, the equation of Paris law can be modified (for example) as follows:

\[
\frac{da}{dN} = C \cdot \left( \Delta K^m - \Delta K_{th}^m \right)
\]

Eq. 2.11

where

- \(a\) = crack size (depth or length),
- \(N\) = number of cycles
- \(C\) = constant of the Paris law,
- \(m\) = constant
- \(\Delta K\) = stress intensity factor range
- \(\Delta K_{th}\) = threshold value of the elastic stress intensity factor range.

\(\Delta K_{th}\) can be set at a fixed value, however some researchers have suggested that \(\Delta K_{th}\) varies, in fact, with the stress ratio \(R\), that is, the ratio of the minimum over the maximum applied stress. This phenomenon can be considered in the model with various expressions relating \(\Delta K_{th}\) to the R-ratio, which can be found in literature (Gurney 1968). Alternatively, the \(da/dN\) versus \(\Delta K\) relationship may be modelled using a 2-slope relationship.
2.5. Deterioration of P/C I-Girders

After reviewing the historical inspection reports of these structures, the deterioration process of P/C girders is a combination of mechanical loading and environmental attack. The cracks (i.e., possibly shear and bearing cracks) appeared on the bottom flange and web at the beam-end of P/C I-girders. Due to the deteriorated deck joints, the water, and de-icing salts drained from the top of the bridges increased the moisture ingress in a cracked section. The cracks were widened after freezing and thaw cycles which led to the spalling of concrete. Eventually, the prestressing tendons were exposed and corroded. The corrosion of tendons is critical since even minor corrosion will lead to huge loss of the load carrying capacity. Previous work also indicated that the deterioration of P/C bridges induced by corrosion was near the beam-ends of P/C girders from field investigations (Whiting D. et al 1999, Needham 2000). Using detailed finite element analysis, Koyuncu et al. (2003) indicated that the cracks at the beam-ends of P/C I-girders were caused by high shear stress exceeding the shear capacity of concrete.
2.6. Deterioration of R/C Bridge Deck

In the AASHTO Load and Resistance Factored Design (LRFD) Bridge Design Specifications (AASHTO 2012), two limit states are considered for bridge deck design: 1) service I limit state that controls the crack width of reinforced concrete deck, and 2) strength limit state that controls the nominal capacity of the concrete deck at ultimate. However, previous studies (Batchelor et al. 1978, Fan et al. 1990, Petrou et al. 1994) indicated that bridge decks subjected to concentrated loads do not fail by flexure, as was traditionally believed, but by fatigue. Starting from the late 1970s, laboratory tests were performed to investigate the failure modes of reinforced concrete decks as a whole. Test results have shown that the fatigue of reinforced concrete deck is governed by the punching shear failure of concrete. From available test data (Petrou 1994, Youn and Chang 1998), all the fatigue models for bridge decks yielded models such as in Eq. 2.12. The equivalent wheel load is regarded as an important load parameter for deck fatigue. Since the rear wheel usually has dual tires while the steering wheel is usually a single tire, the value for steering wheel weight was increased by 1/0.67.

\[ N_{pf} = 10^{a - \log \left(\frac{P}{P_u}\right) / b}, \]  
\[ P = \left( \sum f_i(p_i) \times p_i^{17.95} \right)^{1/17.95} \]

where

- \( N_{pf} \) = number of cycles to failure,
- \( P \) = equivalent wheel load from wheel weight distribution,
- \( P_u \) = ultimate strength of the deck,
- \( a \) and \( b \) = parameters of regression line,
\[ p_i = \text{value of wheel weight (kips) in wheel weight distribution, and} \]

\[ f_d(p_i) = \text{frequency for that weight.} \]

In order to avoid water and chemical penetration and corrosion of steel reinforcement, the AASHTO LRFD Bridge Design Specifications provided provisions to control cracking of concrete deck by limiting the spacing of reinforcement under a limitation that is a function of exposure condition, tensile stress in reinforcement, and thickness of concrete cover. In addition, extensive laboratory work also proved that the crack width of concrete deck at service limit state is proportional to the steel stress with significant variation.

Another major concern about the deck deterioration comes from the chloride-induced corrosion of steel reinforcement bars, specifically for the bridges that receive chlorides through applications of de-icing salts or marine environment. The corrosion will lead to a reduction in the cross-sectional area of the reinforcing steel or loss of bond, which may further lead to the loss of strength and unserviceability. Various models (Liu and Weyers 1998, Stewart and Rosowsky 1998, Vu and Stewart 2000) of this type of failure were developed based on the mechanism of the diffusion of chlorides through the protective concrete cover, showing that the corrosion will be initiated once the chloride concentration exceeds a certain threshold. In addition, cracking, spalling, and delamination induced by mechanical loading accelerates the attack of chloride and prompt deck maintenance helps preserve the deck from corrosion.
2.7. Bridge Life Cycle Cost Analysis (BLCCA)

Steel bridges represent more than 30% of the total number of highway bridges in the United States (FHWA, 2012). In New Jersey, about 54.1% of total bridges are steel bridges from data of NBI. Defects in a steel bridge will generally appear as a result of aging, harsh environment, or a result of traffic loading. Timely identification and quantification of defects and their respective growth rates will help transportation agencies to develop proper efficient maintenance and rehabilitation (M&R) strategies that can greatly influence the associated cost as well as overall service life. Delayed maintenance may not only increase dramatically the M&R costs, but also may result in irreparable damage to the steel components.

The economic performance is usually best assessed using a life cycle cost analysis (LCCA). As one of the widely used evaluation tool, LCCA is a practical yet highly useful approach for selecting and evaluating appropriate construction and maintenance techniques. It is mainly applied to assist decision-makers to find optimum strategies to accurately assess internal and external costs of any transportation maintenance and rehabilitation project with the ultimate objective of satisfying budget constraints.

Many studies have been conducted in the past to illustrate the life cycle cost of steel bridges. Okasha et al. (2012) compared the life-cycle cost (LCC) of a steel bridge girder made of a new maintenance-free steel and conventional painted carbon steel with maintenance. Deterministic approach was applied as well as probabilistic approach, where certain input variables are properly considered with uncertainties, such as maintenance schedule and discount rate. The result showed that new maintenance-free steel, although initially more expensive, is more cost-effective after approximately 15
years. In 2014, Soliman and Frangopol (2014) extended this study to include indirect user and environmental costs from the traffic delays and their social and environmental impacts and had consistent conclusion that the maintenance-free steel represents a more-sustainable alternative to the conventional carbon steel for bridge construction in corrosive environments.

Steel is often considered by the steel industry as a highly recyclable material thus many researchers have conducted studies considering environmental impacts along with LCCA. Gervásio and da Silva (2008) developed an integrated methodology for a life-cycle and sustainability analysis to consider both contemplates environmental and economic aspects for two M&R alternatives, composite concrete-concrete structure and concrete-steel structure, respectively. They assumed that the overall performance ranking is a weighted average between the environmental performance and the economic performance. Their case study showed that despite a clear cost advantage of the concrete solution (20% cheaper), the environmental life-cycle analysis could invert this result in favor of the steel solution.

Du et al. (2014) conducted a life cycle assessment (LCA) focus on the 20 types of environmental impact indicators among five bridge designs that two out of five are using steel components. Although this is not a life cycle cost analysis, the LCA results can be converted into monetary values with common unit by adopting weights method in order to provide an intuitive and comprehensive result for the decision makers. In this study, the environmental costs of each design were assessed by aggregating their characterized mid-point environmental categories and were ranked.
Lee et al. (2006) presented an optimal design of steel bridges in terms of the life cycle cost and environmental stressors such as corrosion and heavy truck traffic. One of the important points is that they brought in annual probability of failure that depends upon the prior and updated load and resistance histories. The author concluded that in the case of urban corrosion environmental regions with heavy average daily traffic volume, the high-performance steel is advantageous not only in view of initial cost effectiveness but also in view of LCC-effectiveness, since the optimum design of high performance steel is achieved at higher allowable stress level compared with conventional ones. Similarly, Kim et al. (2013) also considered annual probability of failure at each damage level to develop a methodology for determining the optimal target reliability and adjusting procedure using minimal LCC. Reliability analyses of bridges using various traffic conditions, span lengths, and number of traffic lanes were performed to verify the alterations of structural reliability. The analysis reveals that the LCC of steel bridges has great influence on target reliability, especially when the bridges have many lanes and long spans. Figure 2.10 shows a summary of previous LCCA on steel components. Ertekin, Nassif, and Ozbek (2008) developed a comprehensive LCCA algorithm using real data from NBI to take all structural components of bridge into consideration and predicted the agency and user costs of rehabilitation, repair, and reconstruction more accurately.
Table 2.6 Summary of previous LCCA on steel components

<table>
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<tr>
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<th>Agency Cost</th>
<th>User Cost</th>
<th>Social Cost</th>
<th>Environmental Impact</th>
<th>Probabilistic</th>
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<td>M. Soliman and D. Frangopol (2014)</td>
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<td>Lee et al. (2006)</td>
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Two operation levels are usually identified in LCCA evaluation: (1) the project level analysis and (2) the network level analysis. Although the objectives, level of information, components, and approach are different in both levels, the decisions attained at one level must interface with those attained at the other level to achieve efficient management.

Project-level analysis is considered a bottom-up approach. It deals with technical concerns and requires very detailed information. The general inputs for project-level analysis include but not limited to acoustic emission and strain gage data from field testing, traffic loading, material characteristics and performance, environmental factors, construction and rehabilitation variables, and costs. One of the major innovations of this task will be to integrate testing data from sensors on instrumented bridges, loading data from WIM stations and inventory data from NBI to predict the cost implications of various traffic and related loading scenarios. The second step is to estimate steel bridge’s probability of failure $P(f)_t$ at year $t$ that depends on the prior and updated loads and
conditional rating. Once the probability of failure is identified, different M&R alternatives will be developed with their type of treatment, schedule and costs associated. From previous analysis, the conditions of bridge components can be divided into three cases:

1. Components with visible crack – (most deteriorated)
2. Components with invisible crack (micro-crack) – (moderate deteriorated)
3. Components without crack – (safe)

Since bridge fatigue life (cycles to failure) versus stress range (bridge response) are not in a linear relationship, prompt repair when cracks are detected or even strengthening in non-cracked details can significantly prolong the fatigue life of bridge components. In addition, different strengthening methods could lead to different fatigue life. Therefore, it is important to investigate the bridge life cycle cost among different strengthening scenarios to find the best timings and types of strengthening. This analysis can help the decision-maker to develop an efficient methodology (Figure 2.9) to specify maintenance schedule.

After examining M&R strategies, LCCA will be performed to determine the most feasible alternative that minimizes the life cycle cost of the given project. Since innovative detection technology is always associated with uncertainties, a probabilistic LCCA approach will be applied. Other important parameters with inherent uncertainties should also be treated probabilistically, such as the discount rate, traffic growth rate, and unit cost of strengthening methods. The final output of LCCA will be probabilistic distributions of the alternatives.
Network-level analysis, on the other hand, is a top-down approach where the overall network or agency goals (i.e. budget) are established first and thus drive the selection process for projects that best achieve these goals. Steps for network-level analysis are:

1. Identify all the fatigue critical bridges in New Jersey
2. Integrate WIM, bridge data
3. Cluster the fatigue critical bridges based on the detail types
4. For each bridge cluster, rank all bridges based upon traffic loading, prior and current bridge condition via analytical models (i.e. Genetic algorithm, weighted method)
5. Set up a threshold, select a certain percentage (i.e. 30%) of top ranked bridges with poor conditions and heavy loads.
6. Perform a life cycle cost analysis on each of the selected bridges and re-rank them based on life cycle cost results.
7. Identify network maintenance and rehabilitation strategies to satisfy the urgent needs for poor condition bridges and agency budget. The network wide strategies will be distributed in different future years. (i.e. repair 3 bridges at year 1, 5 bridges at year 5...)
8. Once the maintenance and rehabilitation actions are taken, go back to step 1 with an updated bridge condition and repeat the process.

Figure 2.10 shows the flow diagram for both project-level and network-level LCCA-based decision making process.
In addition to the probabilistic LCCA methodology described above, a GIS-based web tool that can access state-wide infrastructure data that is being developed by Nassif and Ozbay as part of other NJDOT funded projects (reference to ASSISTME-WIM and
Permit projects) will be modified to incorporate components specific to this project. This current tool is able to automatically extract road and traffic data for each bridge in New Jersey. It will calculate life cycle costs of each individual bridge using the proposed LCCA methodology for different maintenance / repair / replacement scenarios. The RIME team will incorporate a flexible interface to define performance functions specific to the construction materials / technologies used in this project. This tool is also envisioned to serve as a database of performance functions and decision making. Moreover, the advantage of a web-based tool is that it can be used by any user without having to install additional software (and possibly data) on their own computers. This simplifies the process of using the developed software by a large number of users who have simple to a simple web browser.

2.7.1. Life Cycle Cost Analysis Methodology

Aging facilities along with growing demand in terms of operational and environmental requirements as well as increasing costs associated with maintaining the transportation infrastructure have led agencies to seek the development of innovative detection and construction technologies. As steel bridges represent more than 30% of the total number of highway bridges in the United States (FHWA 2012) and more than 54% of the bridges in New Jersey, their damage detection and treatment strategies should take into consideration the life cycle cost, including future maintenance and repair cost, vehicle operation costs, work-zone delay cost, and socio-economic impacts resulting from all of these activities.

Defects in a steel bridge will generally appear as a result of aging, harsh environment, or a result of traffic loading. Timely identification and quantification of
defects and their respective growth rates will help transportation agencies to develop proper efficient maintenance and rehabilitation strategies that can greatly influence the associated cost as well as overall service life. Delayed maintenance may not only increase dramatically the M&R costs, but also may result in irreversible damage to the steel components. However, how to effectively relocate limited budget and resources remain a challenge to every transportation agency. Thus, it is critical to develop a reliable life-cycle and cost assessment methodology.

As one of the widely used evaluation tool, LCCA is a practical yet highly useful approach for selecting and evaluating appropriate construction and maintenance techniques. Several approaches have been developed by researchers and practitioners to evaluate and compare potential maintenance and replacement decisions for steel bridges. The existing methodologies usually covers agency cost as well as indirect cost such as user cost or environmental cost. Various studies have been conducted that take account of both contemplates environmental and economic aspects for diverse M&R alternatives since steel is often considered by the steel industry as a highly recyclable material. Some studies brought in annual probability of failure at each damage level that depends upon the prior and updated load and resistance histories to develop a methodology for determining the optimal target reliability so effective M&R strategies can be performed. The overall performance ranking can be a weighted average from LCCA result, traffic loading conditions, and environmental performance, respectively.

LCCA is performed by summing up the monetary equivalency of all benefits and costs at their respective time of occurrence and are converted into a common time dimension so that different alternatives can be compared correctly. A general expected
life-cycle cost up to time $T$ of known conventional material, $LCC(T)$, can be expressed as follows:

$$LCC(T) = C_C + C_M(T) + C_R(T) + C_U(T) + C_S(T) + SV \quad \text{Eq. 2.13}$$

where

$LCC = \text{Life Cycle Cost}$,

$CC = \text{Construction Cost}$,

$CM = \text{Maintenance and repair Cost}$,

$CR = \text{Rehabilitation Cost}$,

$CU = \text{User Cost}$,

$CS = \text{Socio-economic Cost}$,

$SV = \text{Salvage Value}$,

$T = \text{Time}$.

Figure 2.11 illustrates the objective function of LCCA and its general input parameters and output cost components.
Figure 2.11. General LCCA inputs and outputs
CHAPTER 3

FIELD INSTRUMENTATION AND DATA COLLECTION

A comprehensive field-testing program was enacted to understand the behavior of the selected structures. The target bridge spans were monitored to determine the actual truck loading for bridge evaluation. Various types of sensors were adopted, including Structural Testing System (STS), Laser Doppler Vibrometer (LDV) and Weigh-In-Motion (WIM) sensors. These sensors were placed on various bridge components to collect data under normal traffic. In this section, Structural Health Monitoring (SHM) equipments were introduced and detailed structural instrumentations were presented.

3.1. Testing Equipment

3.1.1. Structural Testing System (STS)

The Structural Testing System (STS) is a modular data acquisition system manufactured by Bridge Diagnostics, Inc. (BDI) of Boulder, Colorado. The system consists of a main processing unit that samples data, junction boxes, and strain transducers. The strain transducers are mounted to structural elements with clamps or bolted to epoxied tabs. Each sensor has a unique identification number and a microchip, so the sensors can be identified easily in the system. The sensor calibration factors are stored in the configuration files and applied automatically. The STS is capable of
sampling more than 32 sensors at 100 Hz and can be programmed and controlled with a laptop computer.

The strain transducers measure strains using a full Wheatstone bridge configuration as shown in Figure 3.1. The system consists of strain transducers, junction box, and the main STS unit. Each test is assigned to an automatic file number and the test is initiated using a trigger button called the clicker. Once the test is completed, the data can be downloaded from the STS unit to a laptop computer. The STS data files contain basic test information such as date, time, duration, sensor ID numbers, and the stress data in ASCII text format. Figure 3.2 shows a typical strain gage installed on the floor beam flange.

![Structural testing system by Bridge Diagnostics Inc. (BDI)](image)

Figure 3.1. Structural testing system by Bridge Diagnostics Inc. (BDI)
Figure 3.2. Strain transducer clamped to the bottom flange of a floor beam

3.1.2. Laser Doppler Vibrometer (LDV)

The Laser Doppler Vibrometer (LDV), shown in Figure 3.3, is a noncontact device that measures displacement as well as the velocity of a remote point. A change in the distance between the laser head and the reflective target produces a Doppler shift in the light frequency which is decoded into displacement and velocity. The system is composed of three parts: 1) the helium neon Class II laser head, 2) the decoder unit, and 3) the reflective target attached to the structure. The laser head is mounted to a tripod, which is positioned underneath the target. The reflective target, typically a retro-reflective tape, provides the strongest signal. The signal strength is read on a scale on the laser head and the tripod is adjusted to maximize the signal prior to a test run.
Figure 3.3. Laser Doppler Vibrometer (LDV): (a) LDV Setup, and (b) Reflective Tape and Strain Transducer for Target Location

3.1.3. WIM Sensors and Data Collection Unit

RoadTrax Class 1 WIM Sensors in Figure 3.4 (a) manufactured by Measurement Specialties was installed permanently on selected sites with a portable WIM data collection unit IRD TC/C-540 as shown in Figure 3.4 (b). The pavement will be cut and cleaned to make a slot for the WIM sensor and cable. The concrete pavement is pre-heated using a torch for better grout bonding, and then the brackets is placed at an interval of 6 in. throughout its length. The WIM sensors is placed into the sensor slot, and the AS475 grout was poured in the slot and cured for 60 minutes. Permanent WIM installation requires slightly more effort and time to cut and clean the pavement. However, it provides more accurate WIM data for a longer period of time, because the WIM sensors were embedded in the pavement and react with the pavement.
3.2. New Jersey Turnpike Structure N6.49

3.2.1. Superstructure Instrumentation

Unit 2 and Unit 11 of Structure N6.49, which was built in 1954 and widened in 2006, were selected as examples of highway bridge instrumentation. Unit 2 is a three-span continuous slab on girder structure and Unit 11 is a Girder-Floorbeam-Stringer structure. They are located on Newarkbay-Hudson County Extension (NB-HCE). Figure 3.5 and Figure 3.6 show the locations of strain transducers and accelerometers installed on Unit 2 and Unit 11 of Structure N6.49.
Figure 3.5. Sensor Locations on Structure No. N6.49, Unit 2 (Not to Scale)
Figure 3.6. Sensor Locations on Structure No. N6.49, Unit 11 (Not to Scale)
Six (6) permanent WIM sensors on the eastbound approach lanes of the Newark Bay-Hudson County Extension (NC-HCE) was installed.

The lane was closed when installing the sensors on that lane. When the eastbound closure was active, the installation team moved to the each lane with the equipment, sensors and grout. Two phases of traffic lane closure were conducted. Closure 1 was for Left Lane (LL) and Center Lane (CL), and Closure 2 was for CL, Right Lane (RL) and Shoulder. The overlap of the CL closure allowed the WIM sensors and cables to be installed extending the curing period. The first traffic closure (LL/CL) took approximately 3 hours while the second closure (CL/RL/Shoulder) took about 5 hours including mixing of the grout (10 minutes) and curing time for the grout (60 minutes).

Figure 3.7 and Figure 3.8 show the aerial and plan views of the WIM sensor configuration, and the detailed sensor locations were as follow:

*Sensors 1 and 2 on right lane*

They are located at 38 ft. (sensor 1) and 48 ft. (sensor 2) away from the west abutment on the asphalt pavement in order to avoid the discontinued concrete pavement at and after 50 ft.

*Sensors 3 and 4 on center lane*

They were installed after the potholes and cracks at 53 ft. (sensor 3) and 63 ft. (sensor 4) away from the west abutment on the concrete pavement.
Sensors 5 and 6 on left lane

They are located at 50 ft. (sensor 5) and 60 ft. (sensor 6) away from the west abutment after the potholes on the asphalt pavement.

Figure 3.7. Aerial View of the WIM Sensor Configuration
Figure 3.8. Plan View of the WIM Sensor Configuration

3.2.2. WIM Installation Steps

Contractor shall cut, clean, dry and grind the slots for WIM installation. The sensor and slot configuration is displayed in Figure 3.9. Steps and responsibilities for Contractor are underlined and the screenshots of each step are summarized in Figure 3.10.

