NCHRP

REPORT 495

NATIONAL
COOPERATIVE
HIGHWAY
RESEARCH
PROGRAM

Effect of Truck Weight on Bridge Network Costs

TRANSPORTATION RESEARCH BOARD
OF THE NATIONAL ACADEMIES

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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Effect of Truck Weight on Bridge Network Costs

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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FOREWORD

By David B. Beal
Staff Officer
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This report contains the findings of a study to develop a methodology for estimating the impact of changes in truck weight limits on bridge network costs. The report describes the research effort and the recommended methodology and illustrates application of the methodology. A software module for automation of the recommended methodology also is included. The material in this report will be of immediate interest to bridge engineers and planners.

TRB Special Reports 225 and *227*, *Truck Weight Limits: Issues and Options* and *New Trucks for Greater Productivity and Less Road Wear: An Evaluation of the Turner Proposal*, respectively, noted that trucks produce significant damage to highway bridges. A truck's gross weight, axle weights, and axle configuration directly affect the useful life of highway bridge superstructures. Damage typically occurs in the bridge deck and in the superstructure elements including floor beams and girders, diaphragms, joints, and bearings. Bridge costs associated with increased truck weights are the result of the accelerated maintenance, rehabilitation, or replacement work that is required to keep structures at an acceptable level of service. Owners need a network-level methodology for estimating these costs.

Truck-weight frequency distributions by vehicle type (truck-weight histograms) are needed to estimate reliably the effects on remaining life and the costs caused by changes in legal and permit truck weights. Changing truck weight limits affect the truck-weight histograms. Because carrying heavier payloads may reduce the operating costs of truck operators, the possibility of a growing share of freight transportation shifting from rail to truck needs to be considered in estimating these histograms.

The objective of this project was to develop a methodology for estimating the bridge network costs associated with changes in legal and permit gross weight, axle weights, or axle configurations. This objective has been achieved with a recommended methodology for estimating changes in truck-weight histograms and for calculating the cost of fatigue and overstress in bridge components. To automate the recommended methodology, a software module that can be integrated with AASHTOWare Bridge-Ware was also developed.

This research was performed at Wayne State University, with the assistance of Fred Moses, Harry Cohen, Dennis Mertz, and Paul D. Thompson. The report fully documents the research leading to the recommended methodology. Step-by-step instructions for applying the methodology are included in an appendix along with detailed examples of the application of the methodology. The accompanying CD-ROM contains the software module implementing the recommended methodology, a user's manual, and the application examples described in Appendix B.

CONTENTS

CHAPTER **1**

INTRODUCTION

1.1 PROBLEM STATEMENT AND RESEARCH OBJECTIVE

TRB Special Reports 225 and *227* (“Truck Weight Limits: Issues and Options” and “New Trucks for Greater Productivity and Less Road Wear: An Evaluation of the Turner Proposal”) noted that trucks produce significant damage to highway bridges (TRB 1990a, 1990b). A truck’s gross weight, axle weights, and axle configuration (collectively referred to as “truck weight” in this study) directly affect the useful life of highway bridge superstructures. Such damage typically occurs in the bridge deck and in the main superstructure elements, including floor beams and girders, diaphragms, joints, and bearings. The severity of damage is a function of the structural element and its material. Bridge costs associated with increased truck weights are the result of the accelerated maintenance, rehabilitation, or replacement work that is required to keep structures at an acceptable level of service. Highway agencies require a network-level methodology for determining these costs.

A concern of agencies is the fatigue damage caused by the increasing population of heavy vehicles. Many of the details used in older steel bridge girders are particularly prone to fatigue failures directly related to truck weight. Repetitive loading may cause fatigue cracking in these steel members and limit the service life of a bridge.

Truck-weight frequency distributions by vehicle type (i.e., truck weight histograms, or TWHs) are needed to estimate reliably the effects on remaining life and the costs caused by changes in legal and permit truck weights. Changing truck weights can affect the truck weight histograms. Because carrying higher payloads can reduce the operating costs of truck operators, the possibility of a growing share of freight transportation shifting (e.g., from rail to truck) needs to be considered in estimating the future truck weight distribution and truck traffic.

The objective of this research is to develop a methodology for estimating the bridge network costs associated with changes in the limits on legal and permit gross weight, axle weights, or axle configurations.

1.2 SCOPE OF STUDY

This research project included the tasks listed below. This report documents the process, findings, and the product of the research effort. Note that bridge network costs herein refer to the costs to the *highway agency only*. Other costs—for example, costs to highway users—are beyond the scope of this study. Further note that the objective methodology is to estimate the incremental (or additional) costs resulting from truck weight limit changes, as opposed to the total costs for accommodating heavy trucks.

Task 1. Review relevant practice, performance data, research findings, and other information related to the effects of truck weight on bridge costs. Review fatigue-truck models and algorithms for predicting remaining fatigue life. This information shall be assembled from technical literature and from unpublished experiences of engineers and bridge owners.

Review literature and practice on predicting changes in truck-weight histograms following changes in truck weight.

Task 2. Describe the types and degrees of damage to bridge components (e.g., prestressed beams, steel girders, bridge decks) caused by increases in truck weight. Identify the data required to estimate the network cost of these damages. Prepare a recommendation on the priority for developing methodologies to estimate the cost of these damages. Develop an estimate of the cost and time to prepare a methodology for each significant type of damage to bridge components.

Task 3. Based on the information obtained in Task 1, propose an algorithm that predicts changes in truck-weight histograms and in fatigue-truck models caused by changes in legal and permit truck weight. Illustrate the application of the algorithm with specific examples.

Task 4. Outline a methodology to determine network maintenance, repair, and replacement costs resulting from fatigue damage to steel girder bridges subjected to increased truck weights. The outline shall include data requirements. Provide a discussion of how network costs will be estimated when data are missing or inadequate.

Task 5. Submit an interim report, within 6 months, to document Tasks 1 through 4 for review by the NCHRP. The report should contain a detailed proposed work plan for the comple-

tion of the project and specifically identify the methodologies that can be developed with the available funds. The contractor will be expected to meet with the NCHRP to review the report. Project panel approval of the proposed work plan must be received before work on the remaining tasks is started.

Task 6a. Based on the approved work plan, develop the fatigue-damage network-cost estimating methodology outlined for steel girder bridges in Task 4.

Task 6b. For other types of damage included in the approved work plan, develop methodologies for estimating the increased bridge network maintenance, repair, and replacement costs associated with proposed increases in truck weights. Provide a discussion of how network costs will be estimated when data are missing or inadequate.

Task 7. Prepare illustrative application examples.

Task 8. Prepare a detailed functional plan for developing a software module that can be integrated with AASHTOWare Bridge Ware to implement the methodologies developed.

Task 9. Submit a final report that documents the entire research effort and includes the methodologies as a standalone document. In addition, provide a companion executive summary that outlines the research results.

A second phase of this project was also conducted after fulfilling the above tasks. That phase had a main objective of developing a software module to implement the proposed methodology for estimating bridge network costs due to changes in truck weight limits. As a result, an Excel program was developed to facilitate such analyses to be performed by highway agencies in the country. A users manual was also developed along with the software to assist with application. The software module and its users manual are contained in the accompanying *CRP-CD-37*.

1.3 ORGANIZATION OF REPORT

Chapter 2 presents a comprehensive review on relevant subjects, with an emphasis on state of the art and state of the practice. In that chapter, Section 2.1 discusses approaches used in previous studies involving estimating bridge network costs. Section 2.2 summarizes the efforts and the results of a survey of state and other transportation agencies inside and outside the United States. These results helped prioritize cost impact categories to be included in the recommended methodology. Section 2.3 presents current understanding on the mechanisms of fatigue or deficiency of bridge components caused by heavy trucks. Section 2.4 reviews previously proposed methods for predicting changes in truck weight histograms as a result of truck weight limit changes. Section 2.5 offers a summary for that chapter.

Chapter 3 presents the concept of the recommended methodology developed in this project. Section 3.1 discusses the general structure and the principles for the estimation methodology. The requirement for data of the methodology is addressed as one of the major factors considered in the process of design and development. Section 3.2 is used to introduce the concept of a recommended method for predicting changes in truck load spectra. This is a fundamental step for all the cost-impact categories covered here, because truck load spectra are the driving force for the relevant bridge network costs. The following sections then present the estimation methodology for each of the four cost-impact categories, as follows: (1) Fatigue of existing steel bridges, (2) Fatigue of existing reinforced concrete (RC) decks, (3) Deficiency due to overstress for existing bridges, and (4) Deficiency due to overstress for new bridges. Section 3.7 discusses the principles for summing the costs for individual cost-impact categories.

Chapter 4 summarizes the conclusions of this study.

Appendix A presents the procedure of the recommended methodology for estimating bridge network costs as a result of truck-weight limit changes. This methodology is for U.S. highway agencies at various levels to predict such costs for planning purposes. It covers four cost impact categories mentioned above. This appendix has separate sections respectively for each of the four categories prioritized in this study. The concept of this methodology is presented in Chapter 3, including the supporting theory and background information. Appendix A also contains several data sets to be used as the default data for application when more detailed site-specific or jurisdiction-specific data are not available. They are intended to meet the minimum requirement for input data to facilitate implementation of the recommended methodology.

The recommended methodology has been applied to two bridge networks as presented in Appendix B. The first example is for the bridges of two routes in the state of Idaho for an increase in permit truck weight limits. In that example, the upper limit was increased from 467 kN (105 kips) to 574 kN (129 kips) for gross vehicle weight (GVW). Three scenarios were investigated: (1) The truck weight limit change is effective only for the two specific routes; (2) The change is implemented in the entire state; and (3) The change is legalized in the entire state (i.e., no permit would be needed to carry the weight of 129 kips if the Bridge Formula is satisfied). For these three scenarios, only the bridges on the two specified routes were covered. The second example applies the recommended methodology to the state of Michigan for legalizing the 3S3 truck configuration in the entire state. The first example covers a smaller number of bridges within the state network. This small size of network permitted a relatively

more detailed analysis. In contrast, the second example estimates the impact costs for a much more extensive network for the entire state of Michigan. These examples illustrate the application of the recommended methodology.

The attachments to this report include the developed software module named "Carris" and its users manual. The software is written using the Microsoft Excel for interactive application. Two application examples have been prepared for using the software. One is the Idaho Example's Scenario 2 and the other is the Michigan Example. The software for the examples and the users manual are contained in *CRP-CD-37*. Note that application of software requires the user change either of the examples in the CD to the case of interest, as explained in the user's manual.

CHAPTER 2

FINDINGS

Trucks were first manufactured in the United States in 1898 (Rudra http://www.marion.lib.in.us/history/indtrucks/report.html). Trucking and the highway system have developed rapidly during the past 100 years in this country. In 1904 there were approximately 328,000 km (204,000 mi) of surfaced roads and streets in the United States. By 1990, there were about 6.4 million km (4 million mi). Over the same period, motor truck registrations have grown from 1,500 to 7.2 million (RJHansen 1979, USDOT 1991b). Motor trucks operating on the highways now provide service to every community in the country. In 1989, for example, trucks traveled more than 2.1×10^{12} vehicle miles (vehicle miles of travel or VMT). This represents a 4.2 percent annual increase from 1983 to 1989, and a 4.7 percent increase from 1985 to 1989. Trucks deliver a significant portion of the nation's product. In 1974, for example, this included 60 percent of all intercity shipments of manufactured products, 80 percent of all fruits and vegetables, and 100 percent of all livestock (RJHansen 1979).

While we benefit from truck transportation, highway agencies spend a significant amount of resources to establish and maintain the highway system in the country. Quantifying the causes of the expenditure has been a focus of several studies in a number of countries. This chapter reviews several aspects related to estimating the cost effects of heavy trucks on highway bridges. This review discusses the latest developments in relevant areas.

Section 2.1 covers the approaches previously used in bridge network cost estimation studies. It helps explain the state of the art in these areas. Based on this understanding, four cost impact categories are prioritized in this study, to be covered in the recommended methodology. They are: (1) fatigue of existing steel bridges, (2) fatigue of existing reinforced concrete decks, (3) deficiency due to overstress for existing bridges, and (4) deficiency due to overstress for new bridges.

Section 2.2 summarizes the survey conducted in this study to understand state of the practice that is relevant to the targeted cost impact estimation. It provides useful and critical information on what data are available or may become available for application of the recommended methodology. This information also helped in determining the appropriate data requirement for the methodology. This requirement should be set such that state agencies will be able to meet it with a minimum effort and that available data are used to the largest extent practically possible.

The following Section 2.3 then presents the mechanisms and current state of knowledge on each of the four prioritized cost impact categories. It also provides the foundation for quantifying the cost impacts in the recommended methodology. Section 2.4 presents a review of predicting changes in truck weight histograms (TWHs) following changes in truck weight limits. The resulting TWH changes are critical to cost impacts, because they represent load changes to bridge structures and are the cause of cost impact.

2.1 RELEVANT COST-IMPACT STUDIES AND APPROACHES

The subject of truck weight effects on bridge costs has attracted research attention for many years. Relevant recent studies are reviewed below in a chronological order, to have an overview of state of the art and the practice in this area. This review covers the various approaches taken. Other studies, either less recent or less representative, are listed in the Bibliography section.

2.1.1 Study by Yoder et al. for Indiana DOT

In 1979, a study was conducted by Yoder et al. (1979) to investigate the impact of a GVW limit increase from 326 kN (73.28 kips) to 356 kN (80 kips) for Indiana DOT. Both bridges and pavements were covered. The following cost impacts were included in this effort for bridges: (a) strength-related costs, (b) steel fatigue-related costs, and (c) deck deterioration costs. The strength-related costs refer to inadequate load carrying capacity of bridges under the new permissible load. It was found to be insignificant for a range of overstress allowance used, being 12, 23, and 35 percent for flexure of prestressed concrete, steel, and reinforced concrete members respectively, and 30, 23, and 35 percent for shear of these members, respectively. The steel fatigue-related costs were also estimated to be negligibly small, based on the data available at the time. Impact costs associated with bridge deck deterioration were estimated using an assumption that cost increase is linearly related to the maximum permitted GVW (Whiteside et al. 1973). An annual increased cost of \$2 to \$3 million was then arrived at, based on an 11 percent increase of GVW.

This study represents an early effort in this area when many data and methods used today were not available. The basis of the overstress criteria was not documented, and they are certainly different from those used in more recent studies below. Apparently, steel fatigue was not as well understood as today. On the other hand, bridge deck deterioration was clearly acknowledged although the damage cost model (i.e., the cost increase is linear with GVW) is not well founded.

2.1.2 Study by BTML for New York State DOT

Byrd, Tallamy, MacDonald, and Lewis (BTML) conducted a study in 1987 for New York State DOT on effects of permit truck weights including those on bridges (BTML 1987). That study reviewed pertinent experience available at the time. Only steel fatigue-induced costs were estimated. An annual cost of $23,500 to the state was projected and thought attributable to the annual permits for divisible overloads. These permits are valid for a year (but renewable) for unlimited trips, which represent the “grandfather exemption” allowed by the federal legislation. This estimate was arrived at using the following approach: (a) A single (typical) bridge was used to project to several hundred bridges thought impacted. (b) One type of fatigue-prone detail (cover plate weld) was considered. (c) The cost impact was estimated as a result of different fatigue accumulations with and without permit-trucks, including the effect that these permits could allow heavier loads but would result in fewer trips. (d) The fatigue damage was estimated using the approach in the AASHTO *Guide Specifications for Fatigue Evaluation of Existing Steel Bridges* (1990), based on the work of Moses et al. (1987). (e) Fixed (zero discount rate) annual costs for 20 years were estimated based on more intensive and detailed inspection and probable repair at the end of estimated fatigue life.

Apparently, information on individual bridges now is more available, and cost estimation could be therefore improved. These annual (renewable) permits should be treated as routine loads on bridges. Thus the design load would also need to be accordingly increased for new bridges expected to carry such loads. More significantly, the bridges that will become strength deficient under these routine loads will need either strengthening or replacement, if the permit trucks’ access to them is not restricted.

2.1.3 Study by Moses for TRB

In 1989, Moses completed a study on effects of proposed new truck weights on bridges for TRB (Moses 1989). The results were summarized in *TRB Special Reports 225* (1990a) and *227* (1990b). This study assessed the costs of fatigue life reduction and substandard load ratings caused by a variety of proposed new truck-weight limits. Both existing and new bridges were covered in the study. Totals of up to several billion dollars per year were estimated as the costs to the nation, depending on the scenario of the proposed truck weight limit. While the cost associated with substandard load ratings was carefully documented, little data were available for the actual fatigue damage costs of steel bridges due to heavier truck loads. Thus expert opinions were solicited on this item. Their cost estimates varied from very small amounts to over $50 million per year nationally. Nevertheless, the cost for replacing substandard bridges due to strength rating was found to be dominant among all the bridge cost impacts considered. Note that this was the first effort of including new bridges as a cost impact category. Moses (1989) also reported incremental costs for the new load limit scenarios considered, which are useful for applying the recommended methodology when more specific data are not available.

2.1.4 Study of Minnesota DOT

As a response to the *TRB Special Report 225* (1990a), Minnesota DOT (1991) investigated the bridge related impacts of the TRB recommendations at the state level. These scenarios were examined: the TTI HS-20 and TTI HS-20/Formula B with a cap of 489 kN (110 kips) or 556 kN (125 kips). The cost impacts were estimated for strength deficiency and steel fatigue both due to higher loads, as well as the costs for enforcement for safety and weight limits, posting, safety, and engineering. For bridge deficiency, reaction (posting, replacement, or their combination) was determined considering the bridge’s specific situation. For example, bridges already eligible to be replaced did not contribute to any cost increase. For steel fatigue, a moment increase factor was calculated first, being the ratio of the moments due to the worst load case under the new regulations and the current legal trucks. A minimum of 1.20 was found for the cases considered. This ratio was then used to estimate the reduction of fatigue life (due to stress range increase). The AASHTO *Guide Specifications* procedure (1990) was used and all other parameters were kept unchanged in the calculation (except the stress range). In addition, maintenance cost increase was mentioned as a possible impact, particularly for bridge decks. However, a specific number could not be given to quantify the increase, apparently due to lack of quantitative knowledge on deck deterioration caused by truck load.