Clean the pavement (asphalt and concrete) surface using broom brush. If necessary, use air-blower to remove excessive dust or dirt.
Measure and mark the location of the WIM sensors and cables on the lane using pavement crayons or paint according to the WIM sensor plan.

Cut the pavement for the sensor slots using saw-cutting machine as below. Sensor slot shall be ¾" (0.75") wide, 1" deep and 11'-8" long. Home run slot shall be ¼" (0.25") wide, 1.5" deep and long enough to reach the parapet.

Please note that the width of home run slot can be ¾" (0.75’’), if changing the blade is not feasible to finish the installation within the limited time.

Clean and dry the sensor slot and home run slot with pressurized air.

Heat the concrete pavement for a couple minutes using a propane torch.

Duct tape along the sides of the slot for sensor and/or cables.

Wipe WIM sensors using cloths soaked with alcohol.

Place the installation brackets on the sensor at an interval of 6 in. throughout its length.

Place the sensor with brackets and/or cables into the cut slot, and check the sensor depth with a depth gage.

Mix the large container of the grout to be homogenous without any lumps using an electric drill (at 450 to 550 rpm) and a mixing paddle.

Pour the grout mixture over the sensor and cables, and spread the grout smooth to make the grout to be slightly higher (1/16") than duct tape.

Remove duct tape prior to the grout curing, and cure the grout for 45~60 minutes.
Once grout is cured, use an angle grinder to grind the excessive grout to make the surface smooth.

Figure 3.9. Slot and Sensor Configuration

(a) Draw Sensor Location  (b) Cutting Pavement Surface
3.2.3. Data Collection and Analysis for the WIM system

Traffic data using WIM sensors has been collected since 06/10/2015. The data was last downloaded on 06/22/2016. The downloaded WIM data from the field were processed with a filter programmed according to Wassef et al. (2014) as shown in Figure 3.11. Individual axle weight, distance between axles, and truck speed can be collected with the WIM system. Traffic information using the WIM system was collected to gather site-specific traffic pattern, truck weight, and volume. Live load models (truck model for
the finite element analysis) were also developed based on the data gathered in the field. Using the special filtering program, all passenger vehicle data was eliminated, and only truck loading data were considered in the analysis.

Figure 3.11. Flowchart of the Filtering Process (NCHRP w201)

Table 3.1 lists various statistics of WIM data. As shown in the table, the traffic volume of the Class 5 vehicle is the largest on the eastbound of NB-HCE. It is also
observed that the heaviest truck is Class 10, a 7 axle truck with Gross Vehicle Weight (GVW) of 164.7 kips. The average speed of the trucks is around 33.6 to 41.5 mph.

Figure 3.12 shows percentage of the truck volume by total weight. The graph shows that about 2.5 percent of the trucks on the bridge weigh over 80 kips. For Class 7 vehicles as shown in Figure 3.13, 49% of them were loaded beyond 80 kips. Furthermore, as shown in Figure 3.14, the overweight percentage (>80 kips) is about 12% for Class 9 vehicles. In addition, the hourly distribution of the trucks was also investigated. As shown in Figure 3.17 (a), 65% of the trucks travels on NB-HCE between 6am to 4pm, and the heaviest observed truck with GVW of 164.7 kips was at 2:27 pm on 03/22/16 (Tuesday). The heaviest trucks are also assumed to travel between 6am to 5pm when the majority of trucks are traveling eastbound on NB-HCE.

### Table 3.1 WIM Data Summary on Eastbound of NB-HCE (Outside Interchange 14C)

<table>
<thead>
<tr>
<th>Class</th>
<th>Axle</th>
<th>Count</th>
<th>Percentage (%)</th>
<th>Avg. Weight (kips)</th>
<th>Max Weight (kips)</th>
<th>Standard Deviation (kips)</th>
<th>Avg. of Upper 20% (kips)</th>
<th>Avg. Speed (mph)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>2 or 3</td>
<td>13372</td>
<td>19.4%</td>
<td>43.2</td>
<td>107.6</td>
<td>12.5</td>
<td>60.4</td>
<td>41.5</td>
</tr>
<tr>
<td>5</td>
<td>2</td>
<td>28096</td>
<td>40.7%</td>
<td>25.4</td>
<td>77.3</td>
<td>7.5</td>
<td>37.2</td>
<td>39.9</td>
</tr>
<tr>
<td>6</td>
<td>3</td>
<td>16014</td>
<td>23.2%</td>
<td>40.0</td>
<td>111.7</td>
<td>13.6</td>
<td>59.9</td>
<td>40.0</td>
</tr>
<tr>
<td>7</td>
<td>4 or 4+</td>
<td>1160</td>
<td>1.7%</td>
<td>75.7</td>
<td><strong>144.4</strong></td>
<td>24.5</td>
<td>107.0</td>
<td>35.9</td>
</tr>
<tr>
<td>8</td>
<td>3 or 4</td>
<td>2243</td>
<td>3.2%</td>
<td>39.0</td>
<td>125.5</td>
<td>14.0</td>
<td>60.5</td>
<td>37.0</td>
</tr>
<tr>
<td>9</td>
<td>5</td>
<td>6267</td>
<td>9.1%</td>
<td>53.6</td>
<td>124.2</td>
<td>20.1</td>
<td>83.9</td>
<td>35.7</td>
</tr>
<tr>
<td>10</td>
<td>6 or 6+</td>
<td>200</td>
<td>0.3%</td>
<td>59.8</td>
<td><strong>164.7</strong></td>
<td>29.5</td>
<td>106.9</td>
<td>35.8</td>
</tr>
<tr>
<td>11</td>
<td>5</td>
<td>132</td>
<td>0.2%</td>
<td>33.6</td>
<td>91.3</td>
<td>16.3</td>
<td>60.8</td>
<td>39.1</td>
</tr>
<tr>
<td>12</td>
<td>6</td>
<td>261</td>
<td>0.4%</td>
<td>22.4</td>
<td>106.7</td>
<td>12.9</td>
<td>41.4</td>
<td>38.9</td>
</tr>
<tr>
<td>13</td>
<td>7 or 7+</td>
<td>131</td>
<td>0.2%</td>
<td>45.1</td>
<td>125.1</td>
<td>25.9</td>
<td>89.6</td>
<td>33.6</td>
</tr>
</tbody>
</table>
Figure 3.12. Weight Histogram for All trucks

Figure 3.13. Weight Histogram for Class 7 Vehicles
Figure 3.14. Weight Histogram for Class 9 Vehicles

Figure 3.15. Weight Histogram for Class 10 Vehicles
Figure 3.16. Hourly Density for Trucks
Figure 3.17. Hourly Density for Trucks
3.3. Newark Bay Bridge

The Newark Bay Bridge is a bridge crosses over the Newark Bay connecting the cities of Newark and Bayonne, New Jersey. The bridge was constructed in 1956 as part of New Jersey Turnpike (I-95) network. The bridge provides the access from the New Jersey Turnpike main line at interchange 14 to Lower Manhattan in New York City via the Holland Tunnel. Newark Bay Bridge contains three different types of superstructures: Floor-Beam Spans (Type A), Beam Span (Type B), and Main Truss Span (Type C). The layout of each type of spans was shown in Figure 3.18.

A comprehensive field testing was conducted to understand the behavior of the Newark Bay Bridge and collect the data for the model calibration. The bridge was monitored using an array of sensors, including LDV, strain transducer, foil strain gages, and WIM sensors, to understand truck loading on the bridge under normal traffic condition and the structural behavior under the truck loading as shown in Figure 3.19 and Figure 3.20. The results of the field tests and monitoring were used to improve accuracy of analytical modeling. The strain transducers and the LDV were placed on various bridge components to collect data under truck loading. With the WIM sensors, information about truck traffic on the bridge was gathered and the results were used as an input for the analytical studies. The truck traffic information gathered using the WIM
sensors can also be used for future design and maintenance of bridges, since the data includes detailed information of the truck traffic.

Figure 3.18. Newark Bay Bridge

Figure 3.19. Field instrumentation equipment

The helium neon Class II laser head, the decoder unit and the reflective target of LDV are shown in Figure 3.19 (a), Figure 3.19 (b), and Figure 3.19 (c), respectively. The
laser head is mounted to a tripod that is positioned underneath the target. The reflective target, typically retro-reflective tape, provides strong signal. The STS system consists of a main processing (Figure 3.19 e), junction boxes (Figure 3.19 f), and strain transducers (Figure 3.19 c) or foil strain gauge completion unit (Figure 3.19 h). The general purpose standard 350 Ω foil gauges (Figure 3.19 g) used here were connected to the STS system with quarter arm foil strain gage completion unit. The permanent Weigh-In-Motion (WIM) system was installed on the east abutment of the bridge to collect traffic information regarding traffic volume, traffic pattern, and truck weight. The piezoelectric sensors were installed on all 4 traffic lanes on the abutment. The east bound sensors were installed approximately about 120-ft away from the east abutment and the west bound sensors were installed approximately about 60-ft away from the east abutment. Layout of the WIM sensors is shown in Figure 3.20.

Figure 3.20. WIM sensor locations
Span W14 was selected for field instrumentation and testing. 7 strain transducers, 5 reflective tapes and 6 foil strain gauges were installed as shown in Figure 3.21. The foil strain gauges were installed on selected fatigue critical details, which is the connection of end floor beam and stringers in this case. Two locations were chosen as shown in Figure 3.22. The detail information of each sensor was listed in Table 3.2. The configuration of testing truck was shown in Figure 3.23. Both static test and dynamic test (5 mph and 30 mph) were performed on the selected span.

Figure 3.21. Instrumentation on Span W14 (Type A span) (ft)

Figure 3.22. Cross section of instrumented Span W14 (Type A span) (ft)
## Table 3.2 Location of each sensor

<table>
<thead>
<tr>
<th>Sensor ID</th>
<th>Sensor type</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>B3216</td>
<td>Strain Transducer</td>
<td>on North Girder, 14 ft from FLB3</td>
</tr>
<tr>
<td>B3238</td>
<td>Strain Transducer</td>
<td>on South Girder, 14 ft from FLB3</td>
</tr>
<tr>
<td>B3679</td>
<td>Strain Transducer</td>
<td>on FLB3, 1 ft from S6</td>
</tr>
<tr>
<td>B3680</td>
<td>Strain Transducer</td>
<td>on FLB4, 0.3 ft from S6</td>
</tr>
<tr>
<td>B3684</td>
<td>Strain Transducer</td>
<td>on S4, 14 ft from FLB 4</td>
</tr>
<tr>
<td>B3685</td>
<td>Strain Transducer</td>
<td>on FLB4, 0.3 ft from S6</td>
</tr>
<tr>
<td>Q2163</td>
<td>Foil strain gage</td>
<td>S4 and FLB6 connection, on the top of bottom flange of S4</td>
</tr>
<tr>
<td>Q2165</td>
<td>Foil strain gage</td>
<td>S4 and FLB6 connection, on the top of top flange of FLB6</td>
</tr>
<tr>
<td>Q2167</td>
<td>Foil strain gage</td>
<td>S4 and FLB6 connection, on the stiffener of FLB6 under S4</td>
</tr>
<tr>
<td>Q2164</td>
<td>Foil strain gage</td>
<td>S3 and FLB6 connection, on the top of top flange of FLB6</td>
</tr>
<tr>
<td>Q2166</td>
<td>Foil strain gage</td>
<td>S3 and FLB6 connection, on the stiffener of FLB6 under S3</td>
</tr>
<tr>
<td>Q2168</td>
<td>Foil strain gage</td>
<td>S3 and FLB6 connection, on the top of bottom flange of S3</td>
</tr>
</tbody>
</table>

Figure 3.23. Configuration and axle weights of calibration truck
3.4. Railway Bridge Instrumentation (NJ Transit)

In this section, three open-deck plate girder railway bridge spans with span lengths of 12.2 m (40 ft) (Bridge A), 17.7 m (60 ft) (Bridge B), and 26.8 m (88 ft) (Bridge C) were selected for investigation as examples of a proposed probabilistic approach for fatigue assessment. Due to the fact that two girders for the bridges under investigation support only one of the tracks, the girders herein are described as fracture-critical members.

3.4.1. Bridges Selected

Three NJ Transit passenger rail line bridges were evaluated in this study. They are all open-deck plate girder bridges. Figure 3.24 shows three bridges that were selected based on the freight rail car traffic use of these bridges. These bridges are also selected for future inclusion in New Jersey’s 286-kip (kilo-pound) rail network.
3.4.2. Bridge A: Raritan Valley Line MP 31.15

The Raritan Valley Line MP 31.15 Bridge is a four-span bridge with a total span length of 164.5 ft over Middle Brook. The bridge, built in 1902 with a superstructure fabricated with open-hearth steel, is located on the NJ Transit Raritan Valley Line. Based on the latest inspection report, the controlling member is the north girder below Track 2 at Span 2 (Chas. H. Sells, Inc, 2007). An elevation view of the bridge and the location of the critical span from the load-rating calculations are shown in Figure 3.25.
From Newark southwest to Cranford, the Raritan Valley Line follows the former right of way of the Lehigh Valley Railroad. West from Cranford, the line follows the main line of the former Central Railroad of New Jersey. Historically, CNJ trains ran on this line, as part of its Lehigh-Susquehanna Division, from Scranton, Wilkes-Barre, Allentown, Bethlehem and Easton in eastern Pennsylvania, through Elizabeth and Bayonne to Jersey City. Until 1967, CNJ service terminated at the company's Communipaw Terminal in what is today Liberty State Park. This station, which was also served by Reading Company trains to Philadelphia and B&O service to Washington, D.C., and beyond, had connections by chartered bus or ferry into Manhattan (the ferries serving the financial district). On the Raritan Valley Line, F40PH-2CAT, GP40PH-2 (A and B) GP40FH-2, Alstom PL42AC, and GE P40DC diesel locomotives haul Comet-series coaches and, since late 2008, Bombardier multilevel coaches. Most trains now consist of an Alstom PL42AC and a 6-car set of multilevel.
Figure 3.25. General View of the Bridge from Inspection Report Cycle 4 Source: Chas. H. Sells, Inc., 2007

Figure 3.26 and Figure 3.27 show a layout of the bridge and locations of the installed strain transducers on Span 2 and Span 3, respectively, of the Raritan Valley Line MP 31.15 Bridge. The strain transducers in Figure 3.26 were installed at the midspan section, first cutoff point (about 5 ft from the support end of the girder), and second cutoff point (about 8 ft and 8.5 in. from the support of the girder). For Span 3 in Figure 3.27, strain transducers are installed at the midspan of each girder under the active track (G5 through G8).
3.4.3. Bridge B: Bergen County Line MP 5.48 (HX Draw)

The HX Draw is a bascule bridge carrying the New Jersey Transit Bergen County Line and Pascack Valley Line across the Hackensack River between Secaucus and East
Rutherford, New Jersey. The bridge is also known as “The Jack-Knife” because of its unusual method of opening.

The HX Draw Bridge is a 17-span bridge with a total length of 1,095 ft over the Hackensack River. This structure carries two active tracks over the Hackensack River between Secaucus in Hudson County, and East Rutherford in Bergen County, New Jersey. Based on the Inspection Report Cycle 4, the controlling member is the north girder below Track 2 at Span 3 (HNTB Corporation, 2006). A general view of the bridge is shown in Figure 3.28.

![General View of Span 3, Span 9 and Span 12 of the Bergen County Line MP 5.48 (HX Draw) Bridge over Hackensack River Bridge from Inspection Report Cycle 4](Source: HNTB Corporation, 2006)

The Bergen County Line (or Bergen Line) is a commuter rail line and service owned and operated by New Jersey Transit. The line loops off the Main Line between the Meadowlands and Glen Rock, with trains continuing in either direction along the Main
Line. Some trains of Metro-North Railroad's Port Jervis Line also operate over the line. The Norfolk Southern Railway provides freight service along the line via trackage rights. As on the Main Line, trains are powered by diesel locomotives that are operated push-pull.

The Pascack Valley Line is a commuter rail line operated by the Hoboken Division of New Jersey Transit. The line runs north from Hoboken, New Jersey, through Bergen County and into Rockland County, New York, terminating at Spring Valley. Additional service within New York is operated under contract with Metro-North Railroad. The line is named for the Pascack Valley region that it passes through in northern Bergen County. The line parallels the Pascack Brook for some distance.

The instrumentation at this structure is mainly focused on the north girder in Span No. 3 (approach span) below Track 2. This girder has the lowest as-inspected ratings, which are E27 at normal level and E43 at the maximum level. Strain transducers were also installed on the south girder under Track 2, the girders under Track 1, and the girders in Span 2, to gain a thorough understanding of the structural response and load distribution of the bridge. Based on the preliminary calculation performed by NJ Transit, the end floorbeam of Span 9 (approach span) and stringers in Span 12 (tower span) were also selected for testing. Figure 3.29 through Figure 3.33 show the locations of sensors on the desired spans specified by the sensor number.
Figure 3.29. Layout of the Strain Transducers Installed in Span 2 of the Bergen County Line MP 5.48 (HX Draw) Bridge

Figure 3.30. Layout of the Strain Transducers Installed in Span 3 of the Bergen County Line MP 5.48 (HX Draw) Bridge

Figure 3.31. Details of Span 3 of Bergen County Line MP 5.48 (HX Draw) Bridge
Figure 3.32. Layout of the Strain Transducers Installed in Span 9

Figure 3.33. Layout of the strain transducers installed in Span 12 of the Bergen County Line MP 5.48 (HX Draw) Bridge

3.4.4. Bridge C: North Jersey Coast Line MP 0.39 (River Draw)

The River Draw Bridge is a steel-truss swing bridge over the Raritan River with 28 deck-girder approach spans and a total length of 2,918 ft. The bridge was erected in
1906 and carries two electrified tracks between Perth Amboy and South Amboy, New Jersey. Based on the Inspection Report Cycle 4, the controlling member is the 88 ft approach span girder and the swing-span-end floorbeam connection (Lichtenstein Consulting Engineering, Inc., 2006). Figure 3.34 shows the elevation view of the bridge.

![Figure 3.34: North Elevation of East Approach Span 1 to 18 of the River Draw Bridge](image)

The North Jersey Coast Line is a New Jersey Transit commuter rail service between New York Penn Station or Hoboken Terminal and Bay Head, New Jersey, electrified as far as Long Branch. Most trains operate between New York Penn Station and Long Branch with frequent rush-hour service and hourly local off-peak service. Diesel shuttle trains between Long Branch and Bay Head meet these electric trains. New York to Long Branch service operates hourly on weekends, with bi-hourly diesel shuttle service between Long Branch and Bay Head. Full hourly service operates during the peak summer season.
Since September 9, 1991, five round-trip diesel trains have run weekdays from Bay Head to Hoboken Terminal using the Waterfront Connection. Passengers can reach New York via the Northeast Corridor Line at Newark, or PATH at Newark or Hoboken. Some electric trains terminate at South Amboy or Aberdeen-Matawan and make all stops from New York Penn Station, providing local service for the Northeast Corridor stops of Rahway, Linden, Elizabeth, and North Elizabeth during rush hours.

The sensor instrumentation installed at this structure focuses mainly on Girder 5 through Girder 8 in Span 26 (88 ft long) since these members rated lowest as inspected (E47 at normal level and E70 at maximum level). Figure 3.35 shows an elevation view and an underneath view of a typical approach span (88 ft long). Sensors were also instrumented on the end floorbeam of Span 20. Figure 3.36 and Figure 3.37 show the locations of the installed strain transducers on Span 26 and Span 20, respectively. It is important to note that sensor 3236 is located at the midspan of the end floorbeam, while sensors 3228, 3217, and 3229 are located at the cutoff point.

![Sensor location](image)

Figure 3.35. Typical approach span (length = 88 ft) of the North Jersey Coast Line MP 0.39 (River Draw) Bridge; (a) Elevation View, and (b) Underneath View
Figure 3.36. Location of Strain Transducers Installed on Various Locations in Span 26 of the North Jersey Coast Line MP 0.39 (River Draw) Bridge

Figure 3.37. Location of Strain Transducers in Span 20 of the North Jersey Coast Line MP 0.39 (River Draw) Bridge
CHAPTER 4

STRUCTURAL ANALYSIS AND FATIGUE LIFE
ASSESSMENT

4.1. Current AASHTO MBE Fatigue Evaluation of Steel Girders

In order to perform bridge life cycle cost for estimation of economic impact of
overweight trucks, the service life is needed. The evaluation method for steel bridges
employed is based on the MBE (AASHTO 2011). For this method, the remaining life was
determined by:

\[ Y = \frac{R_R A}{365n (ADTT)_{SL} \left[ (\Delta f)_{eff} \right]^3} \]

Eq. 4.1

where:

- \( R_R \) = Resistance factor specified for evaluation, minimum, or mean fatigue life as given in
  Table 7.2.5.2-1, The Manual for Bridge Evaluation (AASHTO)

- \( A \) = Detail-category constant given in LRFD Design Table 6.6.1.2.5-1, AASHTO LRFD
  Design Specifications, 2010

- \( N \) = Number of stress-range cycles per truck passage estimated, 1 in this case
(AADT)_{SL} = \text{Average number of trucks/day in a single lane averaged over the fatigue life} \\
(\Delta f)_{\text{eff}} = \text{The effective stress range} \\

The manual identifies two levels of fatigue evaluation: the infinite-life check and the finite-life calculations. The bridge details are subject to the more complex finite-life fatigue evaluation when they fail the infinite-life check. The steps of the evaluation is summarized and shown in Figure 4.1.
4.2. Finite Element Modeling of Bridges

4.2.1. Material Properties

In the AASHTO Guide Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges (AASHTO 2012), recommended material properties of existing bridges are provided based on the year in which the structure was built. In compression, the concrete strength of the portion built in early 1950’s was taken as 2,500psi. The concrete strength of the widened slab in 2006 was determined from laboratory testing results by KeyTech as 3,900 psi. Reconstruction of the original deck slabs began in 2012 for the westbound roadway under Contract T100.125. Material properties used for the reconstructed deck was available from laboratory testing of concrete samples from Contract T100.125, Stage 7. The 28-day compressive strength of new deck concrete was taken as 7500 psi. The modulus of rupture, $f_r$, and the modulus of elasticity, $E_c$, were taken as $7.5\sqrt{f'_c}$ and $57,000\sqrt{f'_c}$, respectively, where $f'_c$ is the concrete compressive strength in psi. Tensile strength of the early-age concrete was taken as 10 percent of the compressive strength.