The reaction to bridge deficiency used in this study (posting, replacement, or their combination) was realistic and reasonable. Today, bridge-related data are better available, and this kind of detailed action selection is possible for state agencies. Further, the fatigue cost impact estimation can be done with more details. For example, historical traffic data could be used to estimate the fatigue life already used. Further, predicted future traffic data available in agency bridge inventories may be used for a more reliable estimation for the remaining fatigue life.

2.1.5 Study by IIT for Illinois DOT

Since increasing the state truck weight limits in 1983, from 326 kN (73,280 lb) to 356 kN (80 kips), Illinois DOT has reported triennially on the effects of this state-legislation change (IDOT 1992). In the first triennial report, specific effects on bridges were not included, primarily because the increased loading was not expected to produce adverse effects. A more in-depth look at the issue in the second triennial report suggested that the heavier trucks were contributing to a reduction in the service life for some older bridges. An annualized cost of this deterioration was estimated at $9 million. The third triennial report concluded an updated annual cost of $12.3 to $30 million attributable to the increased weight limits, based on a study done by Illinois Institute of Technology (Mohammadi et al. 1991). A sample of 15 bridges were measured and analyzed under truck loads. The results were used to project to a population of 1,059 bridges. The shortened life for the 15 sample bridges ranged from 0 to 37 years. The impact costs were based on bridge replacement at the end of the estimated fatigue life. It was stated that a number of bridges have been replaced since 1983 when the new GVW limit was implemented.

It should be noted that many states repair and monitor fatigue cracks when detected, according to the responses received to the survey of this project to be discussed in the next section. To say the least, not every bridge owner replaces bridges because of calculated or observed fatigue damage. Thus, for estimating bridge costs due to truck weight limit changes, highway agencies should have the option to select the most appropriate action, according to the specific situation and the adopted criteria. Apparently, the estimation result will be a function of the action selected in responding to the assessed fatigue and deficiency.

2.1.6 Study by Sorensen and Manzo-Robledo for Washington State DOT

In 1992, Sorensen and Manzo-Robledo reported a study for Washington State DOT, estimating the impact of the Turner trucks on Washington State bridges (1992). There were 3,079 bridges on the Washington State roadway system in 1989 when this study started. 2,024 of them were identified to be strength deficient under the Turner truck scenarios. All of them are concrete bridges. The screening was done through a database search, using current load ratings. Replacement cost for these deficient bridges was estimated as the cost impact. A remaining life factor was used to attribute a portion of the total replacement cost to the Turner truck impact. No detail was given in the report as to how this remaining life factor was determined, but it is stated that fatigue was not considered in this study. The cost of a bridge was estimated by multiplying a unit cost per deck area and the total deck area of the bridge. The unit cost was provided by the state agency.

This study's level of detail may represent a typical situation with respect to data availability. Namely, for deficient bridges, the main data needed are the current load ratings and bridge unit cost per deck area. These data are readily available now.

2.1.7 Study by Moses for Ohio DOT

This ODOT study was done by Moses (1992) to develop a permit fee system based on bridge damage costs. The study covered fatigue and increased number of posted spans. A review of different state practices found that permit fees vary considerably over the nation. The impact costs for the state system for various routine permits were calculated with the same cost methods for the TRB study described in Section 2.1.3. A permit fee system was proposed for Ohio based on these cost impacts. The fees were calculated according to a ton–mile model, which increase for loads above the current legal levels to reflect the cost impacts.

2.1.8 Study by IBI/ADI for Canada

During 1988 and 1989, a Memorandum of Understanding (MOU) was implemented in Canada, which allowed generally larger and heavier trucks to operate on designated highways. A study investigated its impacts (IBI/ADI 1994), including a section of impacts on highway bridges. Both bridge overstress and steel bridge fatigue were acknowledged, but only the cost effect for the former was quantified. This was mainly due to the limited available data and the estimated costs to obtain the needed data being beyond the study scope. A total one-time-cost of $32 million was estimated for upgrading current bridges to be caused by overstress. Two more points need be noted here: (1) The new truck weights were lower than Ontario's regulatory loads already in place, thus Ontario's contribution to the total cost was zero. (2) The contribution of British Columbia was extrapolated from another province to estimate the severity of overstress, if any. This was because the required load ratings were not available for British Columbia.

The U.S. state agencies have a comprehensive database of load rating for the bridges within the jurisdiction. This is not to say that the recorded load ratings are exact for every bridge, because some of these ratings had to be approximated due to inadequate information. Nevertheless, using these data will lead to a higher fidelity in cost impact estimation.

2.1.9 Highway Cost Allocation Studies of FHWA

In 1997, the FHWA completed a highway cost allocation study (FHWA 1997), following an earlier one in 1982 (FHWA 1982). The new study developed bridge cost responsibility (in percentage) for various vehicle fleets, besides other products.

These responsibilities are applicable to federal expenditures on highway bridges. Four groups of costs were covered: new bridges, bridge replacement, major bridge rehabilitation, and minor bridge rehabilitation.

For new bridges, each vehicle fleet's cost responsibility was estimated as the incremental cost required to accommodate that fleet's operation, based on the bridges' design load. Incremental costs were developed for this purpose by considering bridges designed for loads from H-2.5 to H-20 and HS-15 to HS-25 with various increments. Within each fleet, the cost was further distributed according to the VMT. For bridge replacement and rehab costs, load-related costs are allocated to the vehicles that occasion the action, while all vehicles share the responsibility of non-load-related costs. Although fatigue consumption cost to bridges was not explicitly mentioned in the final report (FHWA 1997), there was an attempt to allocate this cost to the vehicle fleets (Laman et al. 1997). Fatigue was considered for both steel components and reinforced concrete (RC) decks. A total of 39 bridges were used to project to the nation's bridge population. It is worth noting that the latest results of research on RC deck fatigue (Perdikaris et al. 1993) were not mentioned in a study by Laman et al. (1997). These results actually are relevant to decks in the US. This will be discussed further in Sections 2.3.2 and 3.4.

Although truck limit changes were not within the scope of this FHWA study, some data developed are relevant to the present NCHRP project. For example, the incremental bridge costs computed for various design loads are useful for application of the recommended methodology contained in Appendix A. These costs were calculated as percentage increases with reference to the HS-20 design load. Part of these data is synthesized in Appendix A as part of the default data that may be used when applying the recommended methodology. These data cover bridges of simple and continuous spans with steel I-girders, I-rolled beams, RC slabs, RC T-beams, prestressed concrete beams, slabs, and multi-cell box beams. The span length varies from 30 to 240 ft, depending on the material and structure type. Note that Moses (1989) also reported incremental bridge costs for each scenario of the proposed truck weight changes investigated. These two sets of cost data are included as the default data in Appendix A.

2.1.10 Study by Ministry of Transportation of Ontario

Recently, the Ministry of Transportation of Ontario (MTO) conducted a study on the effects of several scenarios of truck weight limit change on highway infrastructure (MTO 1997). The effect on bridges was divided into two categories: deficiency due to overstress and life reduction by steel fatigue. A number of bridges were analyzed using models intended to represent respective groups according to type, span length, and the design load used. The output for these groups was projected to a population of 13,200 bridges for cost-effect estimation, based on bridge replacement.

For deficiency due to overstress, a probability is estimated for each representative bridge's ultimate strength being exceeded. This probability was then defined as the percentage of the bridges to be replaced for the group represented. For life reduction due to steel fatigue, the replacement cost is estimated based on the reduced life with reference to an assumed 50 years of original fatigue life. Several millions of dollars were found to be attributable to increased truck weights, depending on the scenario of truck weight limit.

Several points need to be noted here. (1) The analysis of this MTO study included only those fatigue-details equivalent to the A, B, and B′ categories of AASHTO (1996), which are not the most vulnerable details commonly seen in existing bridges in the United States. (2) The proposed owner responses to the truck weight limit changes do not appear to be consistent with U.S. practice. For example, the 50-year fatigue life has been only a philosophical concept and was not quantitatively implemented in design, at least for a vast majority of the U.S. bridges currently in service. When fatigue damage to a component is detected, replacing the bridge may not be the only choice. Further, the probability of the ultimate strength being exceeded certainly is not correlated with the rate of bridge replacement, at least in the U.S. practice, because strength may not necessarily be considered at all in deciding replacement. For example, when load ratings by the allowable stress method are used as one of the factors for deciding bridge replacement, the ultimate strength plays no role in this process.

2.1.11 Study by Heywood for Australia

Heywood and Pearson (1997) conducted a study on cost impact of projected truck weight limit increases for bridges in Australia. Three scenarios of truck load increase were considered there. Only strength deficiency was addressed, and the replacement cost was estimated as the impact. 6,690 bridges were covered in this study of the Australia's National Highways and other primary rural roads. The severity of an identified deficiency was used to determine what action to take. Two levels of severity were included, with a threshold defined to distinguish them. When the deficiency is above the threshold, immediate replacement was considered to be necessary and the total cost is included. Otherwise, only a portion of the replacement cost was attributed to the weight limit change, depending on the bridge's current age using a remaining life model. However, the database for developing this model was not given in the paper. Furthermore, the model apparently was not based on fatigue damage accumulation, because it is a function of the load level only.

2.1.12 Study for the State of Western Australia

Another relevant study in Australia (Bridge Branch 1997, 1998) covered all 2,657 bridges in the State of Western

Australia to estimate cost impacts for possible increases in truck weight limits. Only existing bridges were considered. This effort included two stages: (1) The first stage identified deficient bridges using an existing load rating database, by comparing the moments due to the rating vehicle and the vehicle being considered. (2) The second stage is a bridge-by-bridge analysis for 243 identified deficient bridges. For each bridge, strengthening, rehabilitation, or replacement was selected according to the situation. The associated costs were estimated according to the action selected. On the other hand, no fatigue-related or new-bridge-related costs were included in this study.

2.1.13 Truck Size and Weight Study of USDOT

This FHWA study establishes an ongoing truck size and weight (TS&W) research activity within USDOT. Its 1998 report (USDOT 1998) summarizes a phase of the project relevant to the present NCHRP study. In that phase, several vehicle scenarios of truck weight limit change were considered and compared with a base case to estimate the cost impact. Each scenario included seven or eight truck configurations. The base case represents the condition in Year 2000, absent of any significant changes to the nation's TS&W rules. Then cost impacts of these proposed scenarios were estimated with reference to the base case. Bridge-related costs were considered as part of the agency costs.

The bridge cost impact was estimated based on replacements triggered by overstress due to the scenario vehicles beyond an overstress allowance. This allowance is 30 percent for bridges designed for H-15 loading and 5 percent for bridges designed for HS-20 loading. This set of overstress criteria does not appear to be completely consistent with highway agency practice. While load ratings do play a role in deciding bridge replacement, they are certainly not the only factor considered there. Further, the quantitative overstress criteria (30 percent and 5 percent, respectively, over H-15 and HS-20) are not typically practiced by the agencies in the country.

In this USDOT study, fatigue was considered to be secondary for the following justifications given in the report: (1) It generally affects only steel bridges whose share in the nation's bridge population is decreasing. (2) Fatigue damage can generally be repaired inexpensively. (3) Most bridges have been designed with an adequate fatigue code. It should be noted that the effort reported in Laman et al. (1997) was also part of this study, although it was not explicitly included in the report (USDOT 1998). As mentioned in Section 2.1.9, Laman et al. (1997) did develop cost responsibilities of vehicle fleets for steel fatigue.

The reported FHWA method can be improved for meeting the objective of the present NCHRP project. At the state level or a local level, cost impact estimation could be conducted in more detail, because more detailed bridge data are available and the number of bridges becomes smaller. This is particularly true when dealing with each bridge's load rating and selecting the response to anticipated overstress due to changes in truck weight limits. The objective methodology is to be realistically consistent with agency operation.

2.1.14 Summary

The following conclusions are made as a summary of the above review. (a) For estimating bridge-network cost impacts of truck weight limit changes, realistically selecting the action responding to the identified bridge fatigue failure cost and deficiency is important to reach credible results. (b) Estimating cost impact of deficiency due to truck weight changes has typically used a deterministic approach to selecting the load requirement. This can be improved by introducing probability-based decision approaches covering bridge safety. The latest AASHTO code development has established a benchmark in this regard (AASHTO 1998, AGLichtenstein & Associates 1999). (c) With load ratings and more detailed information available for individual bridges, fidelity of cost impact estimation can be enhanced. (d) Fatigue cost impact should be estimated realistically by considering actions likely to be taken by agencies to address estimated or observed damage. Also, it should use available data (e.g., truck traffic volume and presence of fatigue-prone details) to the fullest extent. (e) Fatigue of reinforced concrete decks has not been quantified, although it is acknowledged. This situation also needs to be improved, because a significant amount of resources has been used to perform deck renewal, including overlay and replacement. (f) The impact on new bridges has been investigated. It can be improved by quantitatively considering the uncertainty involved in future loading. It will be consistent with the latest AASHTO design and evaluation specifications adopted and under review (AASHTO 1998, AGLichtenstein & Associates 1999).

In summary, the objective methodology should be consistent with highway agency practice, as guided by current AASHTO specifications. It also should represent state of the art in analysis techniques and use the available data to the fullest extent. The improvements discussed above have been implemented in the recommended methodology presented in Appendix A.

2.2 HIGHWAY AGENCY PRACTICE RELEVANT TO COST-IMPACT STUDY

A survey of highway agencies was conducted in this study to obtain the following information: (1) Recent experience in predicting truck weight histograms and modeling steel-bridge-fatigue (reduced life). (2) Recent experience with estimating bridge-network costs. (3) Data availability for such estimation (e.g., detailed data of specific damage histories,

bridge related data, etc.). (4) Current practice of treating bridge damage and deficiency, to help validate steps to be included in the recommended methodology, as well as those used in the studies reviewed above.

With assistance of the project panel, NCHRP, and several state agency personnel, two questionnaires were developed for bridge owners within and outside this country, respectively. These questionnaires were developed to minimize the effort required to answer the questions, but to maximize useful leads to more detailed information. On the other hand, this approach required significant efforts for follow up contact to acquire more detailed information.

Thirty-eight agencies responded to the domestic questionnaire, including 36 states, Port Authority of NY and NJ, and NY Thruway. Twelve responses were received from 7 other countries—Australia, Canada, Germany, Japan, New Zealand, Switzerland, and United Kingdom. The results are discussed below.

2.2.1 Prioritizing Cost Impacts Covered in the Objective Methodology

The survey solicited cost impact categories other than the ones mentioned above: (1) fatigue of existing steel bridges, (2) fatigue of existing reinforced concrete decks, (3) deficiency due to overstress for existing bridges, and (4) deficiency due to overstress for new bridges. As a result, the following items were suggested once each by one of the 38 domestic agencies that responded: (1) Hinges and open grid steel deck; (2) timber bridges and small structures (such as culverts); (3) prestressing tendon fatigue and prestressed concrete beam cracking; (4) superstructure uplifting; and (5) expansion joints and bridge railings.

For each of these items, the agency raising the issue was contacted to understand the scale of the problem, the effort spent on investigation if any, and available data to quantify associated costs. Further, a literature search was made for the particular item. It was found that none of these items represents a widely spread problem, or little research effort has been reported on them to provide the data and understanding needed to quantify the cost impact, if significant. With consideration to the time and funding constraints, these items are not prioritized in this research effort.

In the received responses to the domestic questionnaire, no agency reported fatigue damage of prestressed concrete beams. Further, only Washington DOT indicated a case of steel expansion joint fatigue failure. A follow-up discussion with the agency's contact engineer found that it occurred to a very special type of joint, which is not widely used in the country. Thus, these two cost-impact categories were not prioritized here in this project. Furthermore, collision damage to bridges by trucks was mentioned in one of the conversations with state agency personnel. However, it was considered accidental and not necessarily directly correlated with truck weight limit changes.

2.2.2 Accomplishments with Respect to the Survey Objectives

For recent experience in predicting TWHs and modeling steel bridge fatigue, the survey identified no new developments beyond what is summarized in Sections 2.3.1, which is based on the research team's experience. For Question I-4 regarding whether experience or data exist in understanding truck traffic reaction to possible weight limit changes, a few responses indicated positive answers. However, follow-up contacts found no analysis results that could be useful for developing the recommended methodology in this project. On the other hand, the survey did identify several recent studies on estimating bridge cost impact resulting from truck weight limit change. They have been included in the review presented in Section 2.1.

With respect to data available for the application of the recommended methodology (Question I-8), this survey found that most state agencies (29 out of the 36 that responded) have a database of bridge-related costs. These databases can be used in application of the recommended methodology. Obviously, these jurisdiction-specific data would be more accurate than the default data provided in Appendix A. On the other hand, only 7 out of the 36 states indicated that they keep track of fatigue damage-related costs (Question II-1). Further, 6 out of the 36 states indicated that they have studied cost increase by designing bridges for higher design load. Follow-up contacts for gathering these cost data actually found out that the data are not readily available, either because they were not filed or not quantitatively calculated in the first place. This finding indicates a set of data that needs to be provided for application of the recommended methodology. (This need is met by providing the default data included in Appendix A, as an attachment to the recommended methodology.)

This survey also tried to understand agencies' typical reaction to observed damage and deficiency of bridge components, for developing a realistic cost-estimating methodology. According to the answers to Question II-1, all state agencies have experienced steel fatigue failure (cracking). Repairing and monitoring fatigue damage appear to be the common practice. Twenty out of the responding 36 states indicated that they replace damaged components. Thirty-four out of 36 states repair steel bridge members with fatigue damage. For deficiency in load carrying capacity (quantified by load rating), the immediate reaction usually is posting. Then other longer-term solutions (strengthening or replacement) are implemented if warranted. In this respect, action for addressing reinforced concrete bridge deck damage and deterioration is more variable. Understanding these options leads to having reaction options in the recommended methodology. This should be consistent with the state's practice.

2.2.3 Others

The received responses also indicated several other interesting facts: (1) 16 out of the 36 responding states have

reported that there have been changes to their weight limits since 1980, with 3 states not answering that question. The details given by the agencies about these changes all show an increase in the truck weight limits. (2) Truck traffic has not been extensively studied, or at least these studies have not been made available to the personnel who were involved in responding to the questionnaire. This situation is more severe when it comes to illegal over-weighted trucks. (3) A few states (7 out of the responding 36) have fatigue-prone details inventoried. This indicates a plausible trend of establishing such databases. On the other hand, it also highlights a need for the recommended methodology to provide guidelines for identifying vulnerable bridges for fatigue cost impact estimation, when no such database can be used. (4) In terms of actions taken in response to damage and deficiency and efforts spent on studying relevant issues, a similar trend was observed with those agencies outside the country.