The modulus of elasticity of the steel beams and the reinforcing bars $E_s$ is taken as 29,000 ksi. The Poisson’s ratio, $\nu_s$, is taken as 0.3. Since, in contrast to the concrete, inelastic behavior of the steel beam and the reinforcing bars are not expected under
normal traffic, inelastic material behavior for the steel was not considered in the material model.

Under the service loading conditions, it is assumed that the steel frames are not expected to behave beyond the elastic range. Cured concrete can experience stress level beyond the linear elastic range, but it appears to experience limited inelastic behavior. For the fresh concrete, large inelastic behavior is expected, especially in the tension side. The modulus of elasticity of the fresh concrete is significantly lower than the cured concrete, and the effects of fresh concrete on the overall behavior of the bridge is considered to be minimal. Thus, in the study, concrete was assumed to behave linear elastic during the analysis. Because of the very low concrete strength and modulus of elasticity, the early-age concrete was not expected to resist high forces under the truck loading even at large strain.

4.2.2. Element Selection and Analysis Procedure

The finite element analysis models developed for unit 2 and 11 of New Jersey Turnpike Structure N6.49 are shown in Figure 4.2 and Figure 4.3. A 4-node shell element (S4) was selected for the concrete slab. Element type S4 in the FE model (ABAQUS) is a fully integrated, finite membrane strain shell element. Simpson’s rule is used to calculate the cross-sectional behavior of the shell elements.
For the steel girders, floor beams, stringers, and diaphragms, a 2-node linear beam element (B31) was selected in the model. Element type B31 is a first-order, shear deformable beam elements, meaning that shear deformation as well as flexural deformation can be accounted in the analysis.

With the developed model, a set of point loads simulating one or multiple truck loading was applied on the concrete deck. A multiple load case analysis was adopted to apply the truck load at various locations of the concrete deck. This analysis method in ABAQUS allows for the change of applied loads and boundary conditions. The results can then be scaled and linearly combined during post-processing.

The strains at various locations under the calibration truck loading were determined from the FE model and were compared with the field test data. Next, the strain data in the concrete deck was obtained and compared for each construction stage of each modeled structure.

(a) Concrete Slab    (b) Steel Frame
Figure 4.2. FE Model for Structure N6.49, Unit 2
4.2.3. Simple Span Steel Multi-Beam Bridge

Due to the large inventory of simple span steel multi-beam bridges, three bridges were selected to represent the bridges with short, average and long span lengths. The bridges selected for analysis are summarized in Table 4.1 include the NJ Route 34 over I-195 and NJ Route 138 (Str. No. 1307-155), Route I-295 over Clements Bridge Road (Str. No. 0428-164) and Route US 202 over County Route 605 (Queens Road) and Alexauken Creek. Respectively, these structures represent average span length, short span length and long span length bridges.
Table 4.1 Summary of selected bridges

<table>
<thead>
<tr>
<th>Structure No.</th>
<th>Span Length (ft)</th>
<th>Total Width (ft)</th>
<th>Skew Angle</th>
<th>Number of Girders</th>
<th>Girder Spacing (ft)</th>
<th>Slab Thickness (in)</th>
<th>Girder Depth (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1307-155</td>
<td>86.5</td>
<td>112.75</td>
<td>21</td>
<td>16</td>
<td>7.25</td>
<td>10</td>
<td>54</td>
</tr>
<tr>
<td>0428-164</td>
<td>62.5</td>
<td>56.50</td>
<td>11</td>
<td>7</td>
<td>8.75</td>
<td>9.5</td>
<td>36</td>
</tr>
<tr>
<td>1023-153</td>
<td>106</td>
<td>57.25</td>
<td>11</td>
<td>8</td>
<td>7.42</td>
<td>8</td>
<td>57.5</td>
</tr>
</tbody>
</table>

The modulus of elasticity of the steel girders and diaphragms are taken as 29,000 ksi, and the Poisson’s Ratio taken as 0.3. For the concrete slab, the elastic modulus and Poisson’s Ration were taken as 3,600 ksi and 0.18, respectively. Neither the concrete nor steel components are expected to deform beyond the elastic range due to design loads, so only elastic material properties are considered.

The modeled bridges are shown in Figure 4.4. A 4-node shell element (S4) was selected for the concrete slab. Element type S4 in ABAQUS is a fully integrated, finite membrane strain shell element. Simpson’s rule is used to calculate the cross-sectional behavior of the shell elements. For the steel girders, floor beams, stringers, and diaphragms, a 2-node linear beam element (B31) was selected in the model. Element type B31 is a first-order, shear deformable beam elements, meaning that shear deformation as well as flexural deformation can be accounted in the analysis. The slab elements are connected to beam elements using the multi-point constraint, or MPC, which rigidly constrains rotations and translations of the slave node to those of the master node.
With the developed model, a set of point loads simulating one or multiple truck loading was applied on the concrete deck. A multiple load case analysis was adopted to apply the truck load at various locations of the concrete deck. This analysis method in ABAQUS allows for the change of applied loads and boundary conditions. The results can then be scaled and linearly combined during post-processing.

Figure 4.4. Finite element models for (a) Structure No. 1307-155, and (b) Structure No. 0428-164
Maximum Stress Range Induced by AASHTO Fatigue Trucks and Girder

Distribution Factor

The load cases in the FEA are generated to represent an HS-20 truck for the fatigue limit state, which uses a fixed distance of 30 ft. between the middle and rear axles, as shown in Figure 4.5. A dynamic impact factor of 0.15 for fatigue is included. For interior girders, the truck is centered over the girder. For exterior girders, the truck is positioned 1 ft from the sidewalk or the inside face of the parapet. For structures with different girder sections at splice locations, bottom flange stress is evaluated at both midspan and the cutoff location. Figure 4.6, for example, shows that the maximum cutoff stress determined from the FE analysis was 2811 lbf/in² for exterior girders on Str. No. 1307-155. For comparison, the AASHTO method was used to compute the girder stresses and distribution factors manually, shown in detail in Appendix A. The results, given in Table 4.2 through Table 4.4, show that AASHTO stresses for one lane loaded are on average 64 percent higher than the stress results from the FE analysis. From the analysis, the bridge with Str. No. 1023-153 has the maximum stress ranges under the same fatigue truck loading.
Figure 4.5. HS-20 design truck for fatigue

![HS-20 Design Truck](image)

Figure 4.6. Exterior girder bottom flange stress at splice cutoff, located 24 ft. from midspan, due to HS-20 including impact (Str. No. 1307-155)

![Stress Graph](image)

Table 4.2 Str. No. 1307-155

<table>
<thead>
<tr>
<th>Member</th>
<th>Span Position</th>
<th>GDF</th>
<th>Stress (lbf/in²)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>FEM</td>
<td>AASHTO</td>
</tr>
<tr>
<td>Exterior</td>
<td>Midspan</td>
<td>0.29</td>
<td>0.51</td>
</tr>
<tr>
<td>Girder</td>
<td>Cutoff</td>
<td>0.33</td>
<td>0.51</td>
</tr>
<tr>
<td>Interior</td>
<td>Midspan</td>
<td>0.24</td>
<td>0.41</td>
</tr>
<tr>
<td>Girder</td>
<td>Cutoff</td>
<td>0.27</td>
<td>0.40</td>
</tr>
</tbody>
</table>
Table 4.3 Str. No. 0428-164

<table>
<thead>
<tr>
<th>Member</th>
<th>Span Position</th>
<th>GDF</th>
<th>Stress (lbf/in²)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>FEM</td>
<td>AASHTO</td>
</tr>
<tr>
<td>Exterior Girder</td>
<td>Midspan</td>
<td>0.39</td>
<td>0.66</td>
</tr>
<tr>
<td></td>
<td>Cutoff</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>Interior Girder</td>
<td>Midspan</td>
<td>0.33</td>
<td>0.54</td>
</tr>
<tr>
<td></td>
<td>Cutoff</td>
<td>--</td>
<td>--</td>
</tr>
</tbody>
</table>

Table 4.4 Str. No. 1023-153

<table>
<thead>
<tr>
<th>Member</th>
<th>Span Position</th>
<th>GDF</th>
<th>Stress (lbf/in²)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>FEM</td>
<td>AASHTO</td>
</tr>
<tr>
<td>Exterior Girder</td>
<td>Midspan</td>
<td>0.46</td>
<td>0.594</td>
</tr>
<tr>
<td></td>
<td>Cutoff</td>
<td>0.54</td>
<td>0.594</td>
</tr>
<tr>
<td>Interior Girder</td>
<td>Midspan</td>
<td>0.38</td>
<td>0.637</td>
</tr>
<tr>
<td></td>
<td>Cutoff</td>
<td>0.48</td>
<td>0.625</td>
</tr>
</tbody>
</table>

4.2.4. Simple Span P/C Multi-Beam Bridges

Before the finite element modeling, a P/C bridge with span length of 60 ft was designed according the AASHTO LRFD Bridge Design Specification using an in-house design program. Using the information of this designed bridge, a 3-dimensional finite element model was developed for this bridge using ABAQUS. The following sections explains each model element used as well as the constraint/release conditions applied.

**Geometric Modeling**

The structure of the bridge can be modeled using nodes, elements and sections provided by Abaqus.

**Elements**
The ABAQUS program library itself provides numerous options for selection of geometric elements. Out of which for this project, Beam and Shell elements have been identified as the most appropriate and dependable for bridge related problems.

Beam Element

The fact that due to its one-dimensional characteristics, definition of stringers and girders can be modeled with the help of Beam Element. Two node beam element is used to model girder. It should be noted here that the segment generally won’t deform out of plane. This fact/condition can be considered a constraint while defining problem statement. This restriction ensures that the plane section will remain the plane section until whole analysis is done. Figure 4.7 below depicts the beam element with various integration localities. It is observed that the more minutely modeling is done the more preciseness is achieved. This means the accuracy of the outcomes depends upon the degree of discretization. But the problem a user encounters during working with highly discretized bridge is, slow processing of the program.

Figure 4.7. Integration points of two-node linear beam B31

Shell Element
For defining models for concrete bridge decks in FE package, a shell element has been used very commonly due to less magnitude of the dimensions in terms of thickness of the slabs in comparison to its other dimensions. The inbuilt library resource for the shell element is very rich. It has been found that the majority of it are of four node type of shell elements. The element is of a completely integrated, general purpose type with finite-membrane-strain shell element that allows in-plane bending able to permit planar bending (in plane bending). In addition to this, it also allows deflection/deformation in transverse direction. It should be noted that this element is considered to be a thick shell theory. It is obvious that with the increase in the thickness value, the predicted pattern for thin shell explained by the Kirchhoff-Love hypothesis gets dulled. This is very certain as this hypothesis banks upon the condition of homogeneous isotropic materials. Thus implementing it for thick-shelled, laminated anisotropic materials, such as the steel reinforced concrete bridge deck will not yield a proper results. The four-node shell element has six degrees of freedom at each node and four integration points for each element. Figure 4.8 illustrates the integration point and nodes used by the four-node shell elements.
Figure 4.8. Integration points of a four-node shell element (Abaqus)

Tendon Element

For the modeling of prestressing tendons, truss element is introduced to the model. A 2-node linear displacement truss element T2D2 is used to model the tendon elements.

Sections

It is extremely important and inevitable to incorporate the properties of an element into the model. The definitions of the properties and characteristics are incorporated to the model using sections. From the numerous available sections, following types are associated with the current analysis. A section corresponds to a specific material. After identifying section, certain sets of elements are imposed on related section.

Beam Section
Beam section is used to define the cross-section for beam elements when numerical integration over the section is required. Girder of the bridge is modeled using I beam section. The integration points for I beam section in stress are shown in Figure 4.9.

Figure 4.9. Integration points on I section

Shell Section

The use of the shell section is to specify or define a shell cross section in Abaqus input file. The thickness of deck in bridge model is provided using this section.

Rebar Layer

For modeling all the steel reinforcement inbuilt rebar element was implemented. In ABAQUS this element is capable of providing embedment within beam or shell elements. Steel Reinforcing rebars are placed in both transverse layer and longitudinal layers within the shell.

Solid Section
The prestressing tendons in girder are defined using solid section.

**Constraint Elements**

The model built in ABAQUS model is an assembly of individual structural components. Such as beams, shells, studs, etc. Unless these constituent elements are merged to build a bridge, analysis cannot be run. And for joining these elements, constraint elements have to be employed. The most generally used constraint element used a multipoint Constraint (MPC). Rigid joints simulating a beam is ensured by beam MPC. The simulation is between two nodes. And this simulation is mostly employed for slab and beam elements to generate composite action. The displacement and rotation of one node is directly associated with those of connected node.

**Material Properties**

Material modeling consists of defining the properties of the different materials used in the structure. Each material definition is actually a combination of various independent characteristics.

- Density
- Elastic Modulus
- Poisson’s ratio
- Thermal expansion
- Dependent variables
- User defined field

For accurate results the properties of these materials must be determined and input into the program.

**Loading**

The standard fatigue loading that defined by AASHTO Bridge Design Specification was used. In addition, the loading was placed at the location where the maximum stress is produced at the tendons. The initial prestressing force is applied using two commands from Abaqus: initial condition and prestress hold.

**Finite Element Analysis**

Using the information described above, a 3-D FE model was developed as shown in Figure 4.10 and Figure 4.11. Applying the HS-20 fatigue loads, the stress that obtained from the prestressing tendon at the midspan is 140.4 ksi. The initial stress was 200.16 ksi. Therefore, the stress range of prestressed tendon for this bridge is 59.76 ksi under AASHTO fatigue truck model.

![3-D FE model for prestressed concrete bridge](image)

Figure 4.10. 3-D FE model for prestressed concrete bridge
4.3. Model Verification

4.3.1. FEM Calibration of New Jersey Turnpike Structure N6.49

The target instrumented bridges were built in early 1950’s and the eastbound roadway was widened in 2005 to accommodate the increase of the traffic volume. Calibration testing was performed on June 10, 2015. Using a truck of known axle weight and spacing shown in Figure 4.12, the FEM was calibrated such that the predicted response of structural members matched the field measured response under the truck load. Figure 4.13 and Figure 4.14 show that the strains predicted by the FEM compare very well to experimental data for the multi-stringer spans (Unit 2) and floorbeam span (Unit 11).
Figure 4.12. Axle configuration of calibration truck (6/10/2015)
Eastbound Roadway

Figure 4.13. Comparison of measured strains to FEM-predicted strains for the calibration truck in the eastbound right lane of Unit 2 (tested on 6/10/2015)
Figure 4.14. Comparison of measured strains to FEM-predicted strains for the calibration truck in the westbound left lane of Unit 11.
4.3.2. FEM Calibration of Newark Bay Bridge

Various element types were used to validate and ascertain the accuracy of the FE model, the FE analysis results was compared with data collected from field tests and various items were adjusted to improve the model including 1) section properties, 2) material behavior, 3) boundary conditions and 4) interaction between different members. The modulus of elasticity of the steel girder, floorbeam, and stringer, $E$, and Poisson’s Ratio, $\nu_s$, is used as 29,000 ksi and 0.3, respectively. It is noted that the steel girders, floorbeams, and stringer are expected to undergo deformation within the elastic region only and therefore the inelastic behavior of the steel material was not considered. The compression strength for concrete was considered as 8000 $lbf/in^2$. The established model is shown in Figure 4.15 and the comparisons shown in Figure 4.16 confirm the good agreement between the FE model and field testing data.
Figure 4.16. Comparison of strain profiles under truck loading on south lane (17 ft from curb): (a) main girder Sensor B3238; (b) S4 Sensor B3684; (c) floorbeam FLB 4 Sensor B3685;

4.4. **Estimation of Remaining Fatigue Life Based on the AASHTO MBE**

4.4.1. **Fatigue Truck Model Analysis**

A fatigue truck is typically used to represent truck traffic condition at the bridge site. The fatigue truck models provided in the AASHTO LRFD (AASHTO 2010) is shown in Figure 4.17 (a) with a 6 ft (1.82 m) axle width. However, the AASHTO MBE indicate that when the GVVW distribution at the investigated site is available, an effective
GVW can be determined by Eq. 4.2 and this effective GVW can be used to modify the GVW of AASHTO fatigue truck model.

\[ W_{eq} = \left( \sum f_i W_i^3 \right)^{1/3} \quad \text{Eq. 4.2} \]

where \( f_i \) is the frequency of occurrence of trucks with a GVW weight of \( W_i \).

After deriving the effective gross vehicle weight by analyzing the WIM data, the gross weight of the modified AASHTO fatigue truck will be distributed proportionally into each axle as illustrated in Figure 4.17 (b).

Figure 4.17. (a) AASHTO fatigue truck model (b) modified AASHTO fatigue truck model

WIM data collected at this bridge site were used in this study to develop the modified fatigue truck model in this study. The distribution of gross weight and truck distribution in each lane were shown in Figure 4.18. The effective gross weight for fatigue truck calculated in this case was 61.4 kips. The maximum stress range caused by the site specific fatigue trucks and the estimated remaining fatigue life was shown in Table 4.5.
Table 4.5 Stress range caused by site specific fatigue trucks and estimated remaining fatigue life

<table>
<thead>
<tr>
<th>Structural member</th>
<th>Critical location</th>
<th>$\Delta f$ (ksi)</th>
<th>$\Delta f_{IM}$ (ksi)</th>
<th>$\Delta f_{eff}$ (ksi)</th>
<th>({$\Delta f_{eff}$})_{max} (ksi)</th>
<th>Remaining fatigue life prediction by AASHTO</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder</td>
<td>Main girder and FLB4 intersection</td>
<td>1.87</td>
<td>2.15</td>
<td>1.46</td>
<td>2.91</td>
<td>Infinite</td>
</tr>
<tr>
<td>Floorbeam</td>
<td>End floor beam and S6 intersection</td>
<td>3.93</td>
<td>4.52</td>
<td>3.06</td>
<td>6.12</td>
<td>Infinite</td>
</tr>
<tr>
<td>Stringer</td>
<td>Midspan of stringer</td>
<td>4.65</td>
<td>5.35</td>
<td>3.62</td>
<td>7.24</td>
<td>57 (evaluation life), 32 (minimum life), 83 (mean life)</td>
</tr>
</tbody>
</table>
4.4.2. Fatigue Loading on Bridge Based on WIM data

Site 78D WIM data was used to for the analysis to obtain the fatigue loading on bridges. Figure 4.23 shows a typical From-To matrix from Rainflow counting method based on WIM data. Figure 4.20 illustrate the stress range-cycle histogram for the same dataset. Figure 4.21 shows the cycles per truck passange for bridges with span lengths from 10 ft to 300 ft. For bridge with a span length less than 100 ft, more than one cycle was observed for each truck passange. Figure 4.22 shows the ratio of effective stress range caused by the truck traffic from WIM data and current AASHTO fatigue evaluation trucks. It’s the normalized fatigue load based on WIM data.

Figure 4.19. From-To matrix for Site 78D (one month 2011-02) 30ft span
Figure 4.20. Stress range histogram for Site 78D (one month 2011-02) 30 ft span

Figure 4.21. Cycle per truck for Site 78D (three month data)
4.5. Probabilistic Fatigue Assessment of Railway Bridges

4.5.1. Finite Element Bridge Models

The bridges that were tested as previously described were also modeled and analyzed by ABAQUS. Various element types were used to verify the accuracy of the FE models. The results of the FE analysis were compared with the data provided by the field test results. Various parameters, including 1) section properties, 2) material properties, 3) boundary conditions and 4) interaction between different members, were adjusted to improve the model. The modulus of elasticity of the steel girder, steel beams and rails \((E)\) and Poisson’s Ratio \((\nu_s)\), are used as 29,000 ksi and 0.3, respectively. It should be noted
that the steel girders, beams, and rails are expected to undergo deformation within the elastic region only therefore, the inelastic behavior of the steel material was not considered. Material properties for Wood-Tie members, such as modulus of elasticity ($E$), and Poisson’s Ratio ($\nu_v$), are assumed to be 1,600 ksi and 0.3, respectively.

Figure 4.23 illustrates an isometric view of the FE model for two of the selected bridges, with various types of elements used to model different structural members of the bridge in the FE model.