2.3 FATIGUE AND DEFICIENCIES OF BRIDGE COMPONENTS CAUSED BY HEAVY TRUCKS

Heavy trucks demand highway bridges to have certain load-carrying capacity. This demand dictates the bridge design load. It is also observed that heavy trucks more directly consume these facilities than other lighter vehicles (such as cars and 4-tire light trucks). In 1979, the U.S. General Accounting Office (US GAO) submitted a report to the Congress, entitled "Heavy Trucks: A Burden We Can No Longer Take" (US GAO 1979), about truck weight effects on highway deterioration at the national level. A questionnaire was sent to the states to gather information in this regard. With respect to legal trucks' contribution to highway deterioration, the following opinions were reported:

Extent of Trucks' Contribution to Highway Deterioration	Number of States
To very great extent	5
To substantial extent	21
To moderate extent	17
To some extent	6
To little or no extent	0
No response	1

As seen, a large majority (86 percent) of the states felt that trucks contribute at least moderately to highway facility deterioration.

Based on the review of relevant studies presented above and other published and unpublished experience of bridge engineers, the following four cost-impact categories are quantifiable using information available to highway agencies:

1. Fatigue of existing steel bridges,
2. Fatigue of existing RC decks,
3. Deficiency due to overstress for existing bridges, and
4. Deficiency due to overstress for new bridges.

In addition, mechanisms of the categories are better understood compared with other cost impacts. Thus these four categories are prioritized in this study and included in the recommended methodology for predicting induced costs to the agency as a result of truck weight limit change. Each category's mechanism is discussed below in more detail. Knowledge needed to quantify the impact costs is also reviewed in this section to provide a foundation for the approaches used in the recommended methodology.

Note that the order of these prioritized items has no significance but is used for convenient reference. The prioritization of these four categories has considered present understanding of the associated phenomena, available data, and relative significance in the total cost results. For each of the prioritized cost impact categories, two levels of data requirement are recommended, which respectively correspond to two levels of analysis detail. The lower level (Level I) represents the minimum level of data requirement or analysis detail. It is envisioned that all state agencies will be able to meet that level of data requirement for the application of the recommended methodology. The higher level (Level II) represents the highest level for data requirement (and corresponding to the highest level of analysis detail) possibly reachable currently or in the foreseeable future. For example, the level II analysis is envisioned suitable for a true bridge-by-bridge application for steel fatigue assessment, when detailed information on each bridge is made readily accessible through Virtis. On the other hand, currently Virtis is not fully loaded. Thus, it is expected that no state agency may be able to use the Level II analysis for all four cost-impact categories prioritized here because the data availability varies from state to state and from category to category, sometimes very significantly.

2.3.1 Steel Bridge Component Fatigue

Fatigue of steel bridge components has been extensively investigated. Moses et al. (1987) contains a comprehensive list of references on this subject. The survey results for this project indicate that the vast majority of state agencies have experience with fatigue damage (cracking). According to the principles of fracture mechanics, fatigue damages originate from microscopic discontinuities in the material under cyclic loading. These discontinuities cause stress concentration, with a stress much higher than that the member is normally expected to withstand. For steel bridge components, such discontinuities may be caused by lack of fusion in a weld, sudden geometric change at a connection, etc. Current fatigue life estimation in the United States is based on the fatigue category (likelihood of discontinuity), nominal stress range, and number of stress cycles.

Based on quantitative understanding of fatigue behavior by physical testing, current AASHTO bridge design and evaluation specifications include provisions covering this subject (AASHTO 1994, 1996, 1998). Note that the new generation of codes (AASHTO 1990, 1994, and 1998) represents a more realistic approach to modeling and estimating fatigue accumulation. Further note that, despite intensive research efforts spent on this subject, a notable amount of uncertainty is still present in the design and evaluation process as guided by the code provisions. This uncertainty has been quantitatively addressed in the AASHTO codes (1994 and 1998). The recommended methodology uses AASHTO (1990, 1994) and its anticipated load and resistance factor (LRFR) version (AGLichtenstein & Associates 1999) for estimating steel fatigue damage accumulation, as included in Appendix A.

This section reviews the development of the fatigue life assessment procedures contained in the AASHTO *Guide Specifications for Fatigue Evaluation of Existing Steel Bridges* (1990). The new AASHTO *LRFD Bridge Design Specifications* (1998) and the proposed AASHTO LRFR bridge condition evaluation manual (AGLichtenstein & Associates 1999) both excerpt their provisions from the 1990 guide specifications. All of these documents rely, for the calculation of fatigue life of steel attachments, on the work performed in NCHRP Project 12-28(3) and published in *NCHRP Report 299* (Moses et al. 1987), as well as the supporting truck loading database, reliability studies, and fatigue tests carried out in recent years. Of special concern to this NCHRP project is the influence of the truck weight distributions on assessing remaining fatigue life and what, if any, developments have occurred since the fatigue provisions cited above were first proposed.

2.3.1.1 Fatigue Damage Accumulation

Fatigue is a cumulative process in which repetitive stress cycles accumulate damage until failure occurs. The basic concept of the fatigue design and assessment for bridges relates to the fact that each cycle of truck passage causes some damage. The damage due to a population of trucks accumulates until failure (cracking) occurs. The damage caused by each truck depends on the vehicle weight, the bridge's span length, and member section dimensions.

Based on experimental data and fracture mechanics principles, it is observed that

$$\text{Fatigue damage is proportional to (Stress range amplitude)}^3 \qquad (2.3.1.1)$$

The stress range is the difference between the maximum and minimum stress caused by a vehicle passage at the location of concern. The exponent of 3.0 in Eq. 2.3.1.1 for the welded steel attachments is an important parameter in comparing influences of variable stress amplitudes. It means that if the stress amplitude is doubled, the fatigue damage will increase by a factor of eight. To account for different stress ranges due to various truck weights, a linear damage accumulation law is usually assumed (Moses et al. 1987). The damage of one stress cycle is inversely proportional to the life that would exist if that stress of constant amplitude were cyclically repeated. The life for constant stress amplitude is predicted using the stress-life (S-N) curve for that type of attachment based on physical testing. Thus, in a non-dimensional form, failure (cracking) occurs when a damage sum D equals 1.0 (Miner's rule):

$$D = \Sigma \frac{n_i}{N_i} \qquad (2.3.1.2)$$

where n_i is the number of stress cycles due to vehicle weight and class i. N_i is the number of cycles to failure from the S-N curve if only the stress corresponding to vehicle weight and class i were applied.

Using the data developed from tests and the exponent of 3.0 mentioned above, a constant amplitude fatigue life leads to:

$$N_i S_i^3 = b \qquad (2.3.1.3a)$$

where S_i is the constant stress range leading to the number of cycles to failure, N_i. Factor b is a constant depending on the fatigue strength of the detail, and it is explicitly considered and tabulated in the design and evaluation procedures (AASHTO 1990, 1998). Eq. 2.3.1.3a can be rewritten in a different form:

$$\text{Log } S_i = (1/3) \text{ Log } b - (1/3) \text{ Log } N_i \qquad (2.3.1.3b)$$

or

$$\text{Log } S_i = A + B \text{ Log } N_i$$
$$A = (1/3) \text{ Log } b; \qquad B = -(1/3) \qquad (2.3.1.3c)$$

where Log is the logarithm function.

2.3.1.2 Fatigue Truck Modeling

Furthermore, let the stress amplitude S_{ij} for a given vehicle type j and weight level i be taken as proportional to the vehicle weight W_{ij}:

$$S_{ij} = K_j W_{ij} \qquad (2.3.1.4)$$

where K_j is a constant depending on the vehicle's configuration (axle spacings and weight distribution among the axles) and bridge span configuration. For a particular truck type and weight, the frequency of truck weight is $f(W_{ij})$. Substituting for the damage accumulation due to given vehicle class j shows that:

$$D_j = K\, \Sigma_i\, f(W_{ij})W_{ij}^3 = K\, [W_{eq,j}]^3 \qquad (W_{eq,j} = [\Sigma f(W_{ij})W_{ij}^3]^{1/3}) \qquad (2.3.1.5)$$

where K is a constant, and $W_{eq,j}$ is then the equivalent weight of vehicle class j that provides the same fatigue damage as the vehicles of that particular vehicle class. The magnitude of $W_{eq,j}$ is sometimes referred to as the fatigue truck weight and equals the cube root of the sum of the cubes of the truck weight distribution. It was observed from weigh-in-motion (WIM) studies (Moses et al. 1987) that the dominant contribution to fatigue damage of bridge spans comes from the 5-axle tractor-trailer vehicles (3S2s). Thus all truck classes were converted to a single type such that the overall damage could be expressed by the response of the span to a single 3S2 truck with fixed wheel base and axle load distribution. Fig. 2.1 shows this truck for fatigue damage evaluation (AASHTO 1990). Note that each of the two heavier axles, weighing 0.4444 of the GVW, may be viewed to represent 2 tandems of a typical 3S2 vehicle.

To calculate fatigue stresses and evaluate fatigue life, it is convenient to use such a single-vehicle representation in both design and evaluation. The latest AASHTO specifications for design (1998) and evaluation (1994) use this fatigue truck given in Moses et al. (1987). This vehicle was based on observed data from WIM studies in the early 1980s. The fatigue truck weighs 54 kips and its dimensions were selected by examining over a range of bridge spans, so that a random sample of trucks would produce the same fatigue damage for a fixed volume of traffic as the fatigue truck with the same number of crossings.

Both the fatigue design and evaluation procedures were introduced to also provide a predicted fatigue life with some margin of safety. These margins were derived so that for redundant spans (component safety), there was only a probability of 2.7 percent that the failure (cracking) would occur during the assessed life (the safety index equal to 2.0). For non-redundant spans (system safety), the corresponding probability was 0.1 percent during the assessed life (the safety index equal to 3.0).

The random variables considered in this reliability based analysis of fatigue life included the uncertainty of several factors. They are truck weight distributions, traffic volume, accuracy of the stress range calculation, dynamic stress amplitude, truck superposition or bunching, member section properties, constant amplitude fatigue lives, and modeling including the use of Miner's damage accumulation law. Many of these variables exhibit considerable uncertainty. The factors of safety or safety margins must be made larger to cover larger uncertainties. Such uncertainties can be reduced using site-specific truck-traffic data and/or performing more precise stress calculations. These actions are "rewarded" in the evaluation calculation by permitting lower safety factors to be used, according to the AASHTO provisions.

2.3.1.3 Fatigue Life Evaluation Procedure

The following steps are used in the AASHTO (1990) fatigue life evaluation procedure.

1. Calculate the effective stress range for the detail being evaluated. This can be done by using a calculated stress-amplitude based on the fatigue truck model. Alternatively, the evaluator can use stress measurements at the weld detail, or adjust the fatigue truck parameters with site-specific WIM data. Depending on the selection, a different safety factor is used to cover altered uncertainty. Note that if the equivalent stress falls below a certain tabulated value for a given weld detail, then the fatigue life may be assumed as infinite. The following discussions are for those cases where the stress exceeds the so-called infinite life threshold.
2. Use an assumed truck superposition effect of 15 percent to account for situations when the truck traffic leads to frequent side-by-side events. As the volume increases, this 15 percent factor may need reevaluation.
3. Use a fixed dynamic amplitude increase 15 percent in the stress range. This value can be adjusted based on roadway roughness conditions.
4. For a girder bridge, calculate the bending moment range (maximum minus minimum).
5. For a girder bridge, distribute the bending moment to the girder being checked. Although (AASHTO 1990) allows for a rigorous procedure such as finite element analysis with a reduced safety margin, the specifica-

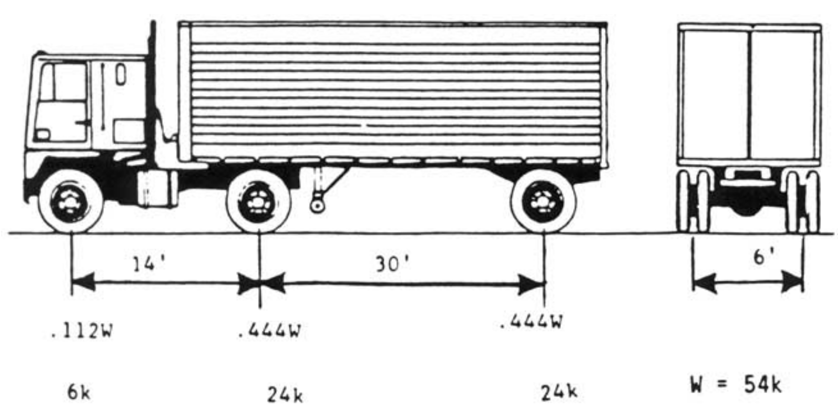


Figure 2.1. AASHTO fatigue truck model.

tion's lateral distribution factors may be used. These formulas have recently been changed in the *AASHTO LRFD Bridge Design Specifications* (1998). This change is discussed further below.

6. Use the girder moment and the section dimensions to estimate the stress range. A more liberal estimate for the section modulus may be used than allowed for in strength capacity checking.
7. Compute the total fatigue life, including safety margin cited above, from the following formula:

$$Y = \frac{fK * 10^6}{T_a C (R_s S_r)^3} \quad (2.3.1.6)$$

where Y is total life in years. K is a constant tabulated in the code (AASHTO 1990) for each category of fatigue sensitive detail. T_a is an estimated lifetime-average daily truck volume in the outer lane. (Note that the procedure given in the AASHTO specifications represents an approximation for T_a. An improvement is recommended in Chapter 3 and included in the recommended methodology in Appendix A.) C is the stress cycles per truck passage provided in the specification. R_s is a reliability or safety factor. It adjusts for the target safety index cited above for redundant and non-redundant spans, and also for the reduction of uncertainties resulting from acquisition of site-specific data. Factor f is taken as 1.0 for calculating the safe life and 2.0 for calculating the mean life.

The remaining mean life is obtained by subtracting the current age from the total life Y in Eq. 2.3.1.6. It is also possible using provisions in the AASHTO guide specifications (1990) to adjust for different fatigue truck weights in different intervals of the life and in the future as well as changes in truck volume and future expected truck volume growth rates. This approach is used in the recommended methodology in Appendix A.

2.3.1.4 Interpretation of Fatigue Life Evaluation Results

It should be strongly emphasized here that the fatigue life assessment is used in two ways. One is at the design stage to size and check for a particular component. If the component does not satisfy the design requirements for fatigue life, then it must be increased in size and rechecked. The cost for increasing the member size is relatively small, if done at the design stage. For example, *TRB Special Report 225* (1990a) reported that increases in design load from HS-20 to HS-30 might involve an average bridge cost increase of only several percent.

The other way of using the fatigue life assessment is to evaluate existing bridges. The life has to be checked with the existing sizes of the members in the span. If the member does not satisfy the check, it means that the calculated safety life with required safety margin is inadequate. Increasing the size or resistance of a component by repair or rehabilitation can be expensive for existing spans. However, a calculated inadequate fatigue life does not mean that failure is imminent because safety is checked with a target reliability goal as discussed above. Because of the considerable uncertainties in fatigue life prediction (Moses et al. 1987), AASHTO (1990) gives several options for this situation. These include (a) calculating the fatigue life more accurately using site-specific data along with the procedures provided; (b) instituting more frequent field inspections guided by the results of the fatigue life calculations; (c) repairing the bridge; and (d) restricting load levels on the bridge. In other words, many spans can remain in service despite the fact that they do not pass an initial test of a calculated remaining fatigue life. As a matter of fact, there are many spans in service that have redundant members performing satisfactorily for years despite revealing the presence of fatigue cracks. State agencies do have such experience, according to the received survey responses.

The fatigue stress checks discussed so far are for main members considering design loads. There is another category of stress related to distortion of members, especially, at ends of connection plates. It causes very high stress concentration leading to fatigue failure. These events are difficult to predict. Although these events are not covered in the conventional design and evaluation procedures, it is also true for such distortion cracking that the hot spot stress is proportional to vehicle weights.

2.3.1.5 Latest Developments in Relevant Areas

The fatigue life assessment provisions discussed above are now well accepted in both design and evaluation of steel bridges. This section discusses developments in several aspects of the fatigue evaluation procedures that have arisen since AASHTO 1990 was adopted. They are: (a) fatigue truck model, (b) truck volume, (c) bridge analysis, and (d) fatigue crack modeling. These developments may have an influence on applying the recommended methodology for fatigue assessment.

2.3.1.5a Fatigue Truck Model

The distribution of truck weights among various classes and their dependence on changes in truck weight limits is of interest in this study. As seen above, Moses et al. (1987) assumed that fatigue damage could be lumped using the 3-S2 tractor–trailer class with a fixed wheel base and axle weight distribution, as shown in Fig. 2.1. It would appear that for some sites and bridge spans, this assumption may no longer be adequate, especially for some scenarios of truck configuration resulting from a truck weight limit change. Truck data from various

WIM stations will need to be examined to determine how much influence separating trucks into individual classes and doing an individual damage sum will cause to the variation in fatigue life estimates. This influence may or may not warrant a change in the fatigue truck model. This issue is investigated further in Section 3.3 for a specific scenario of weight limit change.

It should be noted that a 10 percent increase in effective stress or effective truck weight causes about a 33 percent increase in fatigue damage. When considering an existing span, the remaining life is even further influenced. For example, if a span is 50 years old and the calculated total life is 70 years, then the remaining life is 20 years. If however, the effective stress is assumed to be 10 percent higher, then the calculated total life is only 52.5 years and the remaining life is only 2.5 years. Of course these adjustments in stress must be made in increments depending on when the change in traffic has occurred.

2.3.1.5b Bridge Stress Analysis

Several changes have occurred recently in stress calculation procedures for bridges that affect the calculation of fatigue lives. This discussion concerns the calculation of stress in girder bridge spans being relevant to most steel bridges. In the AASHTO specifications (AASHTO 1996), the lateral distribution factor for steel girders supporting a concrete deck uses a simple formula, namely, S over 14, where S is the spacing of the girders in feet. This formula is intended to distribute the truck load to individual parallel girders for the worst condition for design, not necessarily appropriate for fatigue evaluation.

The AASHTO guide specifications (1990) modified this formula and made the lateral loading formula equal to S over D, where D ranged from 17 (for a short span bridge of 30 ft) to 23 (for a long span exceeding 120 ft). That is, the longer the span, the more equal sharing of the truck load takes place among the girders. For example, D equals 20 for a 60-ft span. For an 8-ft girder spacing (S = 8), the lateral distribution factor would be 8 over 20 or 0.400, compared with S/14 = 8/14 = 0.571. The former is only 70 percent of the latter. Using the accordingly reduced stress range, the calculated fatigue life could increase almost three times compared to using S/14 in the AASHTO design specifications (1996).

Since the publication of the *AASHTO Guide Specifications for Fatigue Evaluation of Steel Bridges* (1990), a new development has taken place regarding the lateral distribution factor. The so-called Imbsen formula was developed, fitted from results of rigorous finite element grillage analyses of bridge spans. Table 2.1 compares these distribution factors for several typical bridges, including the lateral distribution formula for design, the new Imbsen formula, the results of a rigorous grillage model, and that in AASHTO's specifications (AASHTO 1990). Bala Sivakumar with A.G.Lichtenstein & Associates provided these values. This table shows that the Imbsen distribution formula is higher than those from the rigorous grillage analysis. The AASHTO guide specifications (1990) method predicted the load distribution fairly well. Note that for fatigue estimates, a 10 percent difference in stress calculation leads to a 33 percent change in life. Further, when the remaining life of an existing bridge is concerned, the difference could be even larger, depending on the used (current) life.

2.3.1.5c Fatigue Crack Modeling

The fatigue damage accumulation described above uses a simplified Miner damage accumulation law and an assumption that all stress occurrences contribute to the damage. This assumption was based on the data available at the time. Some researchers have called for a further study to verify the behavior of variable amplitude stress cycles in steel bridges. No results can be used at this time. Thus, the AASHTO guide specifications (1990) procedure is used in the recommended methodology in Appendix A. Possible future results from research in this direction may be of interest to bridge engineers and researchers, in relation to future improvement of the methodology.

TABLE 2.1 Comparison of girder distribution factors (single lane) (data provided by B.Sivakumar of AGLichtenstein & Associates)

Span –ft (Spacing-ft)	Imbsen Formula*	AASHTO 1996 Design Specs: S/14	Grillage Method	AASHTO 1990 Guide Specs S/D
1. 65' Steel stringer (S=7.33)	0.383	0.524	0.35	0.360
2. 80'Steel Composite (S=8.0)	0.355	0.571	0.35	0.375
3. 125'Steel Composite (S=7.83)	0.365	0.559	0.30	0.340
4. 161'Steel Composite (S=13)	0.448	0.929	NA**	0.353

* One lane distribution truck load per girder based on new Imbsen formulas without 1.2 multiple presence factor.
** NA = Not available

2.3.2 RC Deck Fatigue

RC bridge decks are commonly used in highway bridges in the United States. They provide the driving surface and also transfer wheel loads to the supporting beams or stringers. It has been observed that these decks deteriorate at a faster (sometimes much faster) rate than the supporting beams or stringers. There are many factors contributing to this deterioration rate. Truck load is one of the major ones. This can be seen in the following data.

2.3.2.1 Effect of Truck Loads to RC Bridge Deck Deterioration

Table 2.2 shows a comparison of condition history for two RC decks in California. One of them (Bridge 33-198 on I-880) permits all trucks, the other (Bridge 33-324 on I-580) allows only trucks with GVW below 5 tons. These two routes are parallel to each other, as shown in Fig. 2.2. Essentially, I-580 is an alternative route to I-880 for lighter vehicles. As shown, the environmental conditions for these two bridges are virtually the same. No deicing chemicals have been used on these two routes shown. Both bridges have continuous spans of reinforced concrete box girders with an RC deck. The deck on Bridge 33-198 is 0.191 m (7.5 in.) thick and that on 33-324 is 0.165 m (6.5 in.) thick, according to the design drawings. The reinforcement is virtually the same in these two decks.

The condition histories in Table 2.2 were directly taken from respective bridge inspection reports without editing. One exception is that two tables of potholes in the 1982 and 1983 inspection reports for Bridge 33-198 are summarized for clarity and saving space. Bridge 33-198 had a significant repair for potholes approximately at the age of 29 years in 1986. In contrast, Bridge 33-324 did not need repair at a similar age. More importantly, the former has shown more potholes since that repair, and the latter still does not need such repairs although some cracking is observed. Note that the 33-198 deck is about 15 percent thicker than the 33-324 deck. This provides significantly higher shear strength to resist wheel loads.

It is concluded that the difference in the two decks' condition was due to the different truck loads carried, also shown by truck traffic in Table 2.2. These two routes have had similar total annual average daily traffic (AADTs) over these years, but very much different truck traffic. At the same time, Bridge 33-198 has carried 15 to 25 times more trucks, which are much heavier than those carried by Bridge 33-324.

Moreover, Table 2.3 shows a similar comparison of another pair of decks, whose locations are also given in Fig. 2.2. Both are RC slab bridges with a much thicker slab. Bridge 33-273 carries I-880 allowing all trucks, and Bridge 33-317 carries I-580 allowing lighter trucks up to 5 tons. Bridge 33-273 has a slab thickness of 0.343 m (1ft and 1.5 in.), and Bridge 33-317's slab thickness is 0.686 m (2 ft and 3 in.). For both bridges, Table 2.3 shows no potholes at all, except some cracks. Note that the truck traffic on Bridge 33-273 has been 9 to 15 times more than that on Bridge 33-317 for any given year. The total traffic has been similar between these two bridges. This comparison in Table 2.3 indicates that environmental factors (such as water presence or exposure to salt) may not necessarily play a driving role in RC deck deterioration. It is the load versus the strength (for fatigue) that is the major factor for RC deck deterioration, at least in areas where no or little salt is used.

It should be noted that for many other areas in the country, a large amount of deicing chemicals is used for winter safety maintenance. RC bridge deck deterioration has been found to be strongly correlated with steel reinforcement corrosion caused by deicing chemicals. Weyers et al. (1993, 1994) provided several methods to predict the service life of an RC deck as a result of rebar corrosion, as part of the products of the SHRP research program. This factor should also be covered in estimating the service life of an RC bridge deck.

2.3.2.2 Fatigue Mechanism of RC Bridge Decks Under Wheel Loads

Fatigue of RC bridge decks due to truck loads has attracted research attention for over two decades. Until recently, this topic was investigated using a stationary load with varying magnitude, referred to as a stationary pulsating load. Such loading setup was used perhaps because steel fatigue testing was typically done this way. During the late 1980s and early 1990s, Professor Shigeyuki Matsui at Osaka University in Japan (Matsui and Muti 1992, Matsui 1991) and Professor P. Perdikaris at Case Western Reserve University (Perdikaris et al. 1993) led independent groups studying this subject using a simulated moving wheel load on deck models. Both groups found that a moving load is much more damaging than a stationary pulsating load. Resulting cracking very closely resembled that observed in real bridge decks in service. Both show a "grid-like" pattern, with the cracks following reinforcement bars, while the pulsating load testing causes a radial or "fan-like" pattern of cracks.

These test results also explained the mechanism of RC deck damage being that of shear fatigue. As discussed earlier, fatigue damage originates from discontinuities such as very small cracks. Two likely causes of visible cracks are concrete shrinkage (Krauss and Rogalla 1996) and truck overloads (Kostem 1978, Fu et al. 1992, 1994). Unfortunately, such cracks cannot be eliminated using today's technology. They are considered the triggers of fatigue damage accumulation in RC decks. Cracks then grow because of load cycles and cause further deterioration.

As shown in Fig. 2.3, when a transverse crack is present, the shear force induced by the truck wheel introduces stress concentration at the crack tip. This stress concentration becomes the driving factor for fatigue damage accumulation. This

TABLE 2.2 RC bridge deck performance comparison (RC box girder bridges)

Age	Year	Total AADT	Truck AADT	Bridge 33-198 on I-880 Allowing Heavy Trucks*	Year	Total AADT	Truck AADT	Bridge 33-324 on I-580 Not Allowing Heavy Trucks*
0	1958				1963			
1	1959			Good condition	1964			
2	1960				1965			
3	1961			Transverse and alligator cracking, at 1' to 3' centers, is apparent in both NB and SB	1966			Good condition
4	1962			Small spalls have developed along some of the numerous deck cracks	1967			
5	1963				1968			
6	1964				1969			Light spalling of deck cracks
7	1965				1970			
8	1966				1971			
9	1967				1972			
10	1968				1973			
11	1969				1974	107000	649	
12	1970				1975	101000	606	
13	1971			Minor spalling along some deck cracks	1976			
14	1972				1977			
15	1973				1978			
16	1974	130000	15340		1979	115000	690	
17	1975	138000	15318		1980			
18	1976				1981	122000	976	There are several deck spalls along the Abutment 10 paving notch, NB
19	1977				1982			The deck shows many transverse and pattern cracks, especially in WB lanes
20	1978				1983			
21	1979	161000	17871	In Span 10 SB lane 3 right wheel line, there is a deck pothole that is propagating	1984	129000	1032	
22	1980			Shallow PCC spalls are developing in the deck in Span 3 in the #4 lane NB. There are several significant deck spalls in the #3 lane SB in Spans 8,9, and 10	1985			
23	1981	182000	15470		1986			
24	1982			17 potholes in 7 different spans of SB & NB	1987	164000	1312	
25	1983			4 more potholes in 4 different spans of SB & NB	1988			
26	1984	193000	17400		1989			
27	1985			Potholes continue to grow in number and size	1990	170000	1360	The deck cracks are now medium to large in width, and transverse or pattern-like, in WB & EB. There is a small spall, approximately 3"* 6"*1" depth, near Bent 8 in EB Lane 4
28	1986				1991			
29	1987	224000	30464	All potholes have been repaired	1992	170000	1360	
30	1988				1993			
31	1989			22 spalls in 10 different spans of SB & NB, sized from 1 to 4 sq. ft	1994			There are many large transverse cracks in the deck. The cracks occur at frequent integrals of approximately 300 mm. Numerous shallow spalls exposing transverse rebar were also observed
32	1990	162000	22032		1995	142000	1338	
33	1991			Over 50 spall along large transverse cracks which vary in size up to 3'*1'*1' deep (Urgent repair is called for).	1996			There are many medium to heavy transverse cracks in the concrete deck surface at 150 to 600mm on centers. There are also a few shallow spalls with exposed reinforcing steel in the concrete deck surface.
34	1992	162000	22[illegible]32		1997			
35	1993			The deck has many spalls along large transverse cracks, some of which have exposed rebar. There are noticeably more spalls in NB than in SB	1998			There are a few small shallow deck spalls over transverse rebar in WB Lane 4 at Bent 3. There are large transverse deck cracks with edge spalls **
36	1994				1999			
37	1995	179000	21355	The deck is in very poor condition with about 70 potholes along the many large transverse and pattern type cracks. These vary in size up to 0.5 m in diameter and generally 40 to 50 mm deep with exposed transposed reinforcement **	2000			

*Heavy Trucks=Trucks weighing more than 5 tons

** Latest inspection result

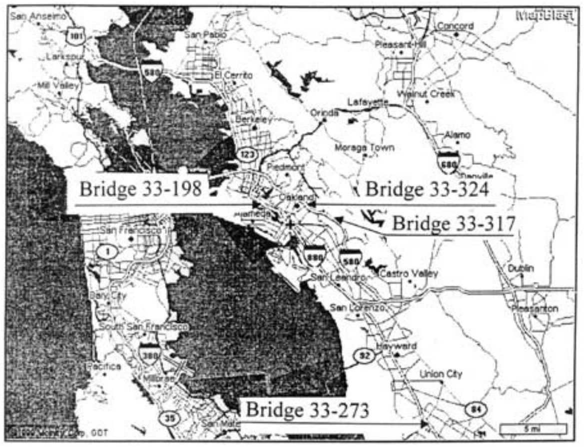


Figure 2.2. Locations of California bridges compared in Tables 2.2 and 2.3.

mechanism is similar to steel fatigue discussed above. Worse yet, the wheel load movement across a deck causes rubbing of the two concrete surfaces of the crack, widening it and accelerating deterioration. When a longitudinal crack is present due to an overload (or discontinuity due to lack of concrete consolidation), the situation is similar although the stress range is not the same. The shear force at a transverse crack changes sign when a wheel crosses the transverse crack, while the shear force at a longitudinal crack does not change sign when a wheel moves along the crack. Portland cement concrete is typically an inhomogeneous material. Thus, existing cracking may propagate in different orientations at various locations, resulting in a system of cracks. When discontinuities exist (e.g., due to lack of concrete consolidation in construction), larger cracks will eventually "connect" them to form a system of cracks. It is found that lack of consolidation in construction often occurs immediately underneath reinforcement bars, because the vibrator sometimes does not reach these areas. This results in original discontinuities. When these discontinuities are gradually connected to each other by larger cracks' propagation, the grid-like cracking pattern develops following reinforcement bars (Matsui 1991, Perdikaris et al. 1993).

Furthermore, when water (rain, snow, or even significant moisture in air) is present in the crack, the situation noticeably worsens (Matsui 1991, Kato and Goto 1984, Okada et al. 1978). In addition, it should be noted that these results (Perdikaris et al. 1993, Matsui 1991) were obtained in the laboratory using models tested over a short period of time. Some worsening factors were not adequately covered. Two major ones are discussed here. First of all, the wheel load's dynamic effects were not modeled because the test load could not reach the real wheel speed for full scale models or its scaled speeds (according to the similitude theorem) for scaled models. It is also difficult to model real bridge-surface-condition, which has been found to dictate dynamic impact. Secondly, environmental effects (salt usage, steel reinforcement corrosion, humidity variation, and freeze-thaw) on concrete crack propagation were not covered due to the short testing time. Nevertheless, these factors would worsen the fatigue performance of the decks. In other words, these test results likely overestimate the fatigue life of RC decks, because these worsening factors were not covered in testing.

2.3.2.3 Fatigue Assessment for RC Decks for Cost-Impact Estimation

For the objective of this project, a method is developed and presented in Section 3.4 for assessing fatigue accumulation in RC decks. Based on this concept, Appendix A presents the procedure for Cost-Impact Category 2 for RC deck fatigue as part of the recommended methodology. The method's basic concept is based on the experimental results obtained by the independent research groups led by Matsui (Matsui and Muti 1992, Matsui 1991) in Japan and Perdikaris (Perdikaris et al. 1993) in the United States. The recommended procedure in Appendix A to be used to estimate related cost impact is calibrated here using the U.S. practice experience and field condition. A sensitivity analysis is presented there to understand the effects of the input data on the final result. This RC deck fatigue assessment procedure is given in a format very similar to that of the AASHTO's steel fatigue assessment. Thus understanding the concept of this new procedure is expected to be relatively easy, which will help implement the recommended methodology.

2.3.3 Deficiency Due to Overstress for Existing Bridges

Highway bridges are designed for the design load at the time. Thus, their load-carrying capacity is thereby limited. When deterioration occurs, this capacity may be impaired. For example, corrosion of steel could reduce the cross section of a primary steel member and capacity of the bridge. The safe load-carrying capacity of U.S. highway bridges is currently quantified by load ratings. There are two load ratings according to current AASHO specifications (1994): the inventory rating and the operating rating. The inventory rating indicates the permissible load under a bridge safety level equivalent to that assured by design. The operating rating permits a higher load, resulting in a lower safety level as a compromise to maximize the use of the bridges, as well as to avoid high costs. Essentially, it gives the absolutely highest permissible load, with an implication to bridge structure safety. These two ratings are based on legal loads and are inventoried in the agency database as well as the database at the federal level, the National Bridge Inventory (NBI). When the truck weight limits are increased to allow heavier trucks, bridges with marginally adequate load ratings may become inadequate

TABLE 2.3 RC bridge deck performance comparison (RC slab bridges)

Age	Year	Total AADT	Truck AADT	Bridge 33-273 on I-880 Allowing Heavy Trucks*	Year	Total AADT	Truck AADT	Bridge 33-317 on I-580 Not Allowing Heavy Trucks*
0	1958				1964			
1	1959				1965			There are a few fine cracks scattered throughout the deck
2	1960				1966			
3	1961				1967			
4	1962				1968			
5	1963			There are medium to large size random deck cracks throughtout the structure. There has been minor spalling along these cracks. There has been some spalling of both paving notches. The larger spalls have been filled with sealant. There has been a minor amount of spalling along the medium size random deck cracks throught the structure. In general, the structure is in good condition	1969			
6	1964				1970			
7	1965				1971			
8	1966				1972			
9	1967				1973			
10	1968				1974	94000	549	
11	1969				1975	87000	435	
12	1970				1976			
13	1971				1977			
14	1972				1978			
15	1973				1979	100000	500	
16	1974	72000	5760		1980			
17	1975	76000	6080		1981	106000	1166	
18	1976				1982			
19	1977				1983			
20	1978				1984	113000	1130	
21	1979	90000	7200		1985			
22	1980				1986			
23	1981	112000	11088		1987	142000	1420	
24	1982				1988			
25	1983				1989			
26	1984	131000	12388		1990	160000	1440	
27	1985				1991			
28	1986				1992	160000	1440	
29	1987	152000	17328		1993			
30	1988				1994			
31	1989				1995	126000	1076	
32	1990	157000	17898		1996			A close deck inspection did not reveal any significant deficiency. It is safe to classify these cracks as shrinkage cracks.
33	1991				1997			
34	1992	149000	16986		1998			**
35	1993							
36	1994							
37	1995	158000	12041	**				

*Heavy Trucks=Trucks weighing more than 5 tons

** Latest inspection result.

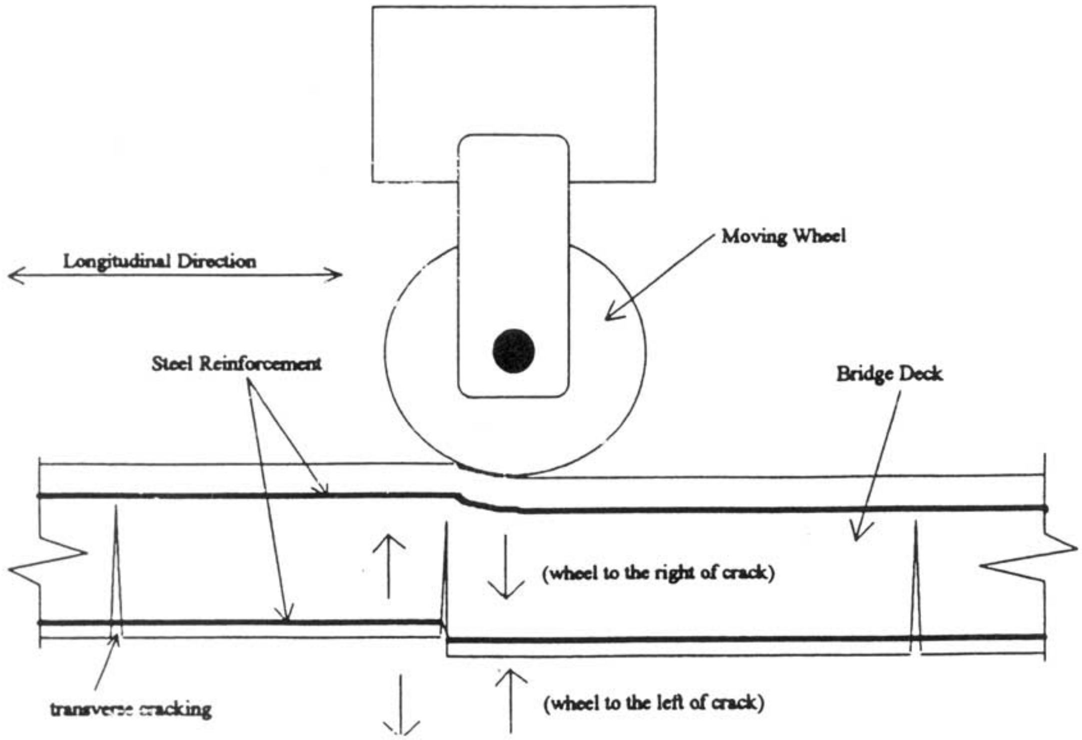


Figure 2.3. Shear fatigue of RC bridge deck under wheel loading (from Perdikaris et al. 1993).

under the new loading requirement. These bridges are then said to be deficient under the new truck weight limits. The recommended methodology includes this category of cost-impact estimation (excluding the costs for those bridges already deficient without the weight limit increase).