![FE Model of Selected Bridge Spans](image)

(a) Bridge B  
(b) Bridge C

**Figure 4.23. FE Model of Selected Bridge Spans**

4.5.2. Model Verification of the Bridge A

Figure 4.24 and Figure 4.25 show the comparison between the FE model and field testing data for the Bridge A. The section modulus used in FE model was modified until the results of the model agreed with the testing data.
Figure 4.24. Comparison of Strain Collected at Midspan for Bridge A
Figure 4.25. Comparison of Strain Collected at Cutoff Point for Bridge A

4.5.3. Model Verification of the Bridge B

Figure 4.26 and Figure 4.27 show the comparison in the results provided by the field tests and the FE model, using static and dynamic testing data, respectively, for Bridge B. As can be seen from the comparison, the results of the FE analysis show a
similar trend toward the experimental data after model calibration, with a difference of 5%. However, the un-calibrated model, with the same boundary conditions and section properties provided by the latest inspection report, did not demonstrate an agreement with the experimental data. From both the field testing and FE model, the South Girder under Track 2 was found to have the maximum stress. Therefore, this was regarded as the critical location in the bridge in this analysis.

Figure 4.26. Comparison of Strain for Bridge B: (a) Second Cutoff Location (b) Midspan

Figure 4.27. Comparison of Strain for Bridge B: (a) Second Cutoff Location (b) Midspan
4.5.4. Model Verification of the Bridge C

Figure 4.28 shows the comparison between the experimental data and the FE analysis results using a calibrated model. The section properties, various boundary conditions, and various connection types were used to calibrate the FE model.

Figure 4.28. Comparison of Strain at Cutoff Locations for Bridge C
4.5.5. Probabilistic Fatigue Approach

In this section, a probabilistic model is proposed for the fatigue evaluation of a railway bridge located on the NJ Transit line. Probabilistic live load and resistance models are developed for this approach. Several random variables are described with provided statistical data, including annual train frequencies \( f_{ti} \), dynamic impact, traffic volumes, freight car weight, and fatigue resistance.

4.5.5.1. Development of Live Load

The probabilistic fatigue load spectra were derived to account for loading uncertainties. In the live load model, the random variables are annual train frequencies \( f_{ti} \), dynamic impact, traffic volume, and freight car loading. Based on previous studies, annual train frequencies are shown to have a lognormal distribution, with a coefficient variation (CoV) of 0.14 (Ebrahimpour et al. 1992). The mean values of the distributions are assumed to be equal to the annual scheduled frequencies. A 3% annual frequency increase was assumed in our study, same as the study by Imam et al. (2008). The statistical information of the annual train frequencies for three bridges is presented in Table 4.6.

The statistical information for dynamic impact was taken from the test program conducted by AAR in the 1950s regarding the impact in 37 girder spans (AREA 1960). In this program, more than 1800 trains ran over instrumented bridges at speeds ranging from
crawling to over 160 km/h (100 mph). According to this database, which is believed to contain the most accurate impact measurements, the best-fit probability distribution for impact was found to be lognormal. The fitted distributions for open-deck spans ranging from 9.1 m (30 ft) to 18.3 m (60 ft) are plotted in Figure 4.29. According to the information provided by NJ Transit, the speed range for passenger trains is between 64.4 km/h (40 mph) and 96.6 km/h (60 mph), while the of freight trains can only reach 32.3 km/h (20 mph).

Table 4.6 Statistical information for Passenger and Freight Train

<table>
<thead>
<tr>
<th>Traffic type</th>
<th>Locomotive type inventory</th>
<th>Railcar type</th>
<th>Annual frequency $f_a$(mean)*</th>
<th>Standard deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Passenger train</td>
<td>F40PH-2CAT/GP40PH-2/GP40FH-2/PL42AC</td>
<td>Comet Coach or Bombardier MultiLevels (6 railcars/train)</td>
<td>9080</td>
<td>1271</td>
</tr>
<tr>
<td>(Bridge A)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Freight train</td>
<td>Six-axle freight locomotive/Four-axle freight locomotive</td>
<td>Coal hopper, Four-axle intermodal, Auto-rack, Four-axle mixed freight (20 railcars/train)</td>
<td>7000</td>
<td>980</td>
</tr>
<tr>
<td>(Bridge A)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Passenger train</td>
<td>F40PH-2CAT/GP40PH-2/GP40FH-2/PL42AC</td>
<td>Comet Coach (6 railcars/train)</td>
<td>13369</td>
<td>1872</td>
</tr>
<tr>
<td>(Bridge B)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Freight train</td>
<td>Six-axle freight locomotive/Four-axle freight locomotive</td>
<td>Coal hopper, Four-axle intermodal, Auto-rack, Four-axle mixed freight (20 railcars/train)</td>
<td>7000</td>
<td>980</td>
</tr>
<tr>
<td>(Bridge B)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Passenger train</td>
<td>F40PH-2CAT/GP40PH-2/GP40FH-2/PL42AC</td>
<td>Comet Coach or Bombardier MultiLevels (6 railcars/train)</td>
<td>14031</td>
<td>1964</td>
</tr>
<tr>
<td>(Bridge C)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Freight train</td>
<td>Six-axle freight locomotive/Four-axle freight locomotive</td>
<td>Coal hopper, Four-axle intermodal, Auto-rack, Four-axle mixed freight (20 railcars/train)</td>
<td>8821</td>
<td>1235</td>
</tr>
<tr>
<td>(Bridge C)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
The uncertainties in passenger volumes were also considered (Kim et al. 2001). The passenger volume is defined as the ratio of the number of passengers to the regular passenger capacity. Therefore, the total weight of trains can be obtained simply by adding the passenger weight to the empty passenger trains. The average weight of one passenger is assumed to be 88.5 kg (195 lbs) (U.S. Department of Health and Human Services, 2012). According to the study of Kim et al., the probabilistic model for passenger volume can be regarded as a lognormal distribution. The statistical information of passenger volumes are provided in Table 4.6, while the distributions are illustrated in Figure 4.30.
During the evaluation of an existing bridge, the freight loading spectrum to which the bridges are currently being subjected is more important than the design load model. In the 1990s, Tobias et al. performed an extensive experiment that measured the loading spectra under current operating conditions. The collected data was analyzed to find the best fit to the test measurements. In order to best reflect the actual train cars currently being used, it has been determined that approximately 60% of simulated trains were coal hoppers, 20% were mixed freights, 7.5% were four-axle intermodal, 3.5% were auto-racks, and 9% were locomotives. Based on the information provided by NJ Transit, each freight train was assumed to have 20 freight railcars, with each railcar containing the same weight. For simplification purposes, considering the large number of simulations, the locomotive cars’ weights are assumed to be deterministic values. All railcar models
used in this analysis are shown in Table 4.7 and Table 4.8. The configuration of rail equipment is shown in Figure 4.31.

### Table 4.7 Rail Equipment used in this Study (Passenger Train)

<table>
<thead>
<tr>
<th>Rail equipment type</th>
<th>( S_O ) (m)</th>
<th>( S_T ) (m)</th>
<th>( S_I ) (m)</th>
<th>Passenger capacity (person)</th>
<th>Car weight ((kN))</th>
</tr>
</thead>
<tbody>
<tr>
<td>F40PH-2CAT</td>
<td>7.1</td>
<td>9</td>
<td>24</td>
<td>N/A</td>
<td>1163</td>
</tr>
<tr>
<td>GP40PH-2</td>
<td>8.1</td>
<td>9</td>
<td>28.25</td>
<td>N/A</td>
<td>1312</td>
</tr>
<tr>
<td>GP40FH-2</td>
<td>8.1</td>
<td>9</td>
<td>25</td>
<td>N/A</td>
<td>1257</td>
</tr>
<tr>
<td>PL42AC</td>
<td>8.5</td>
<td>9.5</td>
<td>33.83</td>
<td>N/A</td>
<td>1281</td>
</tr>
<tr>
<td>Comet Coach</td>
<td>9.7</td>
<td>8.2</td>
<td>49.2</td>
<td>64</td>
<td>445 (empty)</td>
</tr>
<tr>
<td>Bombardier MultiLevels</td>
<td>8.5</td>
<td>8.5</td>
<td>51</td>
<td>128</td>
<td>890 (empty)</td>
</tr>
</tbody>
</table>

### Table 4.8 Rail Equipment used in this Study (Freight Train) (Source: Tobias et al., 1996.)

<table>
<thead>
<tr>
<th>Rail equipment type</th>
<th>( S_O ) (m)</th>
<th>( S_T ) (m)</th>
<th>( S_I ) (m)</th>
<th>Average car load ((kN))</th>
<th>Standard deviation ((kN))</th>
<th>Best fit distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Six-axle freight locomotive</td>
<td>6</td>
<td>7</td>
<td>34</td>
<td>1838</td>
<td>N/A</td>
<td>Deterministic</td>
</tr>
<tr>
<td>Four-axle freight locomotive</td>
<td>8</td>
<td>9</td>
<td>34</td>
<td>1255</td>
<td>N/A</td>
<td>Deterministic</td>
</tr>
<tr>
<td>Coal hopper ((100 , t))</td>
<td>4</td>
<td>6</td>
<td>40</td>
<td>1253</td>
<td>43</td>
<td>Normal</td>
</tr>
<tr>
<td>Four-axle intermodal</td>
<td>4</td>
<td>6</td>
<td>55</td>
<td>634</td>
<td>151</td>
<td>Normal</td>
</tr>
<tr>
<td>Auto-rack</td>
<td>4</td>
<td>6</td>
<td>55</td>
<td>813</td>
<td>70</td>
<td>Gamma</td>
</tr>
<tr>
<td>Four-axle mixed freight</td>
<td>4</td>
<td>6</td>
<td>30</td>
<td>1062</td>
<td>181</td>
<td>Normal</td>
</tr>
</tbody>
</table>

Note: \( P \) - Axle load, \( S_O \) - Outboard Axle Spacing, \( S_T \) - Truck Axle Spacing, \( S_I \) - Inboard Axle Spacing

Figure 4.31. Rail Equipment Configuration
4.5.5.2. Development of Fatigue Resistance Model

For resistance, the relevant S-N curve is randomized. The S-N curve is the standard accepted format for plotting test data and determining fatigue resistance. Eq. 4.3 through Eq. 4.5 describes a modified expression of basic AREA code equations for a fatigue resistance line with a fatigue shift line. In a reliability formulation of fatigue evaluation, it is common practice to assume a lognormal distribution along the regression line. Eq. 4.6 gives a lognormal distribution, which describes the fatigue resistance in the notation of this model.

\[ N_R = \alpha_c S^{-m_c} \]  
Eq. 4.3

when \( 0 \leq N_R \leq N_P \)

\[ N_R = \alpha_c S^{-m_c} \]  
Eq. 4.4

when \( N_P \leq N_R \leq \infty \)

\[ \alpha_p = N_P^{(1 - \frac{m_c}{m_p})} \alpha_c^{\frac{m_p}{m_c}} \]  
Eq. 4.5

where:

\( N_R \) = Fatigue resistance in number of cycles for stress range \( S \)

\( S \) = Fatigue resistance in stress range for number of cycles \( N_R \)

\( N_P \) = Number of cycles after which the fatigue shift resistance line governs

\( \alpha_c, \alpha_p, m_c, m_p \) are constants relevant to the fatigue detail in question.

\[ f_N(N) = \frac{1}{\sqrt{2\pi N \sigma_R} \ln(10)} \exp\left[ -\frac{\left[ \log_{10}(N) - \lambda_p \right]^2}{2\sigma_p^2} \right] \]  
Eq. 4.6

where:
\[ N = \text{Fatigue resistance} \]
\[ \lambda_R = \text{The expected value or mean of the log of } n \]
\[ \zeta_R = \text{The square root of the variance of the log of } n \]

In this study, AREA Category D was used to determine fatigue resistance. For this curve, the assumed fatigue limit was \( S_0 = 48.3 \text{ MPa (7 ksi) at } N_p = 6.414 \times 10^6 \), two slope \( m_c = 3 \) and \( m_p = 4 \), \( a_c = 2.2 \times 10^9 \). A value of \( \zeta_R = 0.21 \) was assumed in the probabilistic estimation of remaining fatigue life. In addition, a new fatigue resistance model was proposed as shown in Figure 4.33.

**Fatigue Critical Locations for Riveted Members**

From the previous studies, the following locations of riveted members are critical for fatigue check. **Location I, II, and VII were found in this study.**

I. Riveted patterns for built-up flexural members (Fisher, 1987)

II. Cover plate terminations (Fisher, 1987; Bruhwiler 1990)

III. Stringer to floor-beam connections (Fisher, 1987; Mang, 1993)

IV. Floor-beam end connections (Fisher, 1987; Mang, 1993)

V. Rivet patterns for truss members (Reemsnyder, 1975; Fisher, 1987)

VI. Gusset plate connections (Fisher, 1987)

VII. Hanger connections (Fisher, 1987)

VIII. Gross section where section loss over 20% due to corrosion (Out, 1984; Abe, 1989)
(a) Location I  (b) Location II  (c) Location III

Figure 4.32. Fatigue Critical Locations in This Study

Figure 4.33. Fatigue Resistance Model – Randomized S-N Curve

**Probabilistic Fatigue Evaluation Procedures**

A flowchart of the fatigue model for fatigue evaluation of the bridge is shown in Figure 4.34. The fatigue load spectra of this bridge are developed each year by converting the stress histories, which are simulated from the calibrated FE model, into stress ranges through the Rainflow counting calculation. The Monte Carlo simulation was then used to
generate a random stress range by multiplying the stress range with random dynamic impact. The sample size for one year was determined by the annual train frequencies. To account for the randomness in train frequency, 1000 samples were used in the simulation to develop the fatigue load spectra for one year. Two scenarios were proposed to investigate the effect of heavy railcars on the selected bridges. For Scenario 1, only passenger trains were considered, while Scenario 2 considered both freight trains and passenger trains. The annual fatigue load spectra for both scenarios are shown for Bridge A, Bridge B and Bridge C in Figure 4.35, Figure 4.36 and Figure 4.37, respectively.

Figure 4.34. Flowchart of Fatigue Model for Fatigue Evaluation of the Bridge
Annual fatigue load spectrum
(First cut-off)

Annual fatigue load spectrum
(Second cut-off)

(a) First cutoff

(b) Second cutoff
Figure 4.35. Annual Fatigue Load Spectrum (Bridge A)
Annual fatigue load spectrum
(First cut-off)

Scenario 1 (Passenger trains only)
Scenario 2 (Including freight trains)

Number of applied cycles
Stress range (MPa)

Fatigue limit

(a) First cutoff

Annual fatigue load spectrum
(Second cut-off)

Scenario 1 (Passenger trains only)
Scenario 2 (Including freight trains)

Number of applied cycles
Stress range (MPa)

Fatigue limit

(b) Second cutoff
Figure 4.36. Annual Fatigue Load Spectrum (Bridge B)
Figure 4.37. Annual Fatigue Load Spectrum (Bridge C)
Fatigue damage is calculated using Miner’s sum (Miner 1945) for the case of variable amplitude loading, which is expressed as Eq. 4.7 and Eq. 4.8.

\[ D = \sum_i \frac{n_i}{N_i} \quad \text{Eq. 4.7} \]

\[ D = \sum_{i=1}^{k_1} \frac{n_i}{N_i} + \sum_{i=1}^{k_2} \frac{n_i}{N_i} \quad \text{Eq. 4.8} \]

where \( D \) is the total damage,

\( n_i \) is the applied number of cycles at a stress range, \( S_i \),

\( N_i \) is the fatigue life corresponding to the same stress range,

\( k_1 \) and \( k_2 \) are the number of stress range blocks \( S_i \) in the fatigue load spectrum higher or lower than the assumed fatigue limit \( S_0 \), respectively, and

\( n_i \) is the number of cycles in each \( S_i \).

Fatigue failure is expected to occur when \( D \) is greater than an assumed damage limit \( \delta \). In a deterministic analysis, \( \delta \) is generally assumed to be a fixed value, while in the probabilistic approach, it is commonly associated with considerable uncertainty. A lognormal distribution is used to describe \( \delta \). In this study, it was predicted that half of the bridge’s useful life had already passed; therefore the mean of variable \( \delta \) is assumed as 0.5. In addition, the CoV of 0.3 is considered here (Wirsching 1995). The limit state function is given as
\[ g = \delta - \sum_{i=1}^{T} D \]  

Eq. 4.9

where \( T \) is the number of years for the estimation of failure probability.

According to the limit state function provided in Eq. 4.9, the probability of fatigue failure can be defined as the probability when \( g < 0 \). This probability is a function of time (years), which can be determined by using Monte Carlo simulation with \( 10^5 \) samples. The sample number was determined by observing the number of samples when the value of the probability converges.

**Determination of Fatigue-Critical Location**

Currently, the fatigue criteria according to AREMA specifications are mainly focused on midspan effect. However, fatigue analysis needs to consider all critical locations along the plate girders, as well as the secondary elements and connections. In this study, the authors focused on the various locations along the plate girders, since the plate girders in open-deck bridges are regarded as fracture-critical members.

The probabilities of fatigue failure, along with the time in years for various locations on the plate girder of the selected three bridges, are shown in Figure 4.38, Figure 4.39 and Figure 4.40. Figure 4.41 shows the fatigue life shortening at each location (midspan or cutoff points) on the girders in three bridges.

For Bridge A (Figure 4.38), it can be seen that before the introduction of heavy freight cars (Scenario 1), the midspan location is the most fatigue-critical location. After
the introduction of heavy freight cars (Scenario 2), the second cutoff and midspan location become the fatigue-critical locations. This might provide guidance for bridge inspection when checking the fatigue-critical locations after introducing heavy freight railcars. In Figure 4.41, if the target reliability indices are assigned to be 3.5 ($P_f=0.0002$), the reduction in the remaining fatigue life is 5 years for the midspan location, 11 years for the second cutoff location and 25 years for the first cutoff location. On the other hand, if the target reliability indices are assigned to be 2.5 ($P_f=0.006$), the reduction in the remaining fatigue life is 12 years for the midspan location, 20 years for the second cutoff location and 34 years for the first cutoff location.

For Bridge B, Figure 4.39 shows that before the introduction of heavy freight cars (Scenario 1), the second cutoff location is the most fatigue-critical location. After the introduction of heavy freight cars (Scenario 2), the first cutoff location becomes the fatigue-critical location. In Figure 4.41, if the target reliability indices are assigned to be 3.5 ($P_f=0.0002$), the reduction in the remaining fatigue life is 11 years for the midspan location, 10 years for the second cutoff location and 16 years for the first cutoff location, while if the target reliability indices are assigned to be 2.5 ($P_f=0.006$), the reduction in the remaining fatigue life is 18 years for the midspan location, 15 years for the second cutoff location and 24 years for the first cutoff location.

For Bridge C, Figure 4.40 shows that both before (Scenario 1) and after (Scenario 2) the introduction of heavy freight cars, the midspan location is the most fatigue-critical
location. In Figure 4.41, if the target reliability indices are assigned to be 3.5 ($P_f=0.0002$), the reduction in the remaining fatigue life is 6 years for the midspan location, and 14 years for the cutoff location. However, if the target reliability indices are assigned to be 2.5 ($P_f=0.006$), the reduction in the remaining fatigue life is 9 years for the midspan location, and 22 years for the first cutoff location.

Overall, the introduction of heavy freight railcars has a greater effect on the first cutoff location, and the reduction in fatigue life is dependent on the target criteria engineers choose (reliability index). On the other hand, if the estimations were reviewed within the context of the whole railway’s network, with the introduction of heavy freight trains, a 0.6% probability failure would imply that one failure out of 167 of the same type of structural member would occur in about 20 years. Considering the large inventory of similar railway bridge structural members, the heavy freight train is considered to have a considerable effect on the remaining life of the structure members. With this in mind, the probability of failure can help the state to allocate public funds properly.
Figure 4.38. Probability of Fatigue Failure for Bridge A

Figure 4.39. Probability of Fatigue Failure for Bridge B

Figure 4.40. Probability of Fatigue Failure for Bridge C
4.5.6. Parametric Study

4.5.6.1. The Effect of Annual Freight Train Frequency

Since the volume of freight railcars will affect the distribution of the fatigue loading spectra, this section will discuss the effect of annual freight train frequency. The values of annual freight train frequency are as follows: 3000 trains, 4000 trains, 5000 trains, 6000 trains, and 7000 trains.
trains, 6000 trains, and 7000 trains. The result is shown in Figure 4.42. If the target reliability indices are assigned to be 3.5 ($P_f=0.0002$), for freight trains the remaining fatigue life is 14 years for 3000 trains, 12 years for 4000 trains, 10 years for 5000 trains, 8 years for 6000 trains and 6 years for 7000 trains. It was found that an increase of 1000 freight trains in annual freight train frequency would have a shortening effect in fatigue life of exactly 2 years. This can help relevant agencies to schedule proper freight frequencies associated with life cycle cost evaluation.

![Figure 4.42. The Effect of Annual Freight Train Frequency (First Cutoff Location of Bridge B)](image)

4.5.6.2. The Effect of the Detail Categories and the Fatigue Shift Line

From the annual fatigue load spectrum, it was found that most of the stress ranges were below the fatigue limit; therefore, it was critical to select a practical value for the
gradient of the fatigue shift line, $m_p$. The effect of different fatigue detail categories chosen and value of $m_p$ were evaluated in this section as shown in Figure 4.43. From the analysis, Category D is shown to be more conservative than Category C, since the fatigue resistance in terms of stress cycles for Category C is higher. The differences in fatigue remaining life was 5 years and 8 years between Category C and D for reliability indices equal to 3.5 ($P_f=0.0002$) and 2.5 ($P_f=0.0062$), respectively. Additionally, no significant difference was found using a different value of $m_p$. This low sensitivity for the probability failure of the gradient of the fatigue shift line was because the stress range in this type of bridge is high. Therefore, the large stress range governed the analysis for similar types of bridges.