This category of cost-impact results from the action responding to the identified deficiency. In general, the following options may be considered as a reaction to an inadequate rating due to a truck weight limit change: (1) posting, (2) strengthening or rehabilitation, (3) replacement, and (4) a combination of (1) and (2) or (1) and (3). Depending on a number of other factors, such as special reductions in computing the FHWA-defined sufficiency rating, the agency may select any one of these options or a combination of them (e.g., post now and replace the bridge a number of years later). It should be noted that posting does not assure the same safety level as strengthening or replacement. Truck weight enforcement measures also need to be implemented, particularly when a large number of bridges need to be posted, as a result of truck weight limit change.

This part of the cost may be estimated considering the following factors: (a) how close the posting tonnage is to the operating rating; (b) how often truckers violate the weight limits in the area; and (c) severity of the consequence if the bridge is overloaded. Factor (a) is a general consideration. As pointed out in Minnesota DOT (1991), "allowing weight limits closer to the physical limits of highway facilities makes compliance more essential." According to current concepts of load rating, operating ratings should be the absolute maximum for posting. On the other hand, some states also consider the inventory rating in posting. The posted weight in these states is set between the inventory and the operating ratings. Factor (b) considers the "behavior culture" of truck operators. It represents the risk involved in this operation. This risk should be kept under control so as not to increase as a result of truck weight limit increases. Factor (c) should be included in the decision-making process as the bottom line. Note that there have been reports on bridge failure due to overloading.

Next, this section discusses the development of truck load requirement for bridge evaluation. This requirement is reflected by one (or a set of) evaluation truck(s) along with the associated load factors. For example, current AASHTO design load requirement is represented by the HS-20 load (including the lane load) as well as the live load factors specified (AASHTO 1996). Further, current AASHTO evaluation load requirement is rather shown by not only the HS-20 vehicle but also three additional rating vehicles (AASHTO 1994). These load requirements are important factors in decision making for bridge maintenance, repair, monitoring, and replacement. These load requirements are expected to change as a result of truck weight limit changes.

Usually, the incremental costs associated with increasing the load-carrying capability of new bridges are not relatively large, as shown by the default data to the recommended methodology in Data Set A-5.2.7 of Appendix A. A small

percentage of incremental cost may provide as much as 25 percent increase in truck-load-carrying capacity (e.g., from HS-20 to HS-25). On the other hand, the incremental costs for strengthening existing bridges could be substantially high. Often, replacing the bridge becomes the most economical alternative compared with strengthening. The review below deals with existing bridges. New bridges are covered later in Section 2.3.4.

2.3.3.1 Modeling Truck Loads for Bridge Evaluation

The bridge evaluation's load requirement should realistically reflect trucks' loads on the bridges. This requirement therefore is expected to be correlated with the real truck load spectra. Unlike design, the current load model for rating includes not only the HS load but also a series of vehicle models known as the AASHTO rating trucks (AASHTO 1994). These vehicles are supposed to envelope the truck weights and configurations allowed under the Federal Bridge Formula. Further, some states supplement these AASHTO rating trucks with additional vehicles to reflect the jurisdiction's weight regulations that may differ from those represented by the Federal Bridge Formula. In addition to the vehicle models used, the live load factor specified should cover uncertainty in truck load effects on the bridge spans, which has been observed to be significant.

In addition to existing guidance (AASHTO 1994), there is also an *AASHTO Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges* (1989b), which uses a load-and-resistance-factors approach to rating. It has been incorporated into a new AASHTO *Manual for Bridge Condition Evaluation* (AGLichtenstein & Associates 1999), currently under review. The live load factors in this new manual will be flexible and can be site dependent. This structure will allow consideration of site-specific information, such as truck traffic volume and bridge condition. This draft manual uses AASHTO rating trucks as nominal loads for rating. Most importantly, the concept of determining the load rating requirement is based on a probabilistic approach that explicitly covers involved uncertainty. This concept is integrated into the recommended methodology, which is summarized below.

2.3.3.2 Truck Load Requirement for Evaluation of Existing Bridges

As discussed above, two aspects need to be covered in a new truck load requirement for bridge rating. The first one is the new truck model, which is deterministic. For a scenario of truck weight limit change considered, this model should represent the practical maximum truck loads, with the GVW, axle configuration, and axle weights specified. This model may include one truck or a set of trucks, depending on the scenario under consideration. Checking bridge spans against these new trucks means that load ratings will be changed in proportion to the moment effects of the new trucks.

The second aspect of a new load requirement is the live load factor, as opposed to the truck model. Conceptually, the live load factor is to cover uncertainty involved in real truck loads. Such uncertainty could be associated with the likelihood of having side-by-side trucks representing the worst load condition, inaccuracy in the analysis using nominal values, possible violation to the weight regulation, possible future increase of loads, and so on.

Within NCHRP Project 12-46, Moses developed the following formula for determining the live load factor (AGLichtenstein & Associates 1999)

$$\gamma_L = 1.8\frac{2W^* + 1.41\,t(ADTT)\,\sigma^*}{240} \qquad (2.3.3.1)$$

where W^* is the average truck weight for the top 20 percent of the truck weight histogram (TWH). σ^* is the standard deviation of the top 20 percent of the TWH. t(ADTT) is a factor depending on the annual average daily truck (ADTT or truck AADT) at the site to quantify the likelihood of having heavy trucks side by side on the bridge. Typically t is in the range of 2.0 to 4.5 (as shown in Appendix A).

This live load factor was calibrated to give appropriate bridge reliability using the current AASHTO operating ratings as a criterion. A site with a severe loading spectra such as that used by Nowak (1992), namely a W^* value of 68 kips and a σ^* of 18 kips with a 5000 ADTT, would lead to a live load factor for rating of about 1.8. More severe load spectra (TWH) would raise this live load factor. The corresponding decrease in the rating factors would be proportional to the increase in γ_L.

In estimating the cost impact of truck weight limit change, it should be emphasized that changes to both or one of the two aspects of load requirement (the truck model and the live load factor) may be unnecessary, depending on the scenario. For example, if a new type of heavy vehicle is permitted but is expected to have only a small number of trips, its influence on selecting the live load factor would be smaller. It is because W^* and σ^* in Eq. 2.3.3.1 would remain almost unchanged. This result would also be consistent with the current concept that bridge evaluation requirements should reflect a balance between the cost and the risk of bridge failure. Namely, a small number of overweight vehicles are permissible under the current permit system. Prohibiting overstress for all bridges may not be of the best interest to the public the agencies serve, because it would be excessively costly.

2.3.4 Deficiency Due to Overstress for New Bridges

The mechanism of this cost-impact category is very similar to deficiency due to overstress in existing bridges. The

bridge design load conceptually is supposed to cover current and future truck loads for the expected life span of the bridges to be designed. When a truck weight limit change triggers truck weight spectra to change, the bridges to be designed according to the current design load may become inadequate because the current bridge design load may not cover the new loads resulting from the truck weight limit change. This cost-impact category is to cover those costs associated with possible additional costs for new bridges to meet the new design requirement as a result of the considered truck weight limit change.

2.3.4.1 Bridge Design Load

Bridge design loads have evolved over the last 50 years to reflect the heavier truck populations. Many older bridges were designed for the H-15 load, which was superseded later by the HS-20 load. In the past decade, a number of state agencies increased their design load from HS-20 to HS-25 for a variety of reasons, including heavier loads on the roads. The design load was effectively raised from HS-20 with the introduction of the LRFD HL93 load model in the *AASHTO LRFD Bridge Design Specifications* (1998). In comparing load models, it is necessary to consider the truck model (including lane load) as well as the live load factors, which further raise the effective load effects on the bridge span.

The safety factors (or live load factors) must account for uncertainties of routine loads and overloads, such as analysis approximation, future growth, etc. In older specifications, safety factors were applied to the calculated stress. For example, 0.55 is applied to the yield strength in stress in current AASHTO design specifications (1996) for the service load design method adopted decades ago. This 0.55 factor for strength is equivalent to a load factor of 1.82 (= 1/0.55) in the traditional allowable stress design which probably has controlled the vast majority of the existing bridges in the United States. Some state agencies in recent years switched their design procedures to the AASHTO load factor design, which reflects greater safety margins for vehicle loads compared to dead loads. The live load safety factor became 2.17 while the dead load factor was decreased to 1.3. Thus compared to the service load method of design, the load factor method decreases the capacity and cost for long spans and increases the total load effects for shorter spans.

The new AASHTO LRFD specification (1998) uses a load factor of 1.75 for live load and approximately 1.25 for dead load. However, the live load factor is now applied to the new HL93 model, which generally raises load effects, compared to the AASHTO load factor of 2.17 multiplied by the HS-20 load model effects. Part of the intent in changing both the load model and the live load factor is to provide cushions for future truck load changes that may occur during the expected life of the bridge, which has been accepted as 75 years in the new AASHTO code (1998). None of these cushions however has been explicitly presented.

Note that rating or evaluation of existing bridges typically uses different load requirement and safety factors than those for design of new bridges. It has been discussed in Section 2.3.3 above. This is because rating is to assess safety for a much smaller time window, typically two years between mandatory inspections. Also, the safety margins are reduced in evaluations because of the more significant cost impacts. Namely, it is relatively inexpensive to build in a wider safety margin at the time of design. The additional cost for the higher load carrying capacity is relatively low. In contrast, if an existing bridge is deficient, it is expensive to rehabilitate or replace it to create a higher capacity. Many existing spans are deficient today because of either deterioration or because they were originally designed for lower standards than are presently required.

The differences in safety margins between design and rating are also reflected by the difference between the inventory and operating ratings discussed in Section 2.3.3. Many agencies make posting and permit decisions based on the less conservative operating ratings. For example, the operating allowable stress limit is increased to 75 percent of the yield strength compared to the 55 percent level used in the service load design. Similarly, for ratings based on the load factor procedure, 2.17 is used as the live load factor design, and it is reduced to 1.3 for the operating ratings of existing bridges.

2.3.4.2 Truck Load Requirements for New Bridges

It is recognized that the AASHTO design load does not change very frequently at the national level. It is not because there have been no needs to change rather because of the lengthy process of developing and implementing the change. Taking the advantage of more WIM data accumulated over the past decade, developing a new load requirement has become relatively simpler and more quantitative. It is also observed that changing the design load at the state level has been realistic, effective, and rational. This observation justifies the practical basis for including this cost impact category for additional costs to prevent overstress in new bridges due to truck weight limit changes.

A similar consideration as that for rating bridges can be made for adjusting the load requirement for designing new bridges, following a change in truck weight limits. The aim is to maintain the target bridge reliability for new bridges (AASHTO 1998, AGLichtenstein & Associates 1999). It again will require two parts: a truck model and a new set of live load actors. The first part will involve selecting an appropriate model to cover expected new truck loads. This model could be based on the practical maximum vehicle loads under the new truck weight limits.

In addition, an adjustment in the live load factor can be introduced to respond to projected changes in the truck load spectrum as a result of the truck weight limit change considered. The formula for this live load factor would be similar

to the rating live load factor in Eq. 2.3.3.1 except that the factor "t(AADT)" must be adjusted for an expected life of 75 years and a higher reliability index needs to be met for design, as opposed to the operating rating level. The formula for the required live load factor becomes:

$$\gamma_L = 1.75 \frac{2W^* + 6.9\,\sigma^*}{265} \tag{2.3.4.1}$$

where W* and σ* have been defined above and are determined from the predicted TWH. Adjusting the live load factor as shown in this formula is a very simple way to account for the possible changes in the truck weight spectra with respect to load requirement for future bridges.

2.4 PREDICTING TRUCK WEIGHT SPECTRA CHANGES

This section reviews several algorithms from the literature for estimating the effects of changes in truck weight regulations on truck weight distributions. Based on this knowledge, a recommended method is presented in Section 3.2 as part of the recommended methodology. The recommended prediction method is intended to improve upon those algorithms reviewed in this section, with consideration to the current scenarios of truck weight limit change.

Truck weight regulations include restrictions on maximum gross vehicle weights, single-axle weights, tandem-axle weights, and weights carried on groups of two or more axles. For example, the Federal Bridge Formula (often referred to as Formula B) limits the amount of weight that can be carried on a group of two or more consecutive axles. It is based on (1) the length of the axle group and (2) the number of axles in the group. There are also some practical limitations on vehicle loading, governed by safety and operational considerations. Most common of these are practical limitations on the amount of weight that can be carried on steering axles. We speak of a truck's practical maximum gross vehicle weight (PMGVW) as the maximum weight it can carry, taking into account not only truck weight regulations but also other practical limitations.

For low-density commodities, the amount of payload that can be carried on a truck is usually limited by its cubic capacity, not by truck weight restrictions. The maximum cubic capacity of a truck is controlled by limits on truck width, height, trailer length, and number of trailers. For high-density commodities with divisible loads (e.g., sand, coal, and sugar beets), shippers will try to load their trucks as close as possible to the PMGVW. Increases or decreases in PMGVW for these vehicles due to changes in truck weight regulations will directly affect truck weight distributions for these vehicles. However, observing and estimating how weight limit changes affect truck weight distributions is complicated because changes in weight limits affect not only weight distributions but also the following factors. (1) Total mileage by loaded trucks required to transport a given amount of freight from one location to another. (2) Mileage by empty trucks returning to their home base or repositioning to pick up their next load. (Single-unit trucks such as dump trucks and concrete mixers typically operate up to half of their mileage empty. For combinations, the percentage of empty mileage is less, typically 10–30 percent.) (3) The types of truck configurations used to carry freight. (4) Competition for freight between trucks and other modes (most importantly rail). (5) The total amount of freight shipped. These factors have not been adequately covered in the earlier efforts reviewed below. They are addressed in the recommended method discussed in Chapter 3.

2.4.1 *NCHRP Report 141*

The first federal limits on vehicle weights were enacted as part of the Federal-Aid Highway Legislation of 1956, which restricted single axles to 80 kN (18 kips), tandem axles to 142 kN (32 kips); and gross vehicle weights to 326 kN (73.28 kips) on Interstate highways. That act also called upon the U.S. Secretary of Commerce to report to Congress regarding "maximum desirable dimensions and weights of vehicles operated on the federal-aid systems." In response to this legislation, the Bureau of Public Roads (later the FHWA) sponsored extensive research on the consequences of alternative size and weight limits. This research culminated in *NCHRP Report 141* (Whiteside et al. 1973).

The *NCHRP Report 141* algorithm for predicting the effects of changes in weight limits on weight distributions is based on the following assumptions:

1. Given an increase in legal weights, the empty weight of the trucks will increase to provide for the strength and durability of the vehicle in use under the heavier payloads.
2. Trucks will carry increased payloads per trip and therefore operate with increased axle and gross weights.
3. Vehicle weight distributions will change from the current legal limits to future limits as a function of the change in PMGVW of each vehicle class.

In implementing these assumptions, each weight under the current limits is adjusted by applying a multiplier to produce the weight at the new limits. The multiplier is 1.0 at the lowest observed gross weight and increases linearly until it reaches the practical maximum weight at the current limit, beyond which the factor remains constant and equals the ratio of PMGVW at the new limit to PMGVW at the current limit.

In applying the NCHRP 141 algorithm to predict the effects of increases in PMGVW, it was assumed that the number of vehicles would remain constant but that their payloads would vary. This assumption is questionable since it implicitly assumes that if PMGVW is increased, the amount of payload ton-miles carried by trucks would also increase so that there is no net change in truck VMT. While some increase

in payload ton-miles carried by trucks would be expected (since the cost of transportation is decreased by an increase in PMGVW), the implicit assumption likely overstates the size of this increase.

2.4.2 Yu and Walton (1982)

In a study for the Texas State Department of Highways and Public Transportation (TSDHPT), Yu and Walton (1982) developed some important improvements to the NCHRP 141 algorithm. They concluded that the NCHRP 141 algorithm overstated the effects of changes in truck weight limits on lightly loaded vehicles. Instead, they suggested that the multipliers should remain at 1.0 up to the 50th percentile of GVW for heavy single-unit trucks. For combinations, they suggested that the multipliers should remain at 1.0 up to the 33rd percentile of GVW. Also, the algorithm proposed by Yu and Walton for TSDHPT held truck payload constant, in contrast to the NCHRP 141 algorithm, which held the number of vehicles constant.

2.4.3 Walton, Yu, and Ng (1983)

Walton, Yu, and Ng (1983) presented an algorithm for shifting GVW cumulative distributions based on the assumptions that the ratio of average GVW to PMGVW will remain constant for each type of truck and that the variance of the GVW distribution will remain constant. Their algorithm consisted of the following three parts.

1. Determining the expected mean and variance of the GVW distribution for a truck type under the proposed legal limit, which involves the analysis of historical data and the application of the average GVW.
2. Constructing a cumulative distribution curve from a set of representative truck weight data provided.
3. Shifting the cumulative distribution curve, whereby the mean and variance of the shifted curve are within the acceptable tolerance of the parameters obtained in the first part of the algorithm.

The authors provided limited support for their assumption that the ratio of average GVW to PMGVW and the variance of the GVW distribution will remain constant. This assumption would be plausible if nearly all truck loads were controlled by the PMGVW. However, a large share of 3-S2s and 2-S1-2s are carrying partial loads or are carrying loads limited by the cubic capacity of their trailers. It is not likely that the average GVW of these vehicles will vary in proportion to PMGVW.

2.4.4 Fekpe and Clayton (1995)

Fekpe and Clayton (1995) and Fekpe et al. (1994) developed an algorithm for predicting heavy-vehicle weight distributions and pavement loadings in terms of governing weight limits and the intensity of enforcement. They group truck configurations into two broad families as follows.

> "The first family comprises configurations that are used for 'all-commodity' freight in which no one commodity or small number of commodities dominate; i.e., there is a fair mix of weight-out (dense) and cube-out (bulky) commodities. Trucks in this family are used to transport the full range of commodities in both truckload and less-than-truckload quantities. Their cumulative GVW distributions follow an extended S-shape with the probability-density distribution approaching the normal distribution. This family is typified by the tractor-semitrailer, straight trucks, and five- and six-axle A-train combinations."

> "The second family comprises truck configurations that are typically operated at GVWs very close to the limit. The probability-density distributions of such configurations have a strong positive skew. This family is dominated by multi-trailer configurations, e.g., the eight-axle B-train. Truck configurations in this family are generally used for hauling dense products (i.e., heavy weight-out commodities) in truckload quantities."

For the "all-commodity" family, Fekpe and Clayton estimated the following model for the cumulative GVW distribution of laden trucks:

$$P(x) = (23 - 1.43x + 0.022x^2)/(100 + v) \qquad \text{for } x > 35 \qquad (2.4.4.1)$$

where:

- x = operating weight expressed as a percentage of the GVW limit
- v = the percentage of vehicles exceeding the GVW limit
- $P(x)$ = the probability of trucks operating at GVW less than or equal to x

At x = 35, assumed by Fekpe and Clayton to be average tare weight, P(x) is equal to 0.

For the "weight-out" family, Fekpe and Clayton estimated the following model for the cumulative GVW distribution of laden trucks:

$$P(x) = (0.0025x - 0.07)/(100 + v) \qquad \text{for } 35 < x < 80 \qquad (2.4.4.2)$$

$$P(x) = (1.0024(x/100)^{10.35})/(100 + v) \qquad \text{for } x > 80 \qquad (2.4.4.3)$$

where x, v and P(x) are as defined above. These relationships were developed using truck weight data for laden trucks from special truck weight studies conducted in Manitoba (in 1989, 1991, and 1992); Saskatchewan (in 1990 and 1992); and Ontario (in 1991).

Fekpe and Clayton's algorithm is similar to the NCHRP 141 algorithm insofar as changes in truck weight limits significantly affect the weights of all laden trucks, not just the weights of trucks operating close to the current weight limit.

Changes in the violation rate (such as might be caused by stricter enforcement of truck weight laws) also affect the weights of all laden trucks, including those that are operating at weights substantially below current limits.

2.4.5 Summary Assessment

The algorithms discussed above provide useful insights into the effects of changes in weight regulations on truck weight distributions. However, they have some short-comings that limit their usefulness in analyzing current proposals for changing truck weight regulations:

- They predict significant weight-shifting for vehicles that are currently carrying weights substantially under current limits. No justification is given for why such shifts would be expected to occur.
- Above a certain threshold (which varies among the algorithms), all traffic is subject to weight-shifting. No attempt is made to distinguish trucks whose loadings are controlled by the weight limits under consideration from other truck traffic (e.g., trucks with partial loads, trucks with loadings controlled by cubic capacity rather than weight, trucks operating with indivisible loads under special permits).
- They do not address what happens to travel by empty trucks when truck weight regulations are changed. However, it would not be difficult to improve each of the algorithms to include consideration of empty trucks.
- They focus exclusively on weight-shifting and do not consider the possibility of shifts of freight to different truck types, shifts to or from rail, and increases or decreases in the total amount of freight shipped.

In most cases, the procedures described above were developed to analyze the effects of across-the-board changes in axle weight limits and GVW limits that would not greatly favor one type of truck configuration over another. For example, a primary focus of the first three algorithms above was a proposed increase at the time in single-axle limits (from 80 kN or 18 kips to 89 kN or 20 kips); tandem axle limits (from 142 kN or 32 kips to 151 kN or 34 kips); and gross vehicle weight (from approximately 320 kN or 72 kips to 356 kN to 80 kips). This weight limit proposal would not have been expected to cause shippers to alter the types of trucks they used.

However, many of the truck weight limit proposals under consideration today would cause shifts among vehicle configurations. A prime example is the proposed elimination of the 356-kN (80-kips) cap on GVW (without changing axle weight limits or the bridge formula). The PMGVW for most five-axle tractor-semitrailers is 356 kN (80 kips) or less, with or without the 356 kN (80 kips) limit on GVW. This is because the two sets of tandem axles are limited to 151 kN (34 kips) each (by the Federal tandem axle limits), and it is generally unlikely to place more than 53 kN (12 kips) on the steering axle of a 3S2. However, other types of configurations could operate with PMGVW over 356 kN (80 kips) if that limit were eliminated. Six-axle tractor-semitrailers could operate over 378 kN (85 kips), and some double trailer combinations could operate over 445 kN (100 kips) without violating the axle weight limits or the Federal Bridge Formula. Hence, it is likely that elimination of the 356-kN (80-kips) limit on GVW would result in large shifts of freight from five-axle tractor-semitrailers to combinations with six or more axles.

A recommend method is presented in Section A-5.1.1 of Appendix A for predicting TWH resulting from truck weight limit change. Its concept is discussed in Chapter 3 below. It offers improvements in those areas commented on above. In principle, weight limited truck traffic is subject to shifting as a result of weight limit changes. This shift could occur with or without changing truck configurations. Further, mode shifting (e.g., from and to rail) can also be included. Two examples are included in Chapter 3 for testing and illustration. The first example deals with a GVW-limit increase from 326 kN (73.28 kips) to 356 kN (80 kips) in Arkansas. Measured data and predicted results show a good agreement. The other example compares the result of WIM data from Idaho with prediction results, for a change of permit truck weight limit increase. Both examples show good agreement between the predicted and measured TWH for the purpose of this project.

2.5 SUMMARY

This chapter has presented a comprehensive review of the latest developments in areas relevant to the objective methodology for predicting bridge network costs as a result of truck weight limit change. In this review, the implications and background of these developments have been discussed with regard to what improvements could be further developed and subsequently integrated into the recommended methodology, as presented in Appendix A. It has been an aim that development of the recommended methodology should represent the state of the art and should maximize the use of available data. Chapter 3 presents the concept of the methodology, including supporting data.

CHAPTER 3

CONCEPT OF RECOMMENDED METHODOLOGY FOR ESTIMATING BRIDGE NETWORK COSTS DUE TO TRUCK WEIGHT LIMIT CHANGES

This chapter presents the concept of the recommended methodology, whose procedure is given in Appendix A. Before applying this methodology to a specific scenario of truck weight limit change, a planning period, PP, in years needs to be determined by the user. This period defines the time span during which the cost impact will be considered effective. This period is recommended to be consistent with the agency's planning period, so that parameters for projecting to the future would be readily available. These parameters may include discount rate, traffic growth rate, and expected funding levels. A 20-year period may be used as the default value for PP, if more specific information is not available to help select a more realistic period.

As mentioned above, four cost-impact categories are covered in the methodology:

1. Fatigue of existing steel bridges,
2. Fatigue of existing RC decks,
3. Deficiency due to overstress for existing bridges, and
4. Deficiency due to overstress for new bridges.

It should be noted that there are other categories that contribute to the cost impact as a result of truck weight limit changes. One example may be fatigue failure of steel expansion joints. However, significant work would be needed to develop quantitative methods for estimating the costs that can cover a variety of situations over the nation. This amount of work has been deemed to be beyond the scope of this project. Further, these costs are believed to be relatively less significant than those covered here. Nevertheless, agencies that have specific data and models for cost impact categories other than those covered here should be encouraged to include them in application of the recommended methodology.

Accordingly, the four prioritized cost-impact categories are addressed respectively in the following sections. The concept presented is intended to be consistent with the typical practice of state transportation agencies, largely guided by the AASHTO specifications. For those areas where no clearly defined guidelines exist or current guidelines can be improved based on the latest research results, recommendations are made based on the latest knowledge in the relevant areas. An example of such a situation is the calculation of the life-averaged daily truck traffic, T_a, needed for fatigue assessment for steel bridge components. It is discussed in detail in Section 3.3.

Before these four cost-impact categories are discussed below, two general concepts need to be presented and discussed first. They are (1) the recommended data requirements for the methodology and (2) the recommended method for predicting truck-load spectra due to truck-weight-limit changes. These two concepts are relevant to all four cost-impact categories. The data requirements are very important to the implementation of the recommended methodology. They should be set to accommodate a variety of situations over the country with respect to data availability. The prediction method for truck weight spectra is part of the procedure for each cost impact category. This is because the load spectrum is the cause of cost impact.

When applying the recommended methodology in Appendix A, an upper and a lower level of data requirements are recognized, referred to as Level II and Level I respectively. These levels refer to individual cost-impact categories. Level I represents the minimum requirement, which is expected to be reachable by all state highway agencies. Level II is a highest requirement, which may be met by only a few agencies at this time but not simultaneously for all cost impact categories. With foreseeable advancement in data availability (e.g., through Virtis becoming loaded and operational), more agencies will be able to meet this level of requirement in the future. Note that these two levels also have implications to the amount of analysis effort and the level of accuracy for the result. In general, the higher level is expected to produce a more accurate result.

3.1 DATA REQUIREMENTS FOR APPLYING THE RECOMMENDED METHODOLOGY

It was recognized, prior to starting the development of the recommended methodology, that the availability of required data could be a critical issue for successful implementation of the research product. This availability varies significantly among state agencies and even further among local agencies that may be interested in using the recommended methodology as well. It is also acknowledged that this may have a

significant implication on the reliability of the application result, because more detailed data generally would permit better fidelity to the analysis and therefore more accurate results.

Based on the survey results discussed in Section 2.2 related to the variability of available data, two levels of data requirement are designed for the recommended methodology. This is to assure that the methodology at least can be used by all state highway agencies in the country. The lower data requirement level is referred to as Level I, where a set of default data has been prepared for application, in case relevant information is not readily available. The higher level is referred to as Level II, which represents a situation where data are available for all specific sites (bridges). In the software developed for the recommended methodology, selecting which level to perform analysis at is allowed for each cost impact category and for any bridge. Completely satisfying the Level II data requirement for all cost impact categories and for every bridge in the network is not realistic for an agency. On the other hand, the agency may be able to satisfy the Level II requirement for an individual cost impact category. Further, the Level I analysis requires the minimum data that all state agencies are expected to be able to provide.

The software module for the recommended methodology is also flexible enough to permit an agency to perform analysis at a level between Levels I and II. For example, for certain bridge sites, the agency may have site-specific data (such as WIM data and bridge structure details) but, for other sites, such data are not readily available. In reality, a large number of state agencies may perform analyses at such a "hybrid" level. This flexibility accommodates virtually all situations in terms of data availability.

3.2 PREDICTING CHANGES IN TWHS AND WHEEL WEIGHT HISTOGRAMS

This section presents the concept of the recommended method for predicting truck load spectra as a result of truck-weight-limit changes. This includes two load types: TWHs and wheel weight histograms (WWHs). The former is relevant to Cost-Impact Categories 1, 3, and 4 for estimating costs related to steel fatigue of existing bridges, deficiency of existing bridges, and deficiency of new bridges. The latter represents the load causing RC bridge deck fatigue. The recommended prediction method for the latter case is based on the result for the former. Thus, it is discussed later.

3.2.1 Related Definitions

In this section and thereafter, several terms are used with specific definitions. They are given here to facilitate further presentation. They are also used in the appendixes when presenting the recommended methodology and other relevant information.

Base Case refers to conditions without the proposed changes in truck weight limits.

Alternative Scenario refers to conditions with the proposed changes in truck weight limits.

Practical maximum gross vehicle weight (PMGVW) is the assumed maximum weight at which a given vehicle can operate under a given set of truck weight limits. PMGVW captures the net effect on gross weight of the various types of weight and dimension limits (e.g., gross weight limit, axle weight limits, bridge formula, length limits, etc.), as well as practical considerations such as maximum weights on steering axles.

Tare weight is the weight of a truck when it is carrying no freight.

Payload is the weight of the freight carried on a truck.

Operating weight is the total gross weight of a truck (tare plus payload), which is interchangeable with GVW.

Payload ton-miles is a measure of the amount of freight carried by trucks. It is calculated as the product of payload (in tons) and VMT. For example, 1,000 VMT by a truck with a tare weight of 20,000 pounds and an operating weight of 50,000 pounds represents 15,000 payload ton-miles (1,000 VMT and a payload of 15 tons). Using the SI system, this quantity is measured by kN-kilometers.

Empty/loaded ratio ($r_{E/L}$) specifies the amount of empty VMT associated with repositioning trucks after they have delivered their payloads. For example for construction trucks hauling dirt from one site to another and then returning to the first site empty, the empty/loaded ratio is typically 1.0. The empty/loaded ratio for trucks carrying intercity freight is much less since such trucks often carry freight in both directions.

Weight-limited traffic is VMT by trucks whose loading is assumed to be directly affected by the weight limit scenario under consideration. Much of weight-limited traffic operates at weights close to the PMGVW. However, they may not be exactly at the PMGVW because of imprecision in truck loading and weighing practices. Also, some vehicles may start out from their home base fully loaded and distribute a portion of their payload at each of several locations. Further, some vehicles, such as garbage trucks, may start out empty and increase their payloads over the course of their trip and return to their home base at the PMGVW. In defining weight-limited traffic, the researchers exclude vehicles operating under special permits that exempt them from the PMGVW. For example, all states grant exceptions to weight-limits for non-divisible loads. The weights of these vehicles will not be affected by changes in the PMGVW. They also exclude trucks

whose PMGVWs are determined by limits in other states. When a truck travels through several states with different truck weight limits, its PMGVW is the most restrictive PMGVW in all of the states in which it operates.

Load shifts occur when operators load trucks heavier or lighter in response to changes in the PMGVW of these truck types.

Truck type shifts occur when operators shift freight from one type of truck to another because of a change in their relative PMGVWs.

Exogenous shifts occur when payloads change. Examples of such shifts are (1) payload shifts from trucks to rail or vice versa and (2) economic growth.

3.2.2 Recommended Method for Predicting TWHs

A new method is presented in this section to offer improvements over the proposed methods commented in Section 2.4, to better deal with current proposed changes in truck weight limits. Note that this new method is able to deal with both cases of increase and decrease in truck weight limits for planning purposes, although virtually all observed weight-limit changes have been increases.

It needs to be emphasized that, for low-density commodities, the permissible payload is usually limited by its cubic capacity, controlled by truck size limits (including those on width, height, trailer length, and number of trailers). For high-density commodities with divisible loads (e.g., sand, coal, beets, and hay), shippers will try to load their trucks as close as possible to the PMGVW. Increases or decreases in PMGVWs for these vehicles (due to changes in truck weight regulations) will directly affect TWHs for these vehicles. However, observing and estimating how weight-limit changes affect TWHs is complicated because these changes also influence the following factors. (1) Total travel distances of loaded trucks (VMT) required to transport a given amount of payload from one location to another. (2) Travel distances by empty trucks returning to their home base or repositioning to pick up their next load. (Single-unit trucks such as dump trucks and concrete mixers typically operate up to half of their mileage empty. For combinations, the percentage of empty mileage is less, typically 10 to 30 percent (TRB 1990b). (3) The type(s) of truck configuration used to carry freight. (4) Competition for freight between trucks and other modes (most importantly rail). (5) The total amount of freight shipped. These factors have not been adequately covered in the earlier efforts reviewed above.

Changes in TWHs due to truck-weight-limit changes may be classified into the following three types of freight shifting: (1) Load shifts without changing truck types (truck configurations), referred to as truckload shift hereafter. (2) Load shifts with changing of truck configuration, referred to as truck type shift below. (3) Exogenous shifts, such as economy growth and mode shift (e.g., from and to rail) due to competition. The new method presented next specifically deals with these shifts. Testing for the proposed method is also presented using measured truck weight data spanning weight limit change in the states of Arkansas and Idaho.

This recommended method of predicting changes in TWHs assumes that a TWH for the Base Case is available for each type of vehicle (as listed in Data Set A-5.2.1 of Appendix A) except for automobiles and 4-tire light trucks. Theses two types of vehicles are not relevant to bridge structures in strength and fatigue-related issues. These assumed TWHs for the Base Case may be obtained using WIM data, possibly available with highway agencies. The FHWA VMT data for Year 2000 may be used as the default data set. A sample of it for a functional class of roads in the State of Minnesota is shown as Data Set A-5.2.1 in Appendix A. Note that this sample needs to be normalized to be a TWH. In other words, each term in that table needs to be divided by the sum of all these terms. Then the sum of the resulting terms will be 1, which qualifies the data to be a TWH. This default data set is available for 12 functional classes in each of the 50 states.

3.2.2.1 Truck Load Shift

In truckload shifting as a result of truck-weight-limit change, trucks of a given type can be loaded heavier (or lighter). This is because the Alternative Scenario's PMGVW is higher (or lower) than the Base Case PMGVW. This type of change in TWHs is expected to occur when the Alternative Scenario does not require trucks to change their configuration for carrying the new allowable loads. A typical example of truckload shift in the United States is the increase of legal GVW limit from 320 or 326 to 356 kN (72 or 73.28 to 80 kips) in the 1970s and 80s. Virtually only 5-axle (3-S2) trucks reacted to this weight-limit change and increased their payloads.