Figure 4.43. The Effect of Fatigue Detail Categories and the Fatigue Shift Line (First Cutoff Location of Bridge B)
4.5.6.3. The Effect of Span Length

In this section, fatigue life shortening was investigated along the span length. The fatigue life shortening value for each bridge at various reliability levels was chosen as the maximum value among the different locations of the bridge. The results are shown in Figure 4.44. The maximum fatigue life shortening years decrease as the span length increases. It was found that the introduction of heavier rail equipment will have a much more significant effect on shorter spans.

![Figure 4.44. The Effect of Span Length on Fatigue Life Shortening](image)

4.5.6.4. Discussion on Reliability Index

The evaluation criteria, which can be represented by the reliability index, β, denote safety and serviceability standards adopted for assessing existing bridges (AASHTO MBE 2011). The AASHTO Load and Resistance Factor Design (LRFD)
Specifications were calibrated based on a conservative target reliability index of 3.5, while the AASHTO Load and Resistance Factor Rating (LRFR) adopt a reduced-target reliability index of approximately 2.5, calibrated to the past AASHTO operating level load rating. The overly conservative standard in rating can cause an unnecessary increase in the cost of rehabilitation and replacement. To recognize the balance between safety and economics, the lower reliability target is recommended for evaluation and rating. In this study, the authors suggest a $\beta$ of 2.5 for the future research.

4.5.7. Conclusions

In this study, a probabilistic model was proposed for fatigue evaluation of railway bridges located on the NJ Transit line to account for uncertainties in both load and material variability. The following conclusions can be drawn from the previous analysis:

1. For Bridge A, it was shown that before the theoretical introduction of heavy freight cars (Scenario 1), the midspan location is the most fatigue-critical location. After the introduction of heavy freight cars (Scenario 2), the second cutoff and midspan location becomes the fatigue-critical location. This may provide guidance when inspecting fatigue-critical locations after introducing heavy freight railcars. If the target reliability indices are assigned to be 3.5 ($P_f=0.0002$), the reduction in the remaining fatigue life is 5 years for the midspan location, 11 years for the second cutoff location and 25 years for the first cutoff location. However, if the target reliability indices are assigned
to be 2.5 \((P_f=0.006)\), the reduction in the remaining fatigue life is 12 years for the midspan location, 20 years for the second cutoff location and 34 years for the first cutoff location.

2. For Bridge B, before the introduction of heavy freights (Scenario 1), the second cutoff location is the most fatigue-critical location. After the introduction of heavy freight cars (Scenario 2), the first cutoff location becomes the fatigue-critical location. If the target reliability indices are assigned to be 3.5 \((P_f=0.0002)\), the reduction in the remaining fatigue life is 11 years for the midspan location, 10 years for the second cutoff location and 16 years for the first cutoff location. However, if the target reliability indices are assigned to be 2.5 \((P_f=0.006)\), the reduction in the remaining fatigue life is 18 years for the midspan location, 15 years for the second cutoff location and 24 years for the first cutoff location.

3. For Bridge C, both before (Scenario 1) and after (Scenario 2) the introduction of heavy freight cars, the midspan location is the most fatigue-critical location. If the target reliability indices are assigned to be 3.5 \((P_f=0.0002)\), the reduction in the remaining fatigue life is 6 years for the midspan location, and 14 years for the cutoff location, while if the target reliability indices are assigned to be 2.5 \((P_f=0.006)\), the reduction in the remaining fatigue life is 9 years for the midspan location, and 22 years for the first cutoff location.

4. Heavy freight cars have a significant effect on critical locations near the support.
5. In fatigue analysis, the midspan location is not always the critical location. For example in Bridge B, the second cutoff location is the most critical location for fatigue before the heavy freight car is introduced (Scenario 1). After the introduction of the heavy freight car (Scenario 2), the first cutoff location becomes the most critical location since cycles of larger stress ranges were found.

6. An increase of 1000 freight trains in the annual freight train frequency will shorten the remaining fatigue life by approximately 2 years. This can help relevant agencies in scheduling proper freight frequencies associated with life cycle cost evaluation.

7. The difference in fatigue remaining life was 17 years between Category C and D ($P_f=0.0002$). For both categories, the change in the value for the slope of the fatigue shift line, $m_p$, does not affect fatigue life significantly in this study.

8. The maximum fatigue-life-shortening years decreases as the span length increases. It was found that the introduction of heavier rail equipment will have a much more significant effect on shorter spans.

9. Remaining fatigue life and the reduction in fatigue life after the introduction of heavier rail cars depends on the chosen reliability index.
CHAPTER 5
THE EFFECT OF OVERWEIGHT TRUCKS ON BRIDGE DETERIORATION

In 1993, Nowak et al. started the study on the effect of truck loads on bridges. Lately, various state agencies initiated studies on the impact of overweight trucks on infrastructure systems. Figure 5.1 shows the annual maximum GVW and annual average daily counts of overweight trucks over the last 20 years from NJ Site 287 as well as the deterioration process for a P/C bridge located on route I-287. It is clearly shown that trends of the annual maximum weight and daily frequency of overweight trucks are both increasing over a period of 20 years. Figure 5.1 (b) shows the deterioration process of the P/C girders. First, cracking and spalling appeared at beam-ends (e.g., high shear and bearing stresses) due to the high volume and heavy truck traffic. Then the rebars and tendons were exposed to the moisture and deicing salts drained from the top of the bridge through the deteriorated deck joints and were eventually corroded.
Annual Maximum GVW (kips)
Daily Counts of Overweight

Site 287 (I-287)

(a) Annual maximum GVW and daily counts of overweight trucks at Site 287

(b) Deterioration process of a P/C bridge

Figure 5.1. Overweight trucks and bridge deterioration on I-287

5.1. Deck Deterioration

Concrete bridge deck is a component that usually undergoes more deterioration than any other due to its direct exposure to heavy and more frequent truck traffic, environmental conditions, and de-icing salts. Therefore, the deterioration for bridge decks could be quite complicated. Previous studies (Azad et al. 1986, Okada et al. 1978, Kato and Goto 1978) have demonstrated that transverse cracks and water penetration during service decreased the ultimate punching shear and fatigue strengths of the concrete.
decks. However, the interaction between the deck deterioration and the overweight loading were not quantified explicitly. The current AASHTO MBE (AASHTO 2011) indicates that the de-icing salts and truck traffic volume affect the deck deterioration rate, but the current practical evaluation of bridge decks is limited to visual inspections only. In reality, a combination of mechanical loading and environmental factors would lead to the end of service life.

From the literature review, the deterioration of bridge deck usually comes from various factor. A reliable prediction of service life of deteriorating highway bridge deck under different loading condition is needed for a rational life-cycle cost analysis.

The deterioration mode of RC deck was a combination of mechanical loading and environmental effect as shown in Figure 5.2.

![Concrete Deck Deterioration Diagram](image)

Figure 5.2. Concrete Deck Deterioration
5.2. Service Life of Highway Bridge Deck in New Jersey

Condition rating data over time for three typical bridges are plotted as examples in Figure 5.3. For SN218162, the bridge had a major rehabilitation, deck replacement, in Year 1994 which brought the rating from 4 to 9. For SN1412171, the bridge deck experienced lots of minor rehabilitation in Year 1994, 2007, and 2011 which maintained the rating above or equal to 4. For SN2113154, there was no rehabilitation for this bridge, the curve represent a natural deterioration of the bridge deck. In reality, the bridge decks will more or less have different types of rehabilitation to assure its condition above certain serviceability requirement. However, the expected trend in the deterioration process is that if there is no improvement made to a bridge member, its condition rating either remains the same or falls to a lower value as the bridge ages. Although there is reconstruction information in the NBI database, there are still lots of unrecorded repair or reconstruction activities on the bridge members due to their sudden increase in condition rating (Saito and Sinha 1989, Bolukbasi et al. 2004, Hatami and Morcous 2011). Therefore, it is important to check the validity of condition rating by examining the records of each bridge.

A flowchart describing the proposed validation procedure is shown in Figure 5.4. The age of a bridge is captured when the rating is downgraded. The deterioration data indicated that outliers consist of the few data points with age less than 10 years and
condition ratings below 6 as well as data points with age 40 years or older and condition ratings above 7. In order to partially address this issue, Hatami and Morcous developed a criteria for the maximum and minimum age number for each condition rating as follows (Hatami and Morcous 2011):

1) Condition rating 9 age less than 0 and more than 30 years
2) Condition rating 8 age less than 0 and more than 40 years
3) Condition rating 7 age less than 0 and more than 50 years
4) Condition rating 6 age less than 10 and more than 60 years
5) Condition rating 5 age less than 20 and more than 70 years
6) Condition rating 4 age less than 30 and more than 80 years

Figure 5.3. Three typical curve from selected sites
After the rating data was validated, the rating data was grouped based on the bridge locations. Figure 5.5 shows the condition rating over deck age for I-80, I-280, US-206, and NJ-18 as examples. The data has a curved shape as a third order polynomial. Different types of regression analysis were performed to find the best fit of condition rating over age. Compared to other simple function such as linear, second order
polynomial, Exponential, logarithmic and power function, the third order polynomial function provided the best fit of the data as shown in Table 5.1. In addition, from previous literature (Saito and Sinha 1989, Bolukbasi et al. 2004, Hatami and Morcous 2011) also indicated that a third order polynomial function would represent the condition rating over time.

The deterioration model assumed here was a third order polynomial function of bridge age in years:

\[
CR = M_0 + M_1x + M_2x^2 + M_3x^3 \quad \text{Eq. 5.1}
\]

Where:

\( CR \) = the Bridge Condition Rating, \( x \) is the age of bridge in years

\( M_0, M_1, M_2, \) and \( M_3 \) = the parameters from regressions

Previous studies concluded that bridge decks usually experienced replacement when the condition rating downgraded to 4 (Saito and Sinha 1989, Bolukbasi et al. 2004, Hatami and Morcous 2011). Additionally, repair events which reflected by the upgrades in condition rating were summarized for bridge deck, bridge superstructure, and bridge substructure in Figure 5.6, Figure 5.7, and Figure 5.8, respectively. A total of 1022 decks upgrade events from 1914 bridges on three types of highways in New Jersey were summarized in the figure below. The time range is from 1992 to 2013. In reality, the repair behavior is complicated, especially for decks, since repair in superstructure and substructure
will lead to total replacement of deck. From the figure, deck upgrades were mostly taking place when the rating has reached level 4 (26.8%).

![Figure 5.5. Condition Rating over Time](image)

**Table 5.1 Comparison of Different Regression Models**

<table>
<thead>
<tr>
<th>Regression Models</th>
<th>I-80 data</th>
<th>US-206 data</th>
<th>NJ-18 data</th>
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<tbody>
<tr>
<td>Third order polynomial</td>
<td>$R^2$=0.83</td>
<td>$R^2$=91</td>
<td>$R^2$=76</td>
</tr>
<tr>
<td>Second order polynomial</td>
<td>$R^2$=0.82</td>
<td>$R^2$=0.90</td>
<td>$R^2$=0.74</td>
</tr>
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<td>linear</td>
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</tr>
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<td>Exponential</td>
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</tr>
<tr>
<td>logarithmic</td>
<td>$R^2$=0.66</td>
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<td>$R^2$=0.55</td>
</tr>
<tr>
<td>Power</td>
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<td>$R^2$=0.63</td>
<td>$R^2$=0.45</td>
</tr>
</tbody>
</table>
Figure 5.9 shows the deterioration curves of bridges deck on different highways. Therefore the service life of bridge decks on each highway is determined at the age of deck when the condition rating downgraded to 4. Note that the service life from Figure 5.9 represents the mean value of service life on each highway. The average service life of bridge decks on interstate highways, US numbered highways, and NJ state highways are 36.8, 48.4, and 52 years respectively. The bridge decks on interstate highway deteriorated with the highest rate while the decks on NJ State Highway deteriorated much more slowly. The deterioration curve of bridge deck for available highways are provided in Appendix.

![Frequency vs Deck Rating Graph](image)

**Figure 5.6. Summary of deck upgrade events**
Figure 5.7. Summary of superstructure upgrade events

Figure 5.8. Summary of substructure upgrade events
Table 5.2 Summary of Regression Models for Each Highway

<table>
<thead>
<tr>
<th>Highway</th>
<th>M₀</th>
<th>M₁</th>
<th>M₂</th>
<th>M₃</th>
<th>R²</th>
<th>Expected service life (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-80</td>
<td>8.457</td>
<td>-0.279010</td>
<td>0.013952</td>
<td>-0.000314</td>
<td>0.76</td>
<td>30.5</td>
</tr>
<tr>
<td>I-78</td>
<td>8.906</td>
<td>-0.370560</td>
<td>0.020138</td>
<td>-0.000455</td>
<td>0.68</td>
<td>29.0</td>
</tr>
<tr>
<td>I-676</td>
<td>8.689</td>
<td>-0.149160</td>
<td>0.003358</td>
<td>-0.000040</td>
<td>0.74</td>
<td>55.4</td>
</tr>
<tr>
<td>I-295</td>
<td>8.731</td>
<td>-0.281460</td>
<td>0.017453</td>
<td>-0.000386</td>
<td>0.67</td>
<td>34.4</td>
</tr>
<tr>
<td>I-195</td>
<td>9.257</td>
<td>-0.147780</td>
<td>0.004320</td>
<td>-0.000052</td>
<td>0.55</td>
<td>63.3</td>
</tr>
<tr>
<td>I-95</td>
<td>9.263</td>
<td>-0.309750</td>
<td>0.012759</td>
<td>-0.000196</td>
<td>0.76</td>
<td>42.9</td>
</tr>
<tr>
<td>I-287</td>
<td>8.285</td>
<td>-0.071826</td>
<td>-0.003415</td>
<td>0.000006</td>
<td>0.91</td>
<td>26.9</td>
</tr>
<tr>
<td>I-280</td>
<td>9.282</td>
<td>-0.171180</td>
<td>0.003212</td>
<td>-0.000054</td>
<td>0.87</td>
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</tr>
<tr>
<td>US-30</td>
<td>8.881</td>
<td>-0.493610</td>
<td>0.039417</td>
<td>-0.001203</td>
<td>0.84</td>
<td>22.5</td>
</tr>
<tr>
<td>US-46</td>
<td>8.749</td>
<td>-0.274130</td>
<td>0.008660</td>
<td>-0.000111</td>
<td>0.63</td>
<td>43.8</td>
</tr>
<tr>
<td>US-9</td>
<td>8.359</td>
<td>-0.182210</td>
<td>0.006012</td>
<td>-0.000086</td>
<td>0.72</td>
<td>48.0</td>
</tr>
<tr>
<td>US-40</td>
<td>8.575</td>
<td>-0.174210</td>
<td>0.003439</td>
<td>-0.000029</td>
<td>0.81</td>
<td>62.6</td>
</tr>
<tr>
<td>US-322</td>
<td>8.185</td>
<td>-0.115930</td>
<td>0.002360</td>
<td>-0.000031</td>
<td>0.70</td>
<td>53.1</td>
</tr>
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<td>US-202</td>
<td>8.394</td>
<td>-0.182660</td>
<td>0.004732</td>
<td>-0.000057</td>
<td>0.62</td>
<td>49.6</td>
</tr>
<tr>
<td>US-206</td>
<td>8.344</td>
<td>-0.111670</td>
<td>0.002289</td>
<td>-0.000037</td>
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<td>49.0</td>
</tr>
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<td>US-130</td>
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<td>0.002230</td>
<td>-0.000023</td>
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<td>US-1</td>
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<td>0.008185</td>
<td>-0.000114</td>
<td>0.75</td>
<td>38.7</td>
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<td>US-22</td>
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<td>0.007360</td>
<td>-0.000083</td>
<td>0.76</td>
<td>56.1</td>
</tr>
<tr>
<td>NJ-31</td>
<td>8.572</td>
<td>-0.210970</td>
<td>0.003373</td>
<td>-0.000020</td>
<td>0.86</td>
<td>46.3</td>
</tr>
<tr>
<td>NJ-15</td>
<td>8.837</td>
<td>-0.162300</td>
<td>0.002751</td>
<td>-0.000018</td>
<td>0.90</td>
<td>86.1</td>
</tr>
<tr>
<td>NJ-18</td>
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<td>0.002884</td>
<td>-0.000041</td>
<td>0.76</td>
<td>54.9</td>
</tr>
<tr>
<td>NJ-33</td>
<td>8.141</td>
<td>-0.039308</td>
<td>-0.000706</td>
<td>0.000007</td>
<td>0.77</td>
<td>76.6</td>
</tr>
<tr>
<td>NJ-55</td>
<td>8.032</td>
<td>-0.063772</td>
<td>0.000213</td>
<td>-0.000004</td>
<td>0.66</td>
<td>61.0</td>
</tr>
<tr>
<td>NJ-73</td>
<td>8.216</td>
<td>-0.103140</td>
<td>0.002510</td>
<td>-0.000031</td>
<td>0.82</td>
<td>62.4</td>
</tr>
</tbody>
</table>
Figure 5.9. Comparison of deterioration models for bridges on various highways
5.3. Deck Service Life Prediction Functions

5.3.1.1. Loading on Bridge Deck

The WIM data that used was collected from various WIM sites the operated by NJDOT. The raw data contains all traffic including cars and trucks and a significant amount of erroneous data. A refined data processing program was proposed as shown in Figure 5.10 to extracted two datasets: “all trucks” dataset, and “legal trucks” dataset. “All trucks” dataset reflects the actual truck loading on the bridges that can be used to correlate with the service life obtained from previous section and then service life prediction functions based on wheel weight could be obtained. “Legal trucks” dataset was utilized to predict the service life of deck under legal trucks traffic without overweight trucks. By comparing the service lives of deck under these two truck traffic conditions, the reduction in service life of bridge decks can be calculated. Figure 5.11 shows the comparisons of ADTT, APD, effective truck weight, equivalent wheel load, average APT, and proportion of overweight trucks over all trucks on the three types of highways. It is found that interstate highways have significantly higher ADTT and APD comparing to the other two highway types. The median and average ADTT are 3884 and 4293 for interstate highways, 437 and 616 for US numbered highways, and 405 and 635 for NJ state highways. Meanwhile the average axles per trucks on interstate highway were found
to be higher than the other two. In addition most overweight trucks on interstate highways are Class 9 trucks with five axles while those on US numbered highway and NJ state highway are Class 7 trucks that has four axles. It is important to note that on average NJ State highway has the highest proportion of overweight trucks. As shown in Figure 5.11 (e) and (f), although the interstate highway has higher effective truck weight than the other two, all three highway types have comparable equivalent wheel loads due to the fact that trucks on interstate highways usually have more axles. The highest equivalent wheel load of 26.1 kips was observed on US-322. Given the similar wheel load level, the reason that decks on interstate highways deteriorated with the highest rate is because of the enormous values of ADTT and APD.

![Flowchart for WIM data processing](image)

**Figure 5.10.** Flowchart for WIM data processing
Figure 5.11. Statistics of “all trucks” dataset from WIM data
5.3.1.2. Correlation between Truck Loading and Deck Service Life

ADTT and APD are considered to be important factors that affect the service life of bridge decks since they indicate the frequency of loading on bridge decks. Datasets that have the comparable wheel weight were extracted and plotted in Figure 5.12. Different types of highways are treated separately in order to exclude the effect of highway types. Each line in the plot shows that a higher value of APD corresponds to lower service life. Comparing two lines in the same highway, lower wheel load levels correspond to higher service life, when the number of axles per day is held constant. Both parameters play roles in determining the service life of decks. In order to consider both parameters, the capacity of bridge decks herein was defined as the lifetime axle count, NA, which represents the total number of axles passing the bridge over service life span as below:

\[ N_A = \sum_{i=0}^{y} APD_i \times 365 \times r \]  

Eq. 5.2

where:

- \( N_A \) is the lifetime axle count,
- \( APD_i \) is average axles per day at year \( i \),
- \( y \) is service life in years predicted, and
- \( r \) is annual truck traffic growth.
The lifetime axle count was plotted versus the equivalent wheel load in Figure 5.13, and linear regressions were performed for three highway types. Note each data point represents the mean value of lifetime axle counts on one highway that under same equivalent wheel load. The total numbers of bridges considered are 597, 220, and 250 for Interstate highway, US numbered highway, and NJ state highway respectively. For Interstate highways, US numbered highways, and NJ state highways, the R-squared for linear regression line are 0.83, 0.71, and 0.65 respectively. It is found that service life of decks on Interstate highways and NJ state highways are more sensitive than on US numbered highways. Based on the correlations established above, prediction functions are proposed for life axle count based on equivalent wheel load as below:

\[
\begin{align*}
\text{Interstate} & \quad N_A = 7.2969 \times 10^8 - 2.7825 \times 10^7 \times P \quad R^2 = 0.83 \\
\text{US Numbered} & \quad N_A = 2.079 \times 10^8 - 6.9013 \times 10^6 \times P \quad R^2 = 0.71 \\
\text{NJ State} & \quad N_A = 6.9273 \times 10^8 - 2.8852 \times 10^7 \times P \quad R^2 = 0.65
\end{align*}
\]

Eq. 5.3

where:

\( P \) is the equivalent wheel load.

If service life and APD are considered, the functions can be expressed as below. Note these functions are data driven models and works for interpretation. In addition, beyond certain point, the service year of deck is not governed by the wheel loads.

Predicted service life of bridge deck is visualized in Figure 5.14. The annual traffic increase and average APT are taken as 2.25 percent and 4.04, 1.5 percent and 3.53, and 1.5 percent and 3.34 for Interstate, US numbered, and NJ state highway respectively.
where

\[ d \] is the annual truck traffic increase.
Figure 5.12. Effect of axles per day on service life of bridge decks

Figure 5.13. Correlation between lifetime axle counts vs equivalent wheel load
Figure 5.14. Predicted service life of deck; (a) interstate highway, (b) US numbered highway, and (c) NJ state highway
5.4. Bridge Girder Deterioration

From the classification of highway bridge inventory in New Jersey as shown in Figure 5.15, three types of bridges were selected for analysis including 1) steel multi-beam bridges, 2) steel floor-beam bridges, 3) P/C multi-beam bridge. More details about the classification of highway bridges in New Jersey are presented in Appendix.

The actual deterioration modes were identified for both steel and P/C bridges. Fourteen bridge structures near available WIM sites were selected as case studies including seven P/C bridges and seven steel bridges. For the P/C bridges, all had the same deterioration mode located at the end-beam of P/C girders with two bridges showing cracking and spalling, while the other five structures have severe corrosion of the prestressing tendons and mild steel reinforcement. After reviewing the historical inspection reports of these structures, the deterioration process is a combination of mechanical loading and environmental attack which can be summarized in four stages as indicated in Figure 5.1 (b). The cracks (i.e., possibly shear and bearing cracks) appeared on the bottom flange and web at the beam-end of P/C I-girders. Due to the deteriorated deck joints, the water and de-icing salts drained from the top of the bridges increased the moisture ingress in a cracked section. The cracks were widened after freezing and thaw cycles which led to the spalling of concrete. Eventually, the prestressing tendons were exposed and corroded. The corrosion of tendons is critical since even minor corrosion
will lead to huge loss of the load carrying capacity. Previous work also indicated that the deterioration of P/C bridges induced by corrosion was near the beam-ends of P/C girders from field investigations (Whiting et al. 1999, Needham 2000). Using detailed finite element analysis, Koyuncu et al. (Koyuncu 2003) indicated that the cracks at the beam-ends of P/C I-girders were caused by high shear stress exceeding the shear capacity of concrete. For steel bridges, the main deterioration mode is corrosion. As paint peels, rusting leads to section loss that reduces the load carrying capacity of steel girders. The critical locations are identified at the abutment bearing (6 out of 7), bottom flange (3 out of 7), and web of stringers (3 out of 7). Figure 5.15 also shows the typical deterioration modes for both types of structure.
Figure 5.15. Selection of prototype bridges for analysis
5.5. Correlation between Bridge Girder Deterioration and Truck Loadings

Data from reported condition ratings were processed and validated using the flowchart shown in Figure 5.16 (a) in order to obtain the complete life cycle without the intervention of unrecorded rehabilitation events. Meanwhile, the WIM data was filtered and processed using the flowchart in Figure 5.16 (b) based on the procedures developed in NCHRP 12-83 (Wassef 2014). In this study, the superstructure condition rating over time is assumed to represent the deterioration of bridges. From previous research (Lou et al. 2016), a third order polynomial regression was the best fit for the deck condition rating data over time with the highest value of coefficient of determination, $R^2$, when compared to other types of regressions, and a similar approach was adopted for the superstructure condition rating data. The regression model representing the deterioration used in this study is shown in Eq. 5.5.

$$CR = M_0 + M_1 x + M_2 x^2 + M_3 x^3$$

Eq. 5.5

where:

$CR = $ condition rating,

$x = $ age of deck in years, and

$M_0, M_1, M_2,$ and $M_3 = $ regression parameters.
5.5.1. Girder Deterioration

Having validated the ratings and ages, the regression models of bridge girders on various highways are plotted in Figure 5.17 and the regression models and expected service life are summarized in Table 5.3. The method of least squares was used to estimate the regression parameters. Most of the regression models have $R^2$ of above 0.75. For further illustration, Figure 5.17 also shows the condition rating data plotted over age as well as the fitted curve for I-80, I-78, I-95, I-295, I-287, NJ-18, US-9, and US-206. The expected service life of bridges was assumed at the age when the condition rating was downgraded to five. The reasons for this assumption are as follows: 1) very few
ratings below five were found in the database, indicating that five is a minimum accepted rating for girders, and 2) most events occurred at a rating of five shown in the rehabilitation data.

Table 5.3 shows that on interstate highways the performance of P/C bridges with an average life of 50.8 years is better than that of steel girders with an average life of 45 years, with the exception of I-95. From the WIM data, the shortest expected service life of bridges on I-95 can be due to the heaviest loading on I-95. In addition, on interstate highways, the standard deviation of expected service life for P/C bridges (5.2 years) is higher than that of steel bridges (2.7 years). The expected service life of P/C girders ranges from 42 years to 56 years while the steel girders have a consistent expected life of approximately 45 years. This indicated that P/C bridges are more sensitive to the loading effects than steel bridges.

Figure 5.18 shows a comparison of P/C girder and steel girder. Being on the same highway, they were assumed to be subjected to similar loading. It should be noted that the deterioration trends for both types of structures are quite different due to the nature of the two materials. As shown in Figure 5.18, the rate of deterioration from a condition rating of seven to five for P/C bridges is extremely high in comparison to the rate of deterioration from a condition rating of nine to seven. However, for steel bridges, the rate of deterioration is more gradual over time. From the definition of the condition rating (Weseman 1995) for P/C bridges, a rating of seven is the age when minor signs of
deterioration such as cracking and small spalling are typically observed. This indicates that once cracking and spalling of concrete on P/C girders occurs, the deterioration process becomes accelerated. As a result, the condition of P/C bridges is hard to preserve even under proper maintenance. On the other hand, frequent inspection and proper maintenance of steel bridge (e.g., painting) would effectively preserve their condition.
(a) P/C multi-beam bridges
(b) Steel multi-beam bridges on interstate highways
Figure 5.17. Condition rating of superstructure over time for highways: (a) P/C multi-beam bridges, (b) steel multi-beam bridges on interstate highways, (c) steel multi-beam on non-interstate highways.
Table 5.3 Summary of Regression Models and Expected Service Life

<table>
<thead>
<tr>
<th>P/C Multi-Beam</th>
<th>$M_0$</th>
<th>$M_1$</th>
<th>$M_2$</th>
<th>$M_3$</th>
<th>$R^2$</th>
<th>Expected Service Life (years)</th>
<th>Number of Bridges</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-95</td>
<td>8.70</td>
<td>-0.16</td>
<td>4.69E-03</td>
<td>-7.20E-05</td>
<td>0.96</td>
<td>42</td>
<td>10</td>
</tr>
<tr>
<td>I-287</td>
<td>9.09</td>
<td>-0.20</td>
<td>5.00E-03</td>
<td>-5.09E-05</td>
<td>0.90</td>
<td>50</td>
<td>39</td>
</tr>
<tr>
<td>I-295</td>
<td>9.69</td>
<td>-0.32</td>
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<td>-1.27E-04</td>
<td>0.84</td>
<td>52</td>
<td>12</td>
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<tr>
<td>I-78</td>
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<tr>
<td>I-80</td>
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<td>-4.15E-05</td>
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<tr>
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<td>Steel-Multi-Beam</td>
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<td>$M_1$</td>
<td>$M_2$</td>
<td>$M_3$</td>
<td>$R^2$</td>
<td>Expected Service Life (years)</td>
<td>Number of Bridges</td>
</tr>
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<td>-9.94E-05</td>
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<td>-1.15E-05</td>
<td>0.80</td>
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<td>-1.24E-05</td>
<td>0.89</td>
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<td>10</td>
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<td>-1.58E-05</td>
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<td>1.27E-03</td>
<td>-7.73E-06</td>
<td>0.61</td>
<td>89</td>
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</table>
Figure 5.18. Comparison of steel and P/C girders for different highways.
5.5.2. Truck Weight on Highways

The Average Daily Truck Traffic (ADTT) and truck weight distribution vary from site to site. Figure 5.19 (a) and (b) show the daily truck and axle counts on various highways, respectively. The vertical and horizontal axes represent the weight and daily counts that are above each weight threshold, respectively. The intersections of each curve with the vertical axis can be interpreted as the daily maximum weight observed. Figure 5.19 (a) also shows the ADTT (GVW>12 kips) on various highways. Although the ADTT for route I-78, I-287, and I-295 is much higher than that of I-95, the truck counts above the legal limit of 80 kips on I-95 is the highest overall. From Figure 5.19 (b), I-95 also has the most axle counts that above the legal limit of 20 kips. These finding support the previous notion that P/C bridges on I-95 have the lowest expected service life.
Federal Bridge Formula

$\text{GVW (kips)}$

$\text{Daily Truck Counts (Log Scale)}$

(a) Daily truck counts
Figure 5.19. Results from WIM data on various highway: (a) daily truck counts, (b) daily axle counts.
5.5.3. Service Life Estimation Based on Truck Weight and ADTT

This study assumed that the majority of the bridges in NJ are subjected to similar environmental and de-icing salts conditions. Bridges located in seaside areas pose an exception and were not considered. Based on this assumption, the variation of expected bridge service life is correlated with the different loading conditions. Thus, knowing the expected service life and the site-specific truck loading associated with these bridges, Figure 5.20 and Figure 5.21 show the expected service life versus daily counts for each weight threshold for P/C and steel bridges, respectively. Figure 5.20 (a) and Figure 5.21 (a) illustrate the expected service life of P/C and steel girders, respectively, versus the truck counts that are above 12, 40, 80, 100, and 120 kips. It is noted that the horizontal axis is a log scale and since the daily counts are annual average counts, the counts could be less than one. Figure 5.20 (a) shows that the expected service life correlates strongly with truck counts above 80 kips which is exactly the legal limit. This indicates that it is mainly the overweight trucks that cause accumulated damage in P/C girders. In the absence of any cracks, trucks below the legal limit might not accelerate the deterioration of P/C bridges. Similarly, as shown in Figure 5.21 (b), the expected service life is highly correlated with the axle counts that are above 16 and 20 kips. Meanwhile, for steel bridges, there is no correlation between expected service life and daily heavy truck counts. This indicates that the service life of steel girders is not sensitive to heavy loads.
The discrepancy between P/C and steel bridges can be attributed to different deterioration mechanism of the two structures. Heavy service load causes severe shear cracks on the P/C girders, accelerating their deterioration process, while the deterioration of steel girders is mainly due to corrosion and environmental effects. Nonetheless, it is noted that the expected service life of steel bridges correlates well with truck counts above 40 kips and 12 kips which means that the ADTT is still an important factor that would affect the service life of steel girders. Based on Figure 5.20 (a), a simple estimation function for the service life of P/C multi-beam bridges is proposed in Eq. 5.6:

\[ y = 88.2 - 17.76 \times \log(ADTT \times p) \]  

Eq. 5.6

where:

- \( y \) is the estimated service life of P/C multi-beam girders,
- \( p \) is the percentage of overweight trucks

Figure 5.22 illustrates the estimation function for service life based on Eq. 5.6.

The distributions of percentage of overweight and ADTT of all available highways from WIM data are also shown in Figure 5.22. The ADTT on major highways in New Jersey ranges from 100 to 15000 and the percentage of overweight trucks ranges from 1.5 to 22 percent.
(a) Expected service life vs. daily truck counts

\[ y = -16.641x + 59.759 \quad R^2 = 0.6103 \]
\[ y = -16.655x + 69.764 \quad R^2 = 0.7638 \]
\[ y = -17.76x + 88.201 \quad R^2 = 0.9585 \]
\[ y = -14.691x + 99.199 \quad R^2 = 0.6495 \]
\[ y = -15.577x + 107.65 \quad R^2 = 0.5756 \]
Figure 5.20. Effect of weight on service life of P/C multi-beam girders: (a) expected service life vs. daily truck counts, (b) expected service life vs. daily axle counts.
Expected service life vs. daily truck counts

\[ y = -13.409x + 63.574 \quad R^2 = 0.2844 \]

\[ y = -15.784x + 74.779 \quad R^2 = 0.2958 \]

\[ y = -25.545x + 105.77 \quad R^2 = 0.5744 \]

\[ y = -29.81x + 142.78 \quad R^2 = 0.8098 \]

\[ y = -35.189x + 172.14 \quad R^2 = 0.7984 \]

(a) Expected service life vs. daily truck counts
Figure 5.21. Effect of weight on service life of steel multi-beam girders: (a) expected service life vs. daily truck counts, (b) expected service life vs. daily axle counts.
5.5.4. Effect of Deck Condition on Service Life of Girders

Figure 5.23 shows the expected deck life of reinforced concrete (R/C) versus the expected service life of P/C and steel bridges. Except for I-95, the expected service life of P/C bridges exhibited a high correlation with that of R/C decks, having a coefficient of determination of about 0.7, while there is not a comparable strong correlation between the expected service life of steel bridges and R/C decks. This phenomenon can also be attributed to the different deterioration mechanisms of these two structures as discussed.
earlier. The condition of the R/C deck will highly affect the preservation process of the P/C bridge’s condition. With the presence of cracks and spalling of concrete in the P/C beam-ends, water and deicing salt from a poor-condition R/C deck will accelerate the deterioration of P/C girders. For steel bridges, despite an overall trend that bridges with lower R/C deck life have lower girder life, the correlation is not as strong as that for P/C girders. This finding suggests that prompt maintenance of the R/C deck, especially at the expansion joint, would help preserve the condition of P/C girders.

![Graph showing the relationship between R/C Deck Life and Girder Life (P/C)](a) R/C Deck Life vs Girder Life (P/C)

\[ y = 1.1505x - 26.115 \]

\[ R^2 = 0.6999 \]
Figure 5.23. Correlation of R/C deck life and girder life on same highway.
CHAPTER 6

BRIDGE LIFE CYCLE COST ANALYSIS

Structural reliability analysis method is a rational, probability based approach to identify the safety or probability of failure of a structure at various limit states. The term limit state is used to define the boundary between “success” and “failure” for various aspects of structural performance.

6.1. Bridge Life Cycle Cost Scheme

The Bridge Life Cycle Coast Analysis (BLCCA) has been required by current regulatory. It is important for infrastructure investments including highway bridge program (Hawk 2003). In BLCCA, the costs are paid by either the “agency” or “user”. The agency costs are usually the direct expenditures of funds for planning, design, construction, operation, and maintenance of a bridge. “User” costs consider a broader category including vehicles, business and residents nearby which rely on the bridges for access or transportation. In this study, only agency cost was considered. There are several economic indicators related to agency cost such as Net Present Value (NPV), Equivalent Uniform Annual Costs (EUAC) and Salvage Value (or residual value). NPV converts all costs to a single base year costs. For assets having useful life remaining at the end of
analysis period, a salvage value should be estimated. After converting all agency costs to \( NPV \) or \( EUAC \), the costs of various investment options can be compared. The expressions for mentioned indicators were shown in Eq. 6.1, Eq. 6.2 and Eq. 6.3. The cash flow diagrams for BLCCA corresponding to the deterioration of decks and the whole bridge are shown in Figure 6.1 and Figure 6.2, respectively.

\[
NPV = \sum_{t=0}^{T} \frac{C_t}{(1+r)^t} \quad \text{Eq. 6.1}
\]

where:

- \( C_t \) is the cost occurring at year \( t \) due to rehabilitation activity,
- \( T \) is the lifetime of the project or analysis period (years),
- \( r \) is the discount rate.

\[
EUAC = NPV \left[ \frac{r(1+r)^T}{(1+r)^T - 1} \right] \quad \text{Eq. 6.2}
\]

where:

- \( EUAC \) is the Equivalent Uniform Annual Costs converting from \( NPV \).

\[
SV = C \left( 1 - \frac{L_A}{L_E} \right) \quad \text{Eq. 6.3}
\]

where:

- \( SV \) is the salvage value of rehabilitation activity,
- \( C \) is the cost due to rehabilitation activity,
- \( L_A \) is the analysis life of rehabilitation in years, and
- \( L_E \) is the expected life of the rehabilitation.
Figure 6.1. Deck deterioration and cash flow diagram for life cycle cost analysis

Figure 6.2. Bridge deterioration and cash flow diagram for life cycle cost analysis
6.2. Damage Cost for R/C Deck

After the prediction function was developed for the service lives of bridge decks, two scenarios were considered in this study to quantify the economic impact of overweight trucks: Case 1 all trucks which represents current truck traffic with all overweight trucks and Case 2 legal truck traffic only without overweight trucks. BLCCA was performed for both scenarios. The annual maintenance costs are assumed to be the same for both scenarios. An analysis period of 75 years was used. The deck replacement cost is 150 dollar per ft$^2$. Percentage increases in annual truck traffic are assumed as 2.25, 1.5, and 1.5 for interstate highways, US numbered highways, and NJ state highways, respectively. The discount rate is assumed as 3 percent. The analysis results of BLCCA were summarized in Table 6.1. Parameters needed from WIM data for two scenarios are also listed. EUAC is annual deck cost per deck area in ft$^2$ due to overweight trucks. The unit cost is the cost of unit weight of overweight trucks. It is in dollar per deck ft$^2$ per kip. Note that weight of overweight here is the marginal weight of overweight trucks above the legal weight. Boxplots of service life reduction in percent and unit costs for three types of highways are shown in Figure 6.3. The percentage reductions of service life in average are 29.6, 26.2, and 49.2 for interstate highways, US numbered highways, and NJ state highways, respectively. The overweight trucks induced more costs on decks of NJ state highways than the other two. This can be attributed to: 1) NJ state highways have
the highest proportion of overweight trucks compared to the other two, 2) overweight trucks introduced much heavier wheel loads than the legal trucks did, and 3) the average number of axles per truck on NJ state highways is relatively less.

Figure 6.3. Effect of overweight trucks on Bridge Decks
Table 6.1 Summary of bridge deck life cycle cost analysis

<table>
<thead>
<tr>
<th>Route (WIM sites)</th>
<th>Service Life (Case 1) (years)</th>
<th>Service Life (Case 2) (years)</th>
<th>EUAC (Case 1) ($)/year/ft$^2$</th>
<th>EUAC (Case 2) ($)/year/ft$^2$</th>
<th>Cost Difference ($)/year/ft$^2$</th>
<th>Unit Cost (Overweight Part) ($/kips/ft$^2$/year)</th>
<th>Unit Cost (Whole Truck) ($/kips/ft$^2$/year)</th>
<th>Life Reduction (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-195</td>
<td>30</td>
<td>59</td>
<td>7.71</td>
<td>5.53</td>
<td>2.18</td>
<td>1.07E-06</td>
<td>2.80E-07</td>
<td>49%</td>
</tr>
<tr>
<td>I-280</td>
<td>63</td>
<td>102</td>
<td>5.39</td>
<td>4.90</td>
<td>0.48</td>
<td>3.85E-06</td>
<td>6.97E-07</td>
<td>38%</td>
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<tr>
<td>I-287 (A87)</td>
<td>19</td>
<td>30</td>
<td>10.48</td>
<td>7.71</td>
<td>2.77</td>
<td>7.29E-06</td>
<td>6.97E-07</td>
<td>37%</td>
</tr>
<tr>
<td>I-287 (287)</td>
<td>19</td>
<td>23</td>
<td>10.48</td>
<td>9.16</td>
<td>1.32</td>
<td>2.60E-06</td>
<td>2.55E-07</td>
<td>17%</td>
</tr>
<tr>
<td>I-78 (78A)</td>
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<td>9</td>
<td>23.35</td>
<td>19.23</td>
<td>4.12</td>
<td>2.15E-06</td>
<td>1.27E-07</td>
<td>22%</td>
</tr>
<tr>
<td>I-78 (78D)</td>
<td>22</td>
<td>26</td>
<td>9.45</td>
<td>8.41</td>
<td>1.04</td>
<td>2.07E-06</td>
<td>1.90E-07</td>
<td>15%</td>
</tr>
<tr>
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<td>19</td>
<td>11.97</td>
<td>10.48</td>
<td>1.49</td>
<td>7.06E-07</td>
<td>9.48E-08</td>
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</tr>
<tr>
<td>I-80</td>
<td>18</td>
<td>47</td>
<td>10.93</td>
<td>6.09</td>
<td>4.84</td>
<td>1.08E-05</td>
<td>1.78E-06</td>
<td>62%</td>
</tr>
<tr>
<td>I-95</td>
<td>32</td>
<td>43</td>
<td>7.41</td>
<td>6.33</td>
<td>1.09</td>
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<td>8.53E-08</td>
<td>26%</td>
</tr>
<tr>
<td>I-676</td>
<td>66</td>
<td>74</td>
<td>5.29</td>
<td>5.07</td>
<td>0.22</td>
<td>2.10E-06</td>
<td>1.57E-07</td>
<td>11%</td>
</tr>
<tr>
<td>I-295</td>
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<td>28</td>
<td>9.77</td>
<td>8.05</td>
<td>1.73</td>
<td>1.66E-06</td>
<td>2.01E-07</td>
<td>25%</td>
</tr>
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<td>US-1</td>
<td>25</td>
<td>36</td>
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<td>3.15E-07</td>
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<td>4.36E-07</td>
<td>5.19E-08</td>
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</tr>
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<td>US-9</td>
<td>29</td>
<td>40</td>
<td>7.87</td>
<td>6.53</td>
<td>1.35</td>
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<td>7.71E-07</td>
<td>28%</td>
</tr>
<tr>
<td>US-22</td>
<td>68</td>
<td>85</td>
<td>5.23</td>
<td>4.99</td>
<td>0.25</td>
<td>1.04E-06</td>
<td>1.54E-07</td>
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<td>70</td>
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<td>5.18</td>
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<td>1.52E-06</td>
<td>2.34E-07</td>
<td>30%</td>
</tr>
<tr>
<td>US-46</td>
<td>80</td>
<td>105</td>
<td>5.02</td>
<td>4.89</td>
<td>0.12</td>
<td>1.31E-06</td>
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<tr>
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<td>84</td>
<td>96</td>
<td>4.99</td>
<td>4.93</td>
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<td>2.06E-06</td>
<td>3.33E-07</td>
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<tr>
<td>US-202</td>
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<td>50</td>
<td>6.39</td>
<td>5.93</td>
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<td>3.56E-06</td>
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<td>16%</td>
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<tr>
<td>US-206</td>
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<td>82</td>
<td>5.18</td>
<td>5.00</td>
<td>0.17</td>
<td>8.10E-07</td>
<td>1.40E-07</td>
<td>15%</td>
</tr>
<tr>
<td>Route (WIM sites)</td>
<td>Service Life (Case 1) (years)</td>
<td>Service Life (Case 2) (years)</td>
<td>EUAC (Case 1) ($/year/ft²)</td>
<td>EUAC (Case 2) ($/year/ft²)</td>
<td>Cost Difference ($/year/ft²)</td>
<td>Unit Cost (Overweight Part) ($/kips/ft²/year)</td>
<td>Unit Cost (Whole Truck) ($/kips/ft²/year)</td>
<td>Life Reduction (%)</td>
</tr>
<tr>
<td>------------------</td>
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<td>-----------------------------</td>
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<td>---------------------------------------------</td>
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<td>US-322</td>
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<td>90</td>
<td>6.03</td>
<td>4.96</td>
<td>1.07</td>
<td>8.83E-06</td>
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<td>47</td>
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<tr>
<td>NJ-15</td>
<td>52</td>
<td>78</td>
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<td>5.03</td>
<td>0.80</td>
<td>8.09E-07</td>
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<td>33</td>
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<tr>
<td>NJ-18 (18D)</td>
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<td>86</td>
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<td>2.70E-06</td>
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<td>NJ-18 (018)</td>
<td>99</td>
<td>139</td>
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<td>4.80</td>
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<td>5.76E-08</td>
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<td>NJ-31</td>
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<td>103</td>
<td>7.87</td>
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<td>2.97</td>
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<td>NJ-33</td>
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<td>119</td>
<td>6.20</td>
<td>4.85</td>
<td>1.36</td>
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<td>NJ-34</td>
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<td>4.74</td>
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<td>7.65E-07</td>
<td>1.32E-07</td>
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<td>NJ-55</td>
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<td>6.46</td>
<td>1.25</td>
<td>6.79E-07</td>
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<td>NJ-68</td>
<td>141</td>
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<td>0.07</td>
<td>4.06E-06</td>
<td>6.69E-07</td>
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<td>168</td>
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<td>4.75</td>
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<td>3.79E-07</td>
<td>6.14E-08</td>
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<td>NJ-73</td>
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<td>137</td>
<td>5.04</td>
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<td>167</td>
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<td>4.75</td>
<td>0.07</td>
<td>3.82E-07</td>
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<tr>
<td>NJ-138</td>
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<td>6.14</td>
<td>4.78</td>
<td>1.36</td>
<td>8.18E-06</td>
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<td>69</td>
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<td>3.72E-06</td>
<td>4.88E-07</td>
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</table>
6.3. Damage Cost for Steel Bridge Girder