Accordingly, load-shifting will be limited within the type of vehicle. In other words, only the TWH for that type of vehicle will be subject to change (shifting). This shifting should be performed only for weight-limit-dependent truck traffic. This amount of traffic is identified using a window shown in Fig. 3.1 over the Bases Case TWH (for the impacted type of trucks) assumed to be available. Namely, the traffic that is within this window may be subject to shifting. The window is defined by five parameters: a_1, a_2, b_1, b_2, and c, which are discussed below. These parameters are referred to as window parameters.

Parameters b_1 and b_2 define a neighborhood of weight-limit-sensitive traffic, with reference to the Base Case's PMGVW. When $GVW_{BC}/PMGVW_{BC}$ is close to 1 between $1 - a_1$ and $1 + a_2$, the level of weight-limit-dependence is described by c. It indicates the percentage of the traffic that is to be changed

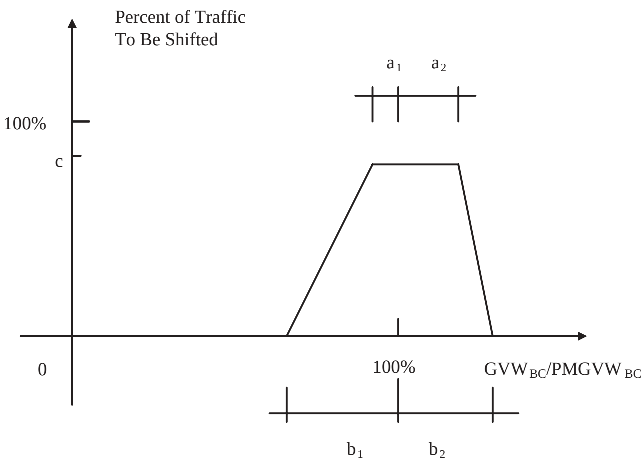


Figure 3.1. Window for truck traffic shifting.

under the Alternative Scenario. Beyond this small range to the left, the level of weight-limit-dependence is assumed to vary linearly from c at $1 - a_1$ to zero at $1 - b_1$ being the lower boundary of the neighborhood. To the right from $1 + a_2$ a similar behavior is assumed of weight-limit-dependence up to $1 + b_2$. Fig. 3.1 can also be expressed analytically as follows

$$TT'_{GVWk,BC} = \begin{cases} cTT_{GVWk,BC} \dfrac{\left(\dfrac{GVW_{k,BC}}{PMGVW_{BC}} - 1 + b_1\right)}{(b_1 - a_1)} \\ \quad \text{for: } 1 - b_1 < \dfrac{GVW_{k,BC}}{PMGVW_{BC}} < 1 - a_1 \\ cTT_{GVWk,BC} \\ \quad \text{for: } 1 - a_1 < \dfrac{GVW_{k,BC}}{PMGVW_{BC}} < 1 + a_2 \\ cTT_{GVWk,BC} \dfrac{\left(1 + b_2 - \dfrac{GVW_{k,BC}}{PMGVW_{BC}}\right)}{(b_2 - a_2)} \\ \quad \text{for: } 1 + a_2 < \dfrac{GVW_{k,BC}}{PMGVW_{BC}} < 1 + b_2 \\ 0 \quad \text{Otherwise} \end{cases} \qquad (3.2.2.1)$$

where TT_{GVWk} stands for truck traffic at weight within the k^{th} GVW interval in the TWH, and TT'_{GVWk} is the amount of traffic that is to be shifted (to be replaced by another amount of traffic) to a different GVW. The subscript BC refers to Base Case. For practical application, when the Base Case TWH is not expressed in traffic amount but in frequency, all TTs are replaced by corresponding frequencies.

After the weight-limit-dependent traffic $TT'_{GVWk,BC}$ is identified as in Eq. 3.2.2.1, the following equations will be thereto applied in modifying the TWH, as a response to the considered changes in truck weight limits:

$$GVW_{AS} = GVW_{k,BC}(PMGVW_{AS}/PMGVW_{k,BC}) \qquad (3.2.2.2a)$$

$$TT_{GVW,AS} = TT'_{GVWk,BC}(GVW_{k,BC} - TARE_{BC})/(GVW_{AS} - TARE_{AS}) \qquad (3.2.2.2b)$$

where the subscripts BC and AS refer to the Base Case and Alternative Scenario, respectively. TARE is the empty weight of truck. $TT_{GVW,AS}$ is the truck traffic at weight GVW_{AS} under the Alternative Scenario.

Eq. 3.2.2.2a indicates change in operating weight. It occurs only within the window defined in Fig. 3.1 (Eq. 3.2.2.1). Eq. 3.2.2.2b enforces the condition that the total payload travel (in kN-km) is conserved during load-shifting since the total amount of freight carried remains constant, that is,

$$TT_{GVW,AS}(GVW_{AS} - TARE_{AS}) = TT'_{GVWk,BC}(GVW_{k,BC} - TARE_{BC}) \qquad (3.2.2.3)$$

Note that when $PMGVW_{AS}$ is greater than $PMGVW_{BC}$ representing an increase in weight limit, the total amount of truck traffic will decrease since fewer trips will be required to transport the same amount of freight (payload). It also should be noted that possible payload changes are covered in Section 3.2.2.3 addressing external factors, such as economy-growth-dependent payload increase and competition-induced payload shift from or to rail.

In applying these equations, GVW_{BC} is taken at the midpoint of a weight interval falling in the window defined in Eq. 3.2.2.1 (Fig. 3.1). Consequently, the value of GVW_{AS} according to Eq. 3.2.2.2a generally will not match the midpoint of a weight interval. It is then appropriate to distribute $TT_{GVW,AS}$ between two neighboring weight intervals to achieve the desired value of GVW_{AS}, which are designated as the i th and the i + 1 th intervals, respectively. The distribution ratios p_i and p_{i+1} for the i th and the i + 1 th weight intervals are required to satisfy the following equations:

$$p_i + p_{i+1} = 1 \qquad (3.2.2.4)$$

$$p_i\, GVW_{i,AS} + p_{i+1}\, GVW_{i+1,AS} = GVW_{AS} \qquad (3.2.2.5)$$

Then the truck traffic equal to $p_i TT_{GVW,AS}$ is to be moved to the i th GVW interval and $p_{i+1}\, TT_{GVW,AS}$ to the i + 1 th interval.

For example, assume that 25 kN-increments are used for defining weight intervals. Use $PMGVW_{AS}$ = 356 kN (80 kips) and $PMGVW_{BC}$ = 326kN (73.28 kips) to express the legal weight limit change for some states in the 1970s and 80s. Typically under this GVW limit change, 3-S2 trucks could increase their weights to the new limit of 356 kN without changing their configurations. For $GVW_{k,BC}$ = 312.5 kN representing a weight range from 300 to 325 kN, GVW_{AS} is equal to 341.3 kN according to Eq. 3.2.2.2a. $TT_{340.6,AS}$ computed using Eq.3.2.2.2b will then be distributed between the weight intervals 325 to 350 kN (with midpoint $GVW_{i,AS}$ equal to 337.5 kN)

and 350 to 375 kN (with midpoint $GVW_{i+1,AS}$ equal to 362.5 kN). The distribution ratios p_i and p_{i+1} respectively are 85 percent and 15 percent, by satisfying Eqs. 3.2.2.4 and 3.2.2.5.

The following assumptions have been used for the proposed method. (A) Not all truck traffic is weight limited. For many commodities (e.g., potato chips), the cubic capacity of the truck is the limiting factor. (B) Heavier trucks excessively above $PMGVW_{BC}$ and operating under special permits may not react to weight-limit changes if other factors (e.g., the permit fee charge system) do not change. (C) The total payload traveled (in kN-km) remains the same before and after the weight limit change, i.e.,

$$\text{Payload (in kN)} \times \text{Distance of Travel (in km)} = \text{Constant} \qquad (3.2.2.6)$$

Eq. 3.2.2.6 has been expressed in Eq. 3.2.2.2b, and the distribution of this traffic over the truck-weight intervals is altered because of shifting. It is also important to note that, truck transportation is influenced by many factors. Therefore, selecting these parameters a_1, a_2, b_1, b_2, and c for the proposed method may require measured data and appropriate engineering judgment. For example, for trucks operating in multiple states, the PMGVW is generally controlled by limits in the most restrictive state. It may be different from PMGVW for trucks operating in a limited area where truck weight limits are uniform, which could dictate the selection of these parameters.

3.2.2.2 Truck-Type Shift

The same equations as Eqs. 3.2.2.1 and 3.2.2.2 used for truck load shifting are recommended to be used for truck-type shifting. However, $TT_{GVWk,BC}$, $TT'_{GVWk,BC}$, $PMGVW_{BC}$ and $TARE_{BC}$ now refer to the truck type from which traffic is shifted, and $TT_{GVW,AS}$, $PMGVW_{AS}$, and $TARE_{AS}$ refer to the truck type to which traffic is shifted.

For example, assume again 25-kN (5.6-kips) increments for weight intervals. Consider the scenario of GVW weight-limit increase from 356 kN (80 kips) as $PMGVW_{BC}$ to 431 kN (97 kips) as $PMGVW_{AS}$. The 3-S2 trucks controlled by the current weight limit 356 kN (80 kips) would need to change to 3-S3 configurations to add weight and also satisfy other requirements, such as the wheel weight limits with consideration to pavement fatigue. This would cause truck-type shift as a result of the proposed truck-weight-limit change. For $GVW_{k,BC}$ = 362.5 kN representing a weight interval between 350 and 375 kN (on 5-axles), GVW_{AS} is found to be 438.9 kN according to Eq. 3.2.2.2a and it will have to be carried by 6 axles. $TT_{439.5,AS}$ computed using Eq. 3.2.2.2b will then be distributed between two weight intervals: (1) 425 to 450 kN (with midpoint $GVW_{i,AS}$ equal to 437.5 kN) and (2) 450 to 475 kN (with midpoint $GVW_{i+1,AS}$ equal to 462.5 kN). The distribution ratios p_i and $p_{i+1,}$ respectively, are 94.5 percent for the former and 5.5 percent for the latter, by satisfying Eqs. 3.2.2.4 and 3.2.2.5. Note again that the both intervals refer to 6-axle trucks now.

3.2.2.3 Exogenous Shift

Exogenous shifts here refer to those changes to TWHs due to external factors, instead of those between weight intervals (truck load shifts) and between different truck types or configurations (truck type shifts). The influencing factors may be, for example, economic growth or competitiveness with other transportation modes (e.g., rail). Cambridge Systematics et al. (1997) and USDOT (1999) provide detailed discussions on transportation modal shifts for freight demand predictions. The guidelines presented there help in understanding relevant issues and in estimating the amount of truck traffic change.

The first step of accounting for these effects is to identify the traffic in the TWHs that is subject to exogenous shift. This can be approached in the same way as it was in Section 3.2.2.1 using a window, although the window needs to be specifically defined according to the situation. For the case of overall economic growth as a likely example, all traffic should be subject to change, unless otherwise objected. This may be readily taken into account by using a growth factor to be applied to all traffic. Equivalently, this can be done to the total traffic for bridge related analyses:

$$ADTT_{AS} = g\, ADTT_{BC} \qquad (3.2.2.7)$$

where g is the growth factor, which could be estimated based on data at the network level.

For the case of transportation modal change due to truck-weight-limit changes, it would be reasonable to use the same window in Fig. 3.1 for identifying the impacted traffic. In addition, a multiplier r can be applied to the affected traffic at weight GVW:

$$TT_{GVW,AS} = r_{GVWk} TT'_{GVWk,BC}(GVW_{k,BC} - TARE_{BC})/(GVW_{AS} - TARE_{AS}) \qquad (3.2.2.8)$$

As indicated, r_{GVWk} can be a function of operating weight GVW at the k^{th} interval. The multiplier is higher than 1.0 for traffic increase and less than 1.0 for decrease. Note that this case of exogenous shift may be likely accompanied by the other two kinds of load-shift. Thus, Eqs. 3.2.2.1 to 3.2.2.5 will be simultaneously applicable. Further, Eq. 3.2.2.7 can be viewed as a special case of Eq. 3.2.2.8 if we set g = r = constant and understand that all traffic is subject to this change.

3.2.2.4 Adjustment of Empty Truck Traffic

Empty truck traffic here refers to the traffic of trucks with no or little payload. Theoretically, this amount of traffic needs to be adjusted for each affected truck type, depending on how much empty truck traffic will be changed as a result of the above three types of load shift. Specifically, an empty-to-loaded ratio $r_{E/L}$ can be used for identifying this amount of

traffic subject to adjustment. These changes should be made to the intervals surrounding the tare weight, as follows

$$\Delta TT_{TARE-GVW,BC} = -r_{E/L}\, TT'_{GVWk,BC} \tag{3.2.2.9}$$

$$\Delta TT_{TARE-GVW,AS} = +r_{E/L}\, TT_{GVWi,AS} \tag{3.2.2.10}$$

where $TT'_{GVWk,BC}$ and $TT_{GVWi,AS}$ are, respectively, the reduced and increased traffic amounts. GVW for these two traffic amounts is different as indicated because of the weight-limit change modeled by Eq. 3.2.2.2a. Accordingly, $\Delta TT_{TARE-GVW,BC}$ and $\Delta TT_{TARE-GVW,AS}$ are the decreased and increased empty truck traffic amounts at the tare weights corresponding to the respective GVWs. When these traffic amounts are identified, their distribution to neighboring weight intervals can be done in the same way as in Eqs. 3.2.2.4 and 3.2.2.5. It is of interest to note that adjustment for empty truck traffic may become insignificant with respect to analyses for bridge strength and fatigue, depending on the magnitude of tare weight because this adjustment is concerned with the lower end of the TWH and bridge strength and fatigue are more relevant to the higher end of the TWH.

3.2.3 Testing Examples for the Recommended Method

Example 1: Effect of Legal Weight-Limit Change on TWH

The legal GVW limit in Arkansas was increased from 326 kN (73.28 kips) to 356 kN (80 kips) in 1983, when a number of states did the same to be in accordance with each other. Truck weight data for 1981 and 1986 are used here respectively as before (Base Case) and after (Alternative Scenario) situations, to be sure that the effects of this weight-limit change had fully developed, because it may take some time for truckers to be prepared for a significant change to their operation. These data were acquired at weigh stations over the State, provided by the Arkansas Highway and Transportation Department. Such data are considered reliable for weighed axles (and thus GVW as the sum of axle weights). Compared with WIM data available now but not available at the time, weigh station data may miss some overloaded trucks. Nevertheless, for the purpose of testing and illustrating application of the proposed method for predicting TWHs, these data are judged to be adequate.

Only 5-axle (mainly 3-S2) trucks are considered in this test because of the following reasons: (1) The available data include a statistically significant number of 5-axle trucks weighed, but only a few trucks of other types. (2) A vast majority of trucks are of this type, traveling in the State's (and in the entire country's) highway system. (3) This truck-group's behavior was expected to change because of the weight-limit change.

Fig. 3.2 shows two TWHs based on the measured data in 1981 and 1986. Note that the peak of heavy weights shifted from the 300–325 kN (67–73 kips) interval in 1981 to the 325–350 kN (73–79 kips) interval in 1986. It also should be noted that the 125–150 kN (28–34 kips) interval has a noticeably higher frequency in 1981 than in 1986. It appears that this was caused by the Motor Carrier Act of 1980 (TRB 1990a).

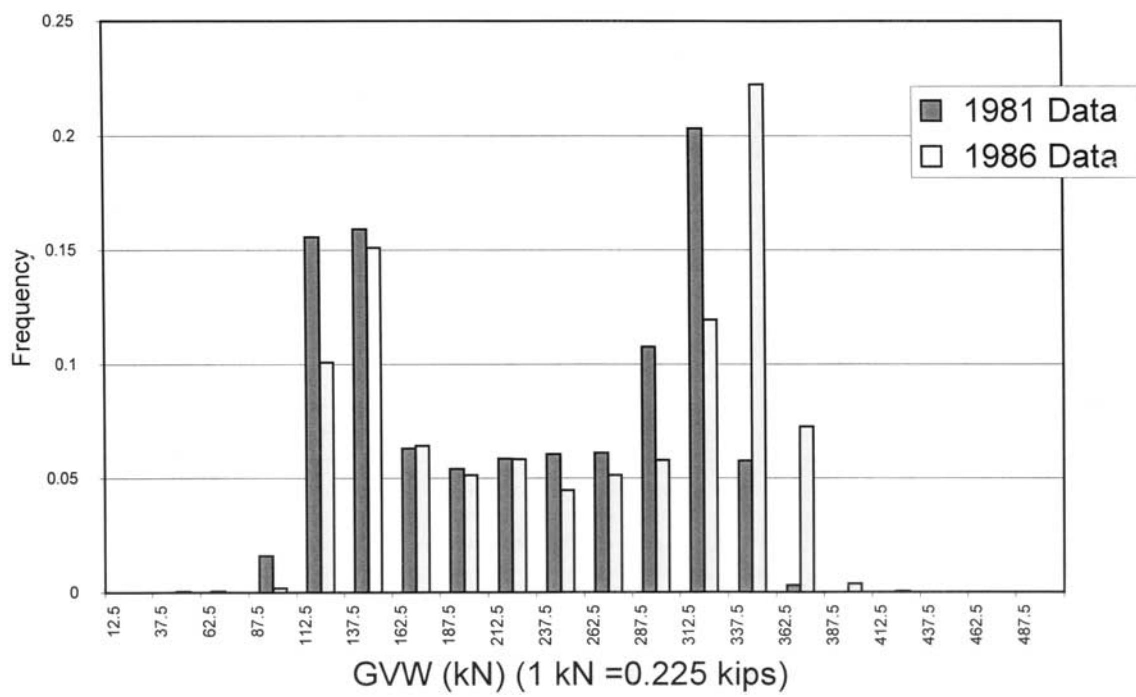


Figure 3.2. Comparison of TWHs for Arkansas: 1981 and 1986 measured (Example 1).

The Act significantly altered the trucking industry by allowing carriers to increase their service territories and the types of commodities they could transport. This deregulation greatly decreased the amount of empty backhaul traffic by allowing trucks to obtain loads for the backhaul portion of their trip, instead of returning to their operation base empty. As a result, fewer empty trips were shown in the 1986 TWH in the 125–150 kN (28-34 kips) interval.