A detailed structural analysis using Finite Element analysis was performed in Appendix. With the prototype bridges, the stress range was obtained under various loading condition from WIM data. After the structural analysis and fatigue life estimation, two scenarios were considered in this study to quantify the economic impact of overweight trucks: Case 1 all trucks which represents current truck traffic with all overweight trucks and Case 2 legal truck traffic only without overweight trucks. BLCCA was performed for both scenarios. The annual maintenance costs are assumed to be the same for both scenarios. An analysis period of 75 years was used. The bridge replacement cost is assumed as $330 per ft². Percentage increases in annual truck traffic are assumed as 2.25, 1.5, and 1.5 for interstate highways, US numbered highways, and NJ state highways, respectively. The discount rate is assumed as 3 percent. The analysis results of BLCCA of Prototype Bridge I and Prototype Bridge II were summarized in Table 6.2 and Table 6.3.
Table 6.2 Summary of bridge girder life cycle cost analysis for Prototype Bridge I

<table>
<thead>
<tr>
<th>Route (WIM sites)</th>
<th>Service Year (w)</th>
<th>Service Year (wo)</th>
<th>EUAC (w)</th>
<th>EUAC (wo)</th>
<th>Cost Difference</th>
<th>Overweight part</th>
<th>Permit truck</th>
<th>Life Reduction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(years)</td>
<td>(years)</td>
<td>($/year/ft²)</td>
<td>($/year/ft²)</td>
<td>($/year/ft²)</td>
<td>($/kips/ft²/year)</td>
<td>($/kips/ft²/year)</td>
<td>(%)</td>
</tr>
<tr>
<td>I-195</td>
<td>131</td>
<td>188</td>
<td>10.59</td>
<td>10.38</td>
<td>0.21</td>
<td>1.03E-07</td>
<td>2.68E-08</td>
<td>30%</td>
</tr>
<tr>
<td>I-280(280)</td>
<td>221</td>
<td>238</td>
<td>10.31</td>
<td>10.28</td>
<td>0.03</td>
<td>2.41E-07</td>
<td>4.37E-08</td>
<td>7%</td>
</tr>
<tr>
<td>I-287(287)</td>
<td>95</td>
<td>105</td>
<td>10.85</td>
<td>10.76</td>
<td>0.09</td>
<td>1.84E-07</td>
<td>1.80E-08</td>
<td>10%</td>
</tr>
<tr>
<td>I-287(A87)</td>
<td>99</td>
<td>103</td>
<td>10.82</td>
<td>10.78</td>
<td>0.04</td>
<td>1.02E-07</td>
<td>9.70E-09</td>
<td>4%</td>
</tr>
<tr>
<td>I-78(78A)</td>
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<td>60</td>
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<td>0.36</td>
<td>1.90E-07</td>
<td>1.12E-08</td>
<td>7%</td>
</tr>
<tr>
<td>I-78(78D)</td>
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<td>10.78</td>
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<td>102</td>
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<td>1.36E-08</td>
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<td>1.38E-07</td>
<td>2.01E-08</td>
<td>30%</td>
</tr>
<tr>
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<td>191</td>
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<td>10.38</td>
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<td>9.77E-09</td>
<td>9%</td>
</tr>
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<td>10.27</td>
<td>0.09</td>
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<td>10.24</td>
<td>0.04</td>
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<td>2.34E-08</td>
<td>11%</td>
</tr>
<tr>
<td>US-22(022)</td>
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<td>10.17</td>
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<td>358</td>
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<td>10.15</td>
<td>0.03</td>
<td>2.79E-07</td>
<td>4.18E-08</td>
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</tr>
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<td>Service Year(w)</td>
<td>Service Year(wo)</td>
<td>EUAC(w)</td>
<td>EUAC(wo)</td>
<td>Cost Difference</td>
<td>Overweight part</td>
<td>Permit truck</td>
<td>Life Reduction</td>
</tr>
<tr>
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<td>-----------------</td>
<td>-----------------</td>
<td>---------</td>
<td>---------</td>
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<td></td>
<td>(years)</td>
<td>(years)</td>
<td>($/year/ft²)</td>
<td>($/year/ft²)</td>
<td>($/year/ft²)</td>
<td>($/kips/ft²/year)</td>
<td>($/kips/ft²/year)</td>
<td>(%)</td>
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<tr>
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<td>28%</td>
</tr>
<tr>
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<td>343</td>
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<td>10.16</td>
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<td>10.18</td>
<td>0.06</td>
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<td>3.07E-08</td>
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<td>10.18</td>
<td>0.11</td>
<td>1.43E-07</td>
<td>3.17E-08</td>
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<td>10.19</td>
<td>0.04</td>
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</tr>
<tr>
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<td>10.14</td>
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<td>5.79E-08</td>
<td>9%</td>
</tr>
<tr>
<td>NJ-55(551)</td>
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<td>210</td>
<td>10.52</td>
<td>10.33</td>
<td>0.19</td>
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<td>1.29E-08</td>
<td>30%</td>
</tr>
<tr>
<td>NJ-68</td>
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<td>406</td>
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<td>10.12</td>
<td>0.01</td>
<td>4.81E-07</td>
<td>7.93E-08</td>
<td>3%</td>
</tr>
<tr>
<td>NJ-72</td>
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<td>10.15</td>
<td>0.05</td>
<td>2.28E-07</td>
<td>3.69E-08</td>
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<td>388</td>
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<td>10.13</td>
<td>0.04</td>
<td>2.40E-07</td>
<td>4.75E-08</td>
<td>16%</td>
</tr>
<tr>
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<td>379</td>
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<td>10.14</td>
<td>0.05</td>
<td>2.48E-07</td>
<td>3.98E-08</td>
<td>16%</td>
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<tr>
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<td>507</td>
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<td>10.08</td>
<td>0.01</td>
<td>1.15E-06</td>
<td>1.98E-07</td>
<td>5%</td>
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<td>393</td>
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<td>10.13</td>
<td>0.03</td>
<td>1.61E-07</td>
<td>4.29E-08</td>
<td>10%</td>
</tr>
<tr>
<td>NJ-168</td>
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<td>466</td>
<td>10.10</td>
<td>10.09</td>
<td>0.01</td>
<td>7.72E-07</td>
<td>1.01E-07</td>
<td>3%</td>
</tr>
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</table>
Table 6.3 Summary of bridge girder life cycle cost analysis for Prototype Bridge II

<table>
<thead>
<tr>
<th>Route (WIM site)</th>
<th>Service Life (Case 1)</th>
<th>Service Life (Case 2)</th>
<th>EUAC (Case 1)</th>
<th>EUAC (Case 2)</th>
<th>Cost Difference</th>
<th>Unit Cost (Overweight part)</th>
<th>Unit Cost (whole truck)</th>
<th>Life Reduction</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-195</td>
<td>93</td>
<td>147</td>
<td>10.88</td>
<td>10.52</td>
<td>0.36</td>
<td>1.77E-07</td>
<td>4.62E-08</td>
<td>37%</td>
</tr>
<tr>
<td>I-280(280)</td>
<td>179</td>
<td>197</td>
<td>10.41</td>
<td>10.36</td>
<td>0.04</td>
<td>3.56E-07</td>
<td>6.45E-08</td>
<td>9%</td>
</tr>
<tr>
<td>I-287(287)</td>
<td>61</td>
<td>70</td>
<td>12.01</td>
<td>11.40</td>
<td>0.61</td>
<td>1.20E-06</td>
<td>1.18E-07</td>
<td>13%</td>
</tr>
<tr>
<td>I-287(A87)</td>
<td>64</td>
<td>68</td>
<td>11.79</td>
<td>11.54</td>
<td>0.25</td>
<td>6.64E-07</td>
<td>6.34E-08</td>
<td>5%</td>
</tr>
<tr>
<td>I-78(78A)</td>
<td>31</td>
<td>34</td>
<td>16.74</td>
<td>15.80</td>
<td>0.94</td>
<td>4.89E-07</td>
<td>2.88E-08</td>
<td>9%</td>
</tr>
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<td>I-78(78D)</td>
<td>63</td>
<td>68</td>
<td>11.89</td>
<td>11.52</td>
<td>0.37</td>
<td>7.40E-07</td>
<td>6.77E-08</td>
<td>8%</td>
</tr>
<tr>
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<td>67</td>
<td>12.64</td>
<td>11.58</td>
<td>1.06</td>
<td>5.03E-07</td>
<td>6.76E-08</td>
<td>20%</td>
</tr>
<tr>
<td>I-80(80B)</td>
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<td>112</td>
<td>10.78</td>
<td>10.71</td>
<td>0.07</td>
<td>1.46E-07</td>
<td>2.39E-08</td>
<td>7%</td>
</tr>
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<td>I-95(95B)</td>
<td>71</td>
<td>113</td>
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<td>37%</td>
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<td>I-676(676)</td>
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<td>10.50</td>
<td>0.14</td>
<td>1.33E-06</td>
<td>9.96E-08</td>
<td>19%</td>
</tr>
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<td>I-295(295)</td>
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<td>11.46</td>
<td>0.55</td>
<td>5.31E-07</td>
<td>6.40E-08</td>
<td>11%</td>
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<td>137</td>
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<td>10.40</td>
<td>0.16</td>
<td>2.17E-07</td>
<td>3.00E-08</td>
<td>25%</td>
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<td>10.20</td>
<td>0.03</td>
<td>6.53E-07</td>
<td>7.76E-08</td>
<td>9%</td>
</tr>
<tr>
<td>US-9(09A)</td>
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<td>0.07</td>
<td>2.43E-07</td>
<td>3.87E-08</td>
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<td>274</td>
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<td>10.23</td>
<td>0.08</td>
<td>3.56E-07</td>
<td>5.25E-08</td>
<td>20%</td>
</tr>
<tr>
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<td>234</td>
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<td>10.29</td>
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<tr>
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<td>296</td>
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<td>0.04</td>
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<td>11%</td>
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<td>10.33</td>
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<td>8%</td>
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<td>10.24</td>
<td>0.06</td>
<td>2.83E-07</td>
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<td>15%</td>
</tr>
<tr>
<td>Route (WIM site)</td>
<td>Service Life (Case 1)</td>
<td>Service Life (Case 2)</td>
<td>EUAC (Case 1)</td>
<td>EUAC (Case 2)</td>
<td>Cost Difference</td>
<td>Unit Cost (Overweight part)</td>
<td>Unit Cost (whole truck)</td>
<td>Life Reduction</td>
</tr>
<tr>
<td>------------------</td>
<td>----------------------</td>
<td>----------------------</td>
<td>---------------</td>
<td>---------------</td>
<td>----------------</td>
<td>------------------------</td>
<td>------------------------</td>
<td>----------------</td>
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<tr>
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<tr>
<td>NJ-18(18D)</td>
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<td>281</td>
<td>10.24</td>
<td>10.22</td>
<td>0.02</td>
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<td>5%</td>
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<tr>
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<tr>
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<td>0.36</td>
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<td>0.07</td>
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<tr>
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<td>326</td>
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<td>5%</td>
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<tr>
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<td>1.03E-06</td>
<td>1.35E-07</td>
<td>4%</td>
</tr>
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</table>
6.4. Damage Cost for P/C Girder

Based on the service life prediction functions from Chapter 5, the summary of Bridge Life Cycle Cost Analysis was shown in Table 6.4.
Table 6.4 Summary of bridge girder life cycle cost analysis for Prototype Bridge III

<table>
<thead>
<tr>
<th>Route ID</th>
<th>Service Life (Case 1) (years)</th>
<th>Service Life (Case 2) (years)</th>
<th>EUAC (Case 1) ($/year/ft²)</th>
<th>EUAC (Case 2) ($/year/ft²)</th>
<th>Cost Difference ($/year/ft²)</th>
<th>Unit Cost (Overweight part) ($/kips/ft²/year)</th>
<th>Unit Cost (whole truck) ($/kips/ft²/year)</th>
<th>Life Reduction (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-195</td>
<td>130</td>
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<td>0.23</td>
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<td>33%</td>
</tr>
<tr>
<td>I-280(280)</td>
<td>223</td>
<td>243</td>
<td>10.31</td>
<td>10.27</td>
<td>0.03</td>
<td>2.68E-07</td>
<td>4.84E-08</td>
<td>8%</td>
</tr>
<tr>
<td>I-287(287)</td>
<td>95</td>
<td>107</td>
<td>10.85</td>
<td>10.75</td>
<td>0.11</td>
<td>2.08E-07</td>
<td>2.03E-08</td>
<td>11%</td>
</tr>
<tr>
<td>I-287(A87)</td>
<td>98</td>
<td>103</td>
<td>10.83</td>
<td>10.78</td>
<td>0.04</td>
<td>1.14E-07</td>
<td>1.09E-08</td>
<td>5%</td>
</tr>
<tr>
<td>I-78(78A)</td>
<td>55</td>
<td>59</td>
<td>12.56</td>
<td>12.16</td>
<td>0.40</td>
<td>2.06E-07</td>
<td>1.21E-08</td>
<td>7%</td>
</tr>
<tr>
<td>I-78(78D)</td>
<td>98</td>
<td>105</td>
<td>10.83</td>
<td>10.76</td>
<td>0.06</td>
<td>1.25E-07</td>
<td>1.14E-08</td>
<td>7%</td>
</tr>
<tr>
<td>I-78(78W)</td>
<td>86</td>
<td>103</td>
<td>10.96</td>
<td>10.78</td>
<td>0.18</td>
<td>8.58E-08</td>
<td>1.15E-08</td>
<td>17%</td>
</tr>
<tr>
<td>I-80(80B)</td>
<td>142</td>
<td>152</td>
<td>10.54</td>
<td>10.50</td>
<td>0.04</td>
<td>9.24E-08</td>
<td>1.52E-08</td>
<td>6%</td>
</tr>
<tr>
<td>I-95(95B)</td>
<td>105</td>
<td>158</td>
<td>10.77</td>
<td>10.48</td>
<td>0.29</td>
<td>1.56E-07</td>
<td>2.28E-08</td>
<td>33%</td>
</tr>
<tr>
<td>I-676(676)</td>
<td>165</td>
<td>199</td>
<td>10.45</td>
<td>10.36</td>
<td>0.09</td>
<td>8.92E-07</td>
<td>6.68E-08</td>
<td>17%</td>
</tr>
<tr>
<td>I-295(295)</td>
<td>96</td>
<td>106</td>
<td>10.85</td>
<td>10.75</td>
<td>0.09</td>
<td>8.95E-08</td>
<td>1.08E-08</td>
<td>10%</td>
</tr>
<tr>
<td>US-1(001)</td>
<td>191</td>
<td>246</td>
<td>10.38</td>
<td>10.27</td>
<td>0.11</td>
<td>1.41E-07</td>
<td>1.95E-08</td>
<td>22%</td>
</tr>
<tr>
<td>US-30(30M)</td>
<td>341</td>
<td>372</td>
<td>10.17</td>
<td>10.14</td>
<td>0.02</td>
<td>5.00E-07</td>
<td>5.95E-08</td>
<td>8%</td>
</tr>
<tr>
<td>US-9(09A)</td>
<td>243</td>
<td>276</td>
<td>10.27</td>
<td>10.23</td>
<td>0.05</td>
<td>1.63E-07</td>
<td>2.60E-08</td>
<td>12%</td>
</tr>
<tr>
<td>US-22(022)</td>
<td>279</td>
<td>344</td>
<td>10.23</td>
<td>10.16</td>
<td>0.06</td>
<td>2.60E-07</td>
<td>3.83E-08</td>
<td>19%</td>
</tr>
<tr>
<td>US-40(040)</td>
<td>224</td>
<td>299</td>
<td>10.31</td>
<td>10.20</td>
<td>0.10</td>
<td>1.93E-07</td>
<td>2.98E-08</td>
<td>25%</td>
</tr>
<tr>
<td>US-46(046)</td>
<td>328</td>
<td>366</td>
<td>10.18</td>
<td>10.15</td>
<td>0.03</td>
<td>3.11E-07</td>
<td>4.65E-08</td>
<td>11%</td>
</tr>
<tr>
<td>US-130(13A)</td>
<td>345</td>
<td>363</td>
<td>10.16</td>
<td>10.15</td>
<td>0.01</td>
<td>4.27E-07</td>
<td>6.90E-08</td>
<td>5%</td>
</tr>
<tr>
<td>US-202(02B)</td>
<td>257</td>
<td>278</td>
<td>10.25</td>
<td>10.23</td>
<td>0.03</td>
<td>1.99E-07</td>
<td>2.88E-08</td>
<td>7%</td>
</tr>
<tr>
<td>US-206(206)</td>
<td>287</td>
<td>334</td>
<td>10.22</td>
<td>10.17</td>
<td>0.04</td>
<td>2.06E-07</td>
<td>3.57E-08</td>
<td>14%</td>
</tr>
<tr>
<td>Route ID</td>
<td>Service Life (Case 1)</td>
<td>Service Life (Case 2)</td>
<td>EUAC (Case 1)</td>
<td>EUAC (Case 2)</td>
<td>Cost Difference</td>
<td>Unit Cost (Overweight part)</td>
<td>Unit Cost (whole truck)</td>
<td>Life Reduction</td>
</tr>
<tr>
<td>-----------</td>
<td>-----------------------</td>
<td>-----------------------</td>
<td>---------------</td>
<td>---------------</td>
<td>----------------</td>
<td>------------------------</td>
<td>------------------------</td>
<td>-----------------</td>
</tr>
<tr>
<td>US-322(322)</td>
<td>323</td>
<td>413</td>
<td>10.18</td>
<td>10.12</td>
<td>0.06</td>
<td>5.04E-07</td>
<td>9.66E-08</td>
<td>22%</td>
</tr>
<tr>
<td>NJ-15</td>
<td>211</td>
<td>306</td>
<td>10.33</td>
<td>10.20</td>
<td>0.13</td>
<td>1.36E-07</td>
<td>1.97E-08</td>
<td>31%</td>
</tr>
<tr>
<td>NJ-18(18D)</td>
<td>338</td>
<td>353</td>
<td>10.17</td>
<td>10.16</td>
<td>0.01</td>
<td>3.04E-07</td>
<td>4.16E-08</td>
<td>4%</td>
</tr>
<tr>
<td>NJ-18(018)</td>
<td>261</td>
<td>330</td>
<td>10.25</td>
<td>10.18</td>
<td>0.07</td>
<td>2.41E-07</td>
<td>3.49E-08</td>
<td>21%</td>
</tr>
<tr>
<td>NJ-31</td>
<td>226</td>
<td>334</td>
<td>10.30</td>
<td>10.17</td>
<td>0.13</td>
<td>1.68E-07</td>
<td>3.71E-08</td>
<td>32%</td>
</tr>
<tr>
<td>NJ-33</td>
<td>279</td>
<td>325</td>
<td>10.22</td>
<td>10.18</td>
<td>0.05</td>
<td>1.96E-07</td>
<td>3.13E-08</td>
<td>14%</td>
</tr>
<tr>
<td>NJ-34</td>
<td>349</td>
<td>389</td>
<td>10.16</td>
<td>10.13</td>
<td>0.03</td>
<td>3.72E-07</td>
<td>6.41E-08</td>
<td>10%</td>
</tr>
<tr>
<td>NJ-55(551)</td>
<td>141</td>
<td>213</td>
<td>10.55</td>
<td>10.33</td>
<td>0.22</td>
<td>1.19E-07</td>
<td>1.51E-08</td>
<td>34%</td>
</tr>
<tr>
<td>NJ-68</td>
<td>399</td>
<td>415</td>
<td>10.13</td>
<td>10.12</td>
<td>0.01</td>
<td>5.37E-07</td>
<td>8.85E-08</td>
<td>4%</td>
</tr>
<tr>
<td>NJ-72</td>
<td>308</td>
<td>376</td>
<td>10.19</td>
<td>10.14</td>
<td>0.05</td>
<td>2.56E-07</td>
<td>4.14E-08</td>
<td>18%</td>
</tr>
<tr>
<td>NJ-73</td>
<td>331</td>
<td>400</td>
<td>10.17</td>
<td>10.13</td>
<td>0.05</td>
<td>2.65E-07</td>
<td>5.23E-08</td>
<td>17%</td>
</tr>
<tr>
<td>NJ-94</td>
<td>319</td>
<td>389</td>
<td>10.18</td>
<td>10.13</td>
<td>0.05</td>
<td>2.76E-07</td>
<td>4.43E-08</td>
<td>18%</td>
</tr>
<tr>
<td>NJ-124</td>
<td>494</td>
<td>521</td>
<td>10.08</td>
<td>10.07</td>
<td>0.01</td>
<td>1.27E-06</td>
<td>2.19E-07</td>
<td>5%</td>
</tr>
<tr>
<td>NJ-138</td>
<td>358</td>
<td>405</td>
<td>10.15</td>
<td>10.12</td>
<td>0.03</td>
<td>1.76E-07</td>
<td>4.69E-08</td>
<td>12%</td>
</tr>
<tr>
<td>NJ-168</td>
<td>463</td>
<td>482</td>
<td>10.10</td>
<td>10.09</td>
<td>0.01</td>
<td>8.31E-07</td>
<td>1.09E-07</td>
<td>4%</td>
</tr>
</tbody>
</table>
CHAPTER 7

SUMMARY AND CONCLUSIONS

7.1. Summary

This dissertation presents a rational procedure to investigate the impact of truck loads on bridges in New Jersey through the utilization of bridge inspection reports, truck WIM data, and the NBI database. Actual bridge deterioration modes were identified from their respective inspection reports. Based on the deterioration modes, the expected service life for each bridge component on various highways were estimated based on the condition rating data from NBI. In addition, weigh-in-motion data from stations in New Jersey were used to extract two datasets: “all trucks” and “legal trucks”. For bridge deck, the “all trucks” dataset was used to develop the deterioration models that could be used to estimate the expected service life while “legal trucks” dataset was used to predict the deck service life under loading without overweight trucks. For prestressed concrete (P/C) bridge girder, the correlation between the service life and daily overweight truck counts was performed and service life prediction function was obtained. For steel bridge girder, steel fatigue was assumed in this study as the deterioration model. Service life of steel girders was calculated with the fatigue assessment. Lastly, Bridge Life Cycle Cost Analysis (BLCCA) was conducted using two contrasting scenarios: one with and the other without overweight trucks to quantify their economic impact on bridge decks.
7.2. Proposed Service Life Prediction Functions for Bridges in New Jersey

As shown in previous Chapters, service life prediction function for different bridge components and types were proposed, including R/C decks, P/C multi-beam girders. The service life of steel girders was assumed as the fatigue life of bridge girders.

7.2.1. R/C Decks

The R/C deck service life prediction functions for different type of highways based on equivalent wheel load is shown below:

- Interstate \( N_A = 7.2969 \times 10^8 - 2.7825 \times 10^7 \times P \) \( R^2=0.83 \)
- US Numbered \( N_A = 2.079 \times 10^8 - 6.9013 \times 10^6 \times P \) \( R^2=0.71 \)
- NJ State \( N_A = 6.9273 \times 10^8 - 2.8852 \times 10^7 \times P \) \( R^2=0.65 \)

\[
y = \frac{\log(1 + \frac{N_A \times d}{365 \times ADTT \times APT})}{\log(1 + d)}
\]

7.2.2. P/C Multi-Beam Girders

The P/C girder service life prediction function based on truck loading is shown below.

\[
y = 88.2 - 17.76 \times \log(ADTT \times p)
\]

7.3. Conclusions

The following conclusions are based on the findings of this work:

(1) The main deterioration mode of P/C girders is the corrosion at beam-ends due to both truck loading (i.e., causing shear cracks) and environmental
attacks. This process is highly affected by the condition of the R/C deck while the deterioration mode of steel girders is mainly due to corrosion (i.e., leaking joints) at the bridge bearing, bottom flange, and web of stringers. The main deterioration of steel girders is the corrosion of steels.

(2) The bridge decks and girders on interstate highways deteriorated the most while those on NJ state highways deteriorated the least.

(3) Compared to US numbered and state highways, interstate highways have highest ADTT and axles per day. This is the reason that decks on interstate highways have shorter service lives. Although interstate highways have higher truck weight than the other two highway types, the equivalent wheel loads are comparable for all three highway types since the trucks on interstate highways usually have more axles to help distribute the GVW.

(4) State highways have the highest proportion of overweight trucks. Most overweight trucks on interstate highways are Class 9 with five axles. For the other two, the majority of overweight trucks are Class 7 with four axles.

(5) Service life prediction functions for decks were proposed based on equivalent wheel loads, helping to quantify service life reduction due to increased wheel load.

(6) Service life estimation functions were proposed based on the daily truck counts with GVW higher than 80 kips.
(7) Overweight trucks caused more marginal damage on NJ state highways due to the presence of a larger proportion of overweight trucks, larger wheel loads from overweight trucks, and fewer axles per truck.

(8) Overall, P/C girders have a slightly better performance than steel girders especially on interstate highways. During the lifetime of the bridge, the deterioration of P/C girders would be accelerated once cracking is initiated.

(9) The expected service life of P/C girders has relatively higher variation than that of steel girders and is highly sensitive to heavy truck loads while steel girders did not exhibit such sensitivity.

(10) Due to the deterioration mechanism, the condition of P/C bridges is hard to preserve once cracks and spalling of concrete are present. On the other hand, frequent inspection and proper periodical maintenance, such as painting, would effectively extend the service lives of steel girder bridges.

(11) Most of damage cost from overweight trucks was attributed to the deterioration of bridge decks. Deck unit costs are about 6-7 times of girder unit costs.
REFERENCES


Bae, H. and M. Oliva. Bridge Analysis and Evaluation of Effects under Overload Vehicles-Phase I. CFIRE 02-03, 2009


APPENDIX A

BRIDGE INVENTORY IN NEW JERSEY

The NBI Database has the most extensive and detailed data on highway bridges in the US. It is a collection of information (database) covering all of the nation's bridges located on public roads, including interstate highways, US numbered highways, state and county roads, as well as publicly-accessible bridges on federal lands. It presents a state by state summary analysis of the number, location, and general condition of highway bridges within each State. In this dissertation, the NBI data for all New Jersey bridges were collected from Year 1992 to Year 2013. The no-delimiter data were columned by the research group. The “Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation’s Bridges” provides instructions for coding of condition rating for bridge structure (Weseman 1995). There are up to 116 data items for bridges which categorized in three main data groups: Bridge Management Items (BRI_MGT_ITEM), Bridge Inventory Items (BRI_INV_ITEM), and Bridge Rating Items (BRI_RAT_ITEM). Each item has specified number which has a specified definition in bridge inspection manual. Based on the needed inputs for bridge deterioration models and bridge cost analysis, a list of items selected in this study was shown in Table A.1.

Before processing the NBI data, following preliminary data filtering steps has been done in order to define and select the highway bridge database. After the
preliminary filter shown above, 6918 bridges were selected and stored in the unified database.

1. Items 5a=1 (Route carried "on" the structure);
2. Items 42a=1 or 4 or 5 or 6 or 7 or 8 (service on the bridge: 1 - Highway, 4 - Highway-railroad, 5 - Highway-pedestrian, 6 - Overpass structure at an interchange or second level of a multilevel interchange, 7 - Third level (Interchange), 8 - Fourth level (Interchange));
3. Item 49 >=6.1meters (Bridge length >= 6.1 meter);
4. Item 112=Y (Structure meet or exceed the minimum length specified to be designated as a bridge for National Bridge Inspection Standards purposes).
5. Remove not applicable and blank data
6. Remove duplicate data.

Table A.1 List of items selected in this study

<table>
<thead>
<tr>
<th>#</th>
<th>Data Item</th>
<th>Item # in NBI</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Structure Number</td>
<td>item 8</td>
</tr>
<tr>
<td>2</td>
<td>Route Signing Prefix</td>
<td>item 5B</td>
</tr>
<tr>
<td>3</td>
<td>Designated Level of Service</td>
<td>item 5C</td>
</tr>
<tr>
<td>4</td>
<td>Route Number</td>
<td>item 5D</td>
</tr>
<tr>
<td>5</td>
<td>Directional Suffix</td>
<td>item 5E</td>
</tr>
<tr>
<td>6</td>
<td>Functional classification of inventory route</td>
<td>item 26</td>
</tr>
<tr>
<td>7</td>
<td>Year Built</td>
<td>item 27</td>
</tr>
<tr>
<td>8</td>
<td>Design Load</td>
<td>item 31</td>
</tr>
<tr>
<td>9</td>
<td>Structure type</td>
<td>item 43a and 43b</td>
</tr>
<tr>
<td>10</td>
<td>Number of spans in Main Unit</td>
<td>item 45</td>
</tr>
<tr>
<td>11</td>
<td>Structure length</td>
<td>item 49</td>
</tr>
<tr>
<td>12</td>
<td>Deck Width, out-to-out</td>
<td>item 52</td>
</tr>
<tr>
<td>13</td>
<td>Year Reconstructed</td>
<td>item 106</td>
</tr>
<tr>
<td>14</td>
<td>Deck Structure Type</td>
<td>item 107</td>
</tr>
</tbody>
</table>
Deterioration curves were prepared for superstructure and deck. The deterioration level is quantified using condition rating indices, which were also used by NBI. This is a numeric ranking system from “0” to “9”, where “0” represents “Failed Condition” and “9” represents “Excellent Condition” (Table A.2). Such condition rating data is available for deck, superstructure and substructure in the NBI database. NJDOT uses the same condition rating system as well.

Table A.2 NBI bridge condition ratings explanation

<table>
<thead>
<tr>
<th>Condition Rating</th>
<th>Interpretations</th>
</tr>
</thead>
<tbody>
<tr>
<td>9</td>
<td>Excellent Condition</td>
</tr>
<tr>
<td>8</td>
<td>Very Good Condition – no problems noted.</td>
</tr>
<tr>
<td>7</td>
<td>Good Condition – some minor problems.</td>
</tr>
<tr>
<td>6</td>
<td>Satisfactory Condition – structural elements show some minor deterioration.</td>
</tr>
<tr>
<td>5</td>
<td>Fair Condition – all primary structural elements are sound but may have minor section loss, cracking, spalling or scour.</td>
</tr>
<tr>
<td>4</td>
<td>Poor Condition – advanced section loss, deterioration, spalling or scour.</td>
</tr>
<tr>
<td>3</td>
<td>Serious Condition – loss of section, deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present.</td>
</tr>
<tr>
<td>2</td>
<td>Critical Condition – advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may</td>
</tr>
</tbody>
</table>
be present or scour may have removed substructure support. Unless closely monitored it may be necessary to close the bridge until corrective action is taken.

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Imminent Failure Condition – major deterioration or section loss present in critical structural components or obvious vertical or horizontal movement affecting structure stability. Bridge is closed to traffic but corrective action may put it back in light service.</td>
</tr>
<tr>
<td>0</td>
<td>Failed Condition – out of service; beyond corrective action.</td>
</tr>
</tbody>
</table>

### A.1 Bridge Classification

Deterioration of bridge elements depend on several important parameters related to bridge design, material, geographical location and environment, and traffic volume and weight. Therefore, it is important to classify bridges based on the values of these parameters so that targeted bridge type can be identified for developing deterioration models in the following bridge analysis part. To achieve this goal, filtered data records are classified based on the following parameters that are discussed in more detail in the following subsections:

- Route Signing Prefix
- Material types
- Structure types
- Bridge ages
- Bridge span length

Route Signing Prefix identified the route signing prefix for the inventory route including interstate highway, U.S. numbered highway, state highway, county highway,
city street, federal lands road, state lands road, and other. The distribution is shown in Figure A.1.

**Figure A.1. Distribution of Route Signing Prefix in bridge inventory**

There are different types of materials used in bridge superstructure. Material type is presented in item ITEM43A in NBI database using a number from 0 to 9. Figure A.2 shows the percentages of using steel, steel continued, reinforced concrete, reinforced concrete continued, prestressed concrete, prestressed concrete continued, and others in bridge superstructure. This figure clearly indicates that most of bridges are simply supported in each types of material. Nearly half of bridges are simply supported steel bridges and the second is simply supported prestressed concrete bridges.
Figure A.2. Distribution of bridge material in bridge inventory

Type of structure represents the structural system of the bridge and is presented in item ITEM43B. Type of structures has numbers from 00 to 22 as described in Table A.3. The distribution of structural type for each material type are shown from Figure A.3 to Figure A.7. For simply supported concrete bridge, most of them are culvert type and while for continuous concrete bridge are slab bridges. For both simply supported and continuous steel bridges, most of them are multi-beam bridge type and girder-floorbeam system. For prestressed concrete bridge, the highest proportion is multi-beam structure.
Table A.3 Bridge material code definition in NBI database

<table>
<thead>
<tr>
<th>Code</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Slab</td>
</tr>
<tr>
<td>2</td>
<td>Stringer/Multi-beam or Girder</td>
</tr>
<tr>
<td>3</td>
<td>Girder and Floorbeam System</td>
</tr>
<tr>
<td>4</td>
<td>Tee Beam</td>
</tr>
<tr>
<td>5</td>
<td>Box Beam or Girders - Multiple</td>
</tr>
<tr>
<td>6</td>
<td>Box Beam or Girders - Single or Spread</td>
</tr>
<tr>
<td>7</td>
<td>Frame (except frame culverts)</td>
</tr>
<tr>
<td>8</td>
<td>Orthotropic</td>
</tr>
<tr>
<td>9</td>
<td>Truss - Deck</td>
</tr>
<tr>
<td>10</td>
<td>Truss - Thru</td>
</tr>
<tr>
<td>11</td>
<td>Arch - Deck</td>
</tr>
<tr>
<td>12</td>
<td>Arch - Thru</td>
</tr>
<tr>
<td>13</td>
<td>Suspension</td>
</tr>
<tr>
<td>14</td>
<td>Stayed Girder</td>
</tr>
<tr>
<td>15</td>
<td>Movable - Lift</td>
</tr>
<tr>
<td>16</td>
<td>Movable - Bascule</td>
</tr>
<tr>
<td>17</td>
<td>Movable - Swing</td>
</tr>
<tr>
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</tr>
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</tr>
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<td>Channel Beam</td>
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**Figure A.3.** Distribution of structural types in simply supported concrete bridges
Figure A.4. Distribution of structural types in continuous concrete bridges

Figure A.5. Distribution of structural types in simply supported steel bridges
Figure A.6. Distribution of structural types in continuous steel bridges

Figure A.7. Distribution of structural types in simply supported prestressed concrete bridges
Figure A.8. Distribution of structural types in continuous prestressed concrete bridges
APPENDIX B

NBI DATA PROCESSING AND RESULTS

The validated condition rating data from NBI was shown in this appendix. Section B.1 presents the R/C deck condition rating over time. B.2 shows the P/C multi-beam condition rating over time. B.3 shows P/C box-beam condition rating over time. B.4 presents the steel multi-beam condition rating over time.

B.1 R/C Deck Condition Rating over Time

![Graph showing the deterioration of concrete deck on I-78. The equation is y = -0.0005x^3 + 0.0201x^2 - 0.3706x + 8.9059 with R² = 0.6773.]

Figure B.1. Deterioration of Concrete Deck on I-78
Figure B.2. Deterioration of Concrete Deck on I-80

Figure B.3. Deterioration of Concrete Deck on I-676
Figure B.4. Deterioration of Concrete Deck on I-295

Figure B.5. Deterioration of Concrete Deck on I-195
Figure B.6. Deterioration of Concrete Deck on US-30

\[ y = -0.0012x^3 + 0.0394x^2 - 0.4936x + 8.8806 \]
\[ R^2 = 0.8399 \]

Figure B.7. Deterioration of Concrete Deck on US-46

\[ y = -0.0001x^3 + 0.0087x^2 - 0.2741x + 8.7487 \]
\[ R^2 = 0.6296 \]
Figure B.8. Deterioration of Concrete Deck on US-9

Figure B.9. Deterioration of Concrete Deck on US-40
Figure B.10. Deterioration of Concrete Deck on US-322

Figure B.11. Deterioration of Concrete Deck on US-202
Figure B.12. Deterioration of Concrete Deck on US-130

\[ y = -2E-05x^3 + 0.0022x^2 - 0.125x + 8.146 \]
\[ R^2 = 0.6763 \]

Figure B.13. Deterioration of Concrete Deck on US-1
Figure B.14. Deterioration of Concrete Deck on US-22
B.2 P/C Multi-Beam Condition Rating over Time

Figure B.15. Deterioration of P/C Multi-Beam Superstructure on I-80

Figure B.16. Deterioration of P/C Multi-Beam Superstructure on I-78
Figure B.17. Deterioration of P/C Multi-Beam Superstructure on I-295

Figure B.18. Deterioration of P/C Multi-Beam Superstructure on I-95
Figure B.19. Deterioration of P/C Multi-Beam Superstructure on I-287

Figure B.20. Deterioration of P/C Multi-Beam Superstructure on US-202
Figure B.21. Deterioration of P/C Multi-Beam Superstructure on US-202

\[ y = -1E-05x^3 + 0.0013x^2 - 0.0877x + 8.6648 \]

\[ R^2 = 0.8748 \]
B.3 P/C Box-Beam Condition Rating over Time

Figure B.22. Deterioration of P/C Box-Beam Superstructure on US-46

Figure B.23. Deterioration of P/C Box-Beam Superstructure on US-9
Figure B.24. Deterioration of P/C Box-Beam Superstructure on US-40

Figure B.25. Deterioration of P/C Box-Beam Superstructure on US-322
Figure B.26. Deterioration of P/C Box-Beam Superstructure on US-206

\[ y = -1E-05x^3 + 0.0022x^2 - 0.1486x + 9.136 \]

\[ R^2 = 0.9427 \]
B.4 Steel Multi-Beam Condition Rating over Time

Figure B.27. Deterioration of Steel Multi-Beam Superstructure on I-80

Figure B.28. Deterioration of Steel Multi-Beam Superstructure on I-78
Figure B.29. Deterioration of Steel Multi-Beam Superstructure on I-295

Figure B.30. Deterioration of Steel Multi-Beam Superstructure on I-280
Figure B.31. Deterioration of Steel Multi-Beam Superstructure on I-195

Figure B.32. Deterioration of Steel Multi-Beam Superstructure on I-95
Figure B.33. Deterioration of Steel Multi-Beam Superstructure on I-287

Figure B.34. Deterioration of Steel Multi-Beam Superstructure on US-46
Figure B.35. Deterioration of Steel Multi-Beam Superstructure on US-9

Figure B.36. Deterioration of Steel Multi-Beam Superstructure on US-202
Figure B.37. Deterioration of Steel Multi-Beam Superstructure on US-206

Figure B.38. Deterioration of Steel Multi-Beam Superstructure on US-130
Figure B.39. Deterioration of Steel Multi-Beam Superstructure on NJ-31

Figure B.40. Deterioration of Steel Multi-Beam Superstructure on NJ-18
APPENDIX C

WIM DATA PROCESSING AND RESULTS

This appendix contains analysis results for WIM data throughout New Jersey. The map of WIM sites in New Jersey is shown in Figure C.1. The information of WIM sites is summarized in Table C.1. The distributions of GVW and axle weight on various highways are shown in Figure C.2 through Figure C.7.
Figure C.1. Map of WIM sites in New Jersey (NJDOT 2012).
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Figure C.2. Distribution of GVW on interstate highway
Figure C.3. Distribution of axle weight on interstate highway
Figure C.4. Distribution of GVW on US numbered highway
Figure C.5. Distribution of axle weight on US numbered highway
Figure C.6. Distribution of GVW on NJ state highway
Figure C.7. Distribution of axle weight on NJ state highway