Fig. 3.3 shows the predicted TWH as a result of the weight-limit change, by applying the proposed method using $a_1 = a_2 = 10\%$, $b_1 = b_2 = 20\%$, $c = 95\%$, and $r_{E/L} = 0.2$ in Eqs. 3.2.2.1, 3.2.2.9, and 3.2.2.10. These parameters are selected based on review of previously reported data in the literature and more recent data studied in the FHWA's truck size and weight study (USDOT 1998). Only truck-load-shift is considered because only 5-axle (mainly 3-S2) vehicles were expected to react to the limit change. The 1986 measured data are also shown in the same figure for comparison. It is seen that the heavy-truck peak's shift is clearly captured by the proposed method—from the 300–325 kN (73-79 kips) interval to the 325–350 kN (67-73 kips) interval. It is apparent that the light-truck peaks at the 100–125 kN interval in the two TWHs are noticeably different, not due to the truck-weight-limit change as commented on above.

For steel bridge fatigue evaluation, TWHs are used as a load spectrum. According to the AASHTO procedure (1990), an equivalent truck weight is calculated based on equivalence in fatigue damage (Moses et al. 1987):

$$W_{eqv} = (\Sigma f_i \, GVW_i^3)^{1/3} \quad (3.2.3.1)$$

where GVW_i is the GVW for interval i in the histogram (taken as the midpoint), and f_i is its frequency. For this test example, W_{eqv} is calculated using the predicted and measured TWHs for comparison. They agree with each other fairly well, as shown in Table 3.1.

Example 2: Effect of Overweight-Permit Weight-Limit Change on TWH

In July 1998, Idaho Transportation Department (ITD) launched a pilot project of lifting its 467-kN (105-kips) limit for annual overweight permits to 574 kN (129 kips). The vehicle configurations are still restricted according to the Bridge Formula. Under this change, these overloads are permissible only on two specific routes in the State. For all possibly impacted truck types, the new permit weight limit requires some changes to the vehicle configuration based on the Bridge

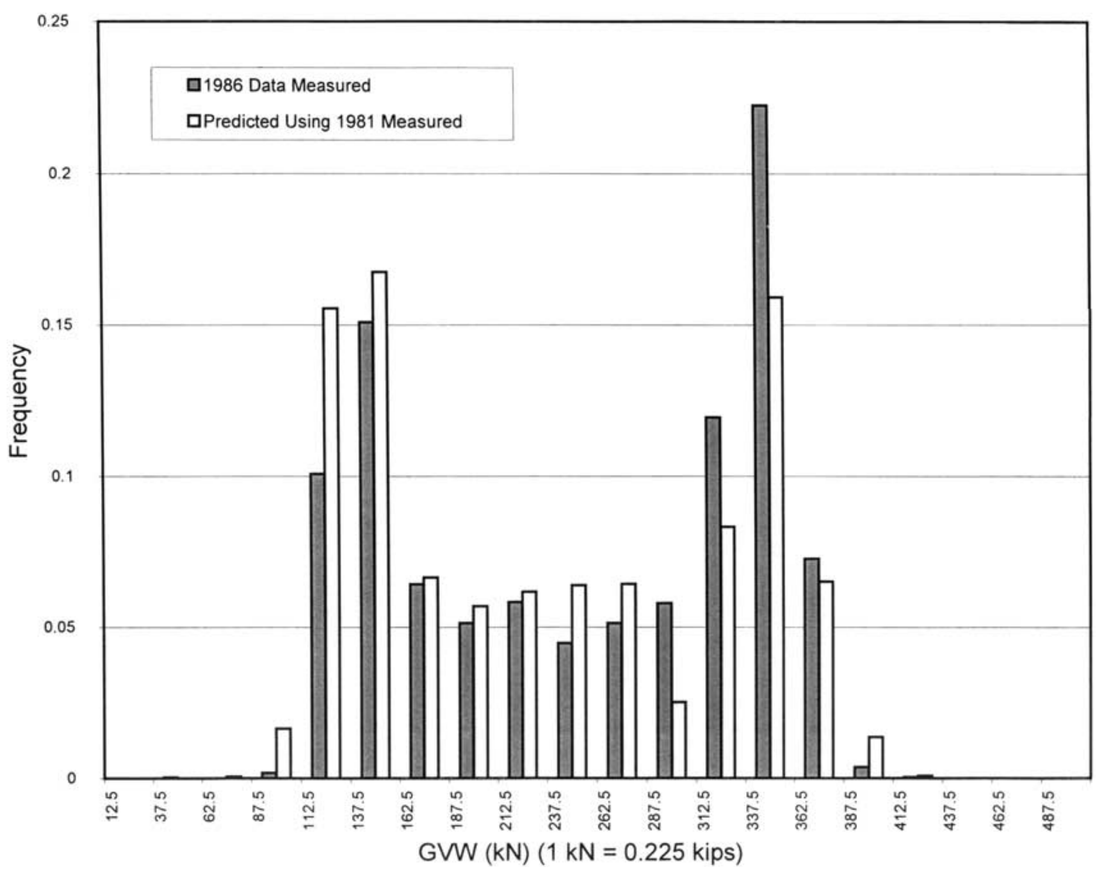


Figure 3.3. Comparison of TWHs for Arkansas: predicted vs. measured (Example 1).

TABLE 3.1 Comparison of equivalent truck weights W_{eqv} for Example 1 (Arkansas)

	(1) Base Case	(2) Alternative Scenario (Predicted using proposed method)	(3) Alternative Scenario (1986 measured)	(4) Error (2)/(3)-1
W_{eqv} in kN (kips)	246 (55.3)	258 (58.0)	273 (61.4)	-5.5%

Formula. It should be noted that typically permit weight-limit changes might affect only a small fraction of the total truck traffic.

WIM data were obtained before the weight limit change in 1997 and after the change in 1998 and 1999. The 1997 data are used as the Base Case for predicting the TWH under the new permit weight limit. Based on the registered permits and conversations with the registered trucking companies, it is assumed that only those trucks with more than 5 axles may respond to this limit change. Further, the $PMGVW_{AS}$ for 6-, 7-, 8-, and 9-axle vehicles are set respectively at 512 kN (115 kips), 529 kN (119 kips), 552 kN (124 kips), and 574 kN (129 kips), according to the Bridge Formula. The recommended default window parameters are used: $a_1 = a_2 = 10\%$; $b_1 = b_2 = 20\%$, $c = 95\%$, and $r_{E/L} = 0.2$. Table 3.2 shows comparison of the resulting equivalent truck weight for steel bridge fatigue evaluation according to the AASHTO specifications (1990). The results based on the predicted and measured TWHs are very close to each other. Note that this example includes truck-type shifts. Exogenous shifts have been considered to be negligible if any, because the new permit limit is only applicable to limited routes.

3.2.4 An Illustrative Example of Predicting TWH

An example of lifting the legal GVW limit of 80 kips to 97 kips is used here for illustration. Under this scenario, the axle weight limits will not change (i.e., single-axle weight up to 20 kips and tandem axle weight up to 34 kips). Note that this is one of the realistic scenarios that have been debated and investigated in previous studies. In those scenarios, change to axle weight limits has not been an option.

As a first step for this example, the types (configurations) of truck that will be affected need to be identified. This identification requires knowledge of truck weight and size regulations. Based on this knowledge, it is then possible to approximate the trend of change in trucking behavior. For this example, the up limit weight of 80 kips is typically loaded on 5 axles in a 3S2 configuration. However, this configuration is not allowed to carry a 97-kip GVW. At least one additional axle will need to be added to the 3S2 configuration for this new legal weight of 97 kips. Based on this, it is determined that the truck-weight-limit change considered here is expected to affect the behavior of 3S2 vehicles (with an up limit of 80 kips) and an additional axle is needed to carry the new weight limit of 97 kips. It is also seen that a structural engineering background would not be adequate in making such a determination. If the user of the methodology happens to be a structural/bridge engineer without any knowledge of commercial vehicle regulations on weight and size, assistance will be needed from those who have such a background.

The following steps are carried out to predict TWH for the alternative scenario. They use the VMT data under the Base Case as shown in Table 3.3, where each $GVW_{k,BC}$ interval is designated using its mid-interval value of GVW. Note that the raw VMT data in Table 3.3 needs to be normalized to obtain the TWH for the Base Case, as shown in Table 3.4. Its last column shows the TWH including all the truck types, with the frequencies for all $GVW_{k,BC}$ intervals summed to unity.

Step A. Quantitatively determine the PMGVW and TARE for the affected truck types under both the Base Case (BC) and the Alternative Scenario (AS).

Based on above discussion, only the 3S2T and 3S2S trucks (both with a 3S2 configuration) are considered to have a traf-

TABLE 3.2 Comparison of equivalent truck weights W_{eqv} for Example 2 (Idaho)

	(1) Base Case	(2) Alternative Scenario (Predicted using proposed method)	(3) Alternative Scenario (Average of 1998 and 1999 measured) .	(3) Error (2)/(3)-1
W_{eqv} in kN (kips)	302 (67.8)	319 (71.6)	313 (70.4)	+1.8%

TABLE 3.3 VMT data as input for predicting TWH under alternative scenario

GVWk,BC	SU3	SU4	CS3	CS4	3S2T	3S2S	CS6	CS7	CT4	CT5	CT6	DS5	DS6	DS7	DS8	TRP
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7.5	0	0	0.01	0	0	0	0	0	0.081	0	0	0	0	0	0	0
12.5	0	0	0.16	2.05	0	0	0	0	0.4185	0.015	0	0	0	0	0	0
17.5	0.8	0.048	0.91	1.37	0.0671	0.011	0.08	0	0.4861	0.177	0.033	0	3E-04	0	0	0
22.5	3.14	0.488	1.66	2.79	0.2014	0.102	0.13	0	0.7126	0.147	0.052	0.034	0.001	0.05	0	0
27.5	3.57	1.922	2.62	3	0.5369	0.116	0.31	0	0.8251	0.276	0.083	0.159	0.004	0.064	0	0
32.5	3.38	2.181	2.08	3.11	2.8861	0.122	1.41	0.012	0.8191	0.365	0.151	0.347	0.01	0.057	0	0
37.5	2.83	2.066	1.35	3.26	4.2284	0.107	1.23	0.056	0.5266	0.39	0.168	0.534	0.01	0.127	0	0
42.5	2.11	0.621	1.01	3.32	4.6983	0.093	1.18	0.164	0.3315	0.416	0.211	0.642	0.019	0.113	0.6395	0
47.5	2.63	0.859	0.71	3.26	5.7722	0.078	0.77	0.229	0.1575	0.291	0.157	0.699	0.017	0.163	0.7817	0
52.5	2.16	1.575	0.22	3.11	5.9064	0.087	0.57	0.309	0.06	0.32	0.163	0.585	0.018	0.198	0.7106	0
57.5	1.58	1.981	0.13	1.63	8.0542	0.096	0.4	0.201	0.042	0.309	0.153	0.466	0.018	0.156	1.5633	0
62.5	0.72	2.244	0.02	1.05	6.7118	0.107	0.41	0.124	0.0345	0.328	0.161	0.489	0.014	0.163	1.2791	0
67.5	0.48	2.029	0	0.32	6.1749	0.104	0.51	0.084	0.015	0.365	0.07	0.295	0.017	0.205	1.4212	0
72.5	0.05	1.504	0	0.37	4.9667	0.076	0.72	0.072	0.009	0.298	0.079	0.222	0.019	0.149	0.7817	0
77.5	0	0.788	0	0.11	4.4969	0.069	1.13	0.056	0	0.144	0.056	0.114	0.013	0.255	0.4264	0
82.5	0	0.382	0	0.05	5.5037	0.06	1.57	0.036	0	0.158	0.027	0.148	0.008	0.205	1.0659	0
87.5	0	0.239	0	0.11	7.6515	0.044	2.17	0.04	0	0.147	0.035	0.057	0.004	0.234	1.5633	0
92.5	0	0.119	0	0	5.705	0.042	2.7	0.116	0	0.151	0.01	0.017	0.002	0.156	1.137	0
97.5	0	0.048	0	0	2.6847	0.031	2.08	0.104	0	0.059	0.014	0.023	9E-04	0.191	1.7055	0
102.5	0	0	0	0	1.3424	0.007	1.2	0.12	0	0.029	0.006	0	3E-04	0.134	1.3502	0
107.5	0	0	0	0	0.6712	0.004	0.7	0.048	0	0.004	0.006	0	6E-04	0.092	0.9948	0
112.5	0	0	0	0	0	0.007	0.44	0.068	0	0	0.004	0	0	0.071	0.4974	0
117.5	0	0	0	0	0	0	0.23	0.008	0	0	0.008	0	2E-04	0.085	0.4974	0
122.5	0	0	0	0	0	0	0.09	0.008	0	0	0.002	0	0	0.057	0.6395	0
127.5	0	0	0	0	0	0	0.08	0.004	0	0	0	0	0	0.035	0.7106	0
132.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0.05	0.9238	0
137.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0.042	0.4974	0
142.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0.021	0.2132	0
147.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0.028	1.0659	0
Total	23.4	19.1	10.9	28.9	78.3	1.4	20.1	1.9	4.5	4.4	1.6	4.8	0.2	3.1	20.5	0

fic amount to shift away, and the CS6 trucks (with an 3S3 configuration) will receive an additional traffic amount. Namely, as a result of this weight-limit change, the 3S2T and 3S2S truck traffic will reduce and the CS6 truck traffic will increase. The normalized traffic amounts for these three truck types are shown in Table 3.4 as shaded. The PMGVW and TARE are determined as $PMGVW_{BC}$ = 80 kips, $TARE_{BC}$ = 30 kips, $PMGVW_{AS}$ = 97 kips, and $TARE_{AS}$ = 35 kips. Note that the user of the recommended methodology needs to provide these values for the specific Base Case and Alternative Scenario under consideration.

This determination of PMGVW and TARE is also based on knowledge about trucking behavior for these truck types. The tare weight for BC may be obtained using measurement data of tare weight of the trucks. For example, WIM data may be used to extract such information. 3S2 weight data usually show a bi-modal behavior. The peak at the lower end typically represents trucks at their tare weight. The data around this peak may be used to estimate 3S2 truck tare weights. The tare weight for AS may be estimated using the tare weight for BC with an additional amount to cover the additional axle(s). For this example, the additional axle is estimated at 5 kips. The PMGVW values in this example are taken as the weight up limits for the respective configurations (80 kips for the 3S2 configuration and 97 kips for the 3S3 configuration). This actually assumes that the weight limits can be realized with other constraints irrelevant. For example, these other restraints can be those for length and/or width. Namely, for some special types (configurations) of vehicles, the up limits of weight may not be realizable because other limits are applicable. Again these determinations require knowledge on the size and weight limits of trucks. Appropriate personnel may need to be consulted on these issues if the user happens to have no background in these areas.

Step B. Determine the window parameters for shifting, as defined in Fig. 3.1.

Note that in Fig. 3.1 c is the maximum percentage of the traffic to shift from the impacted truck types, i.e., the 3S2T and 3S2S in this example. Namely, for the traffic at a $GVW_{k,BC}$ equal to $PMGVW_{BC}$ (i.e., for $GVW_{k,BC} = PMGVW_{BC}$ = 80 kips), 95% of the traffic is predicted to become a new amount of traffic at a new GVW under the Alternative Scenario. a_1 and a_2 indicate a range (i.e., $a_1 + a_2$) where c will be applied. In

TABLE 3.4 Normalized VMT data from Table 3.3 and summed to TWH,BC as last column (impacted truck types 3S2T and 3S2S (shift from) and CS6 (shift to) are shaded)

GVWk,BC	SU3	SU4	CS3	CS4	3S2T	3S2S	CS6	CS7	CT4	CT5	CT6	DS5	DS6	DS7	DS8	TRP	TWH,BC
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7.5	0	0	3.62E-05	0	0	0	0	0	0.000363	0	0	0	0	0	0	0	0.000399
12.5	0	0	0.000724	0.009208	0	0	0	0	0.001877	6.61E-05	0	0	0	0	0	0	0.011874
17.5	0.003578	0.000214	0.004088	0.006139	0.0003009	4.983E-05	0.000363	0	0.002179	0.000793	0.000148	0	1.39E-06	0	0	0	0.017853
22.5	0.0141	0.002187	0.007453	0.012513	0.0009028	0.0004584	0.0005704	0	0.003195	0.000661	0.000234	0.000153	5.58E-06	0.000222	0	0	0.042655
27.5	0.015994	0.008619	0.011758	0.013458	0.0024074	0.0005182	0.0014	0	0.003699	0.001239	0.000373	0.000713	1.81E-05	0.000286	0	0	0.060482
32.5	0.015152	0.009777	0.009334	0.01393	0.0129396	0.0005481	0.0063257	5.4E-05	0.003672	0.001635	0.000677	0.001554	4.46E-05	0.000254	0	0	0.075898
37.5	0.01267	0.009262	0.006042	0.014638	0.018958	0.0004784	0.0054961	0.000252	0.002361	0.001751	0.000755	0.002395	4.39E-05	0.000571	0	0	0.075673
42.5	0.009449	0.002782	0.004522	0.014874	0.0210645	0.0004186	0.0052887	0.000737	0.001486	0.001866	0.000946	0.002879	8.72E-05	0.000508	0.002867	0	0.069777
47.5	0.011811	0.003852	0.003184	0.014638	0.0258792	0.0003488	0.003448	0.001025	0.000706	0.001305	0.000703	0.003134	7.67E-05	0.00073	0.003505	0	0.074345
52.5	0.009663	0.007062	0.000977	0.01393	0.026481	0.0003887	0.0025407	0.001385	0.000269	0.001437	0.000729	0.002624	7.88E-05	0.000888	0.003186	0	0.07164
57.5	0.007087	0.008882	0.000579	0.007319	0.0361105	0.0004285	0.0017888	0.000899	0.000188	0.001387	0.000686	0.002089	7.88E-05	0.000698	0.007009	0	0.07523
62.5	0.003221	0.010059	0.000109	0.004722	0.0300921	0.0004784	0.0018407	0.000558	0.000155	0.00147	0.000721	0.002191	6.28E-05	0.00073	0.005735	0	0.062143
67.5	0.002147	0.009096	0	0.001417	0.0276847	0.0004684	0.0022814	0.000378	6.73E-05	0.001635	0.000313	0.001325	7.46E-05	0.00092	0.006372	0	0.054178
72.5	0.000215	0.006741	0	0.001653	0.0222681	0.0003388	0.0032406	0.000324	4.04E-05	0.001338	0.000356	0.000994	8.51E-05	0.000666	0.003505	0	0.041764
77.5	0	0.003531	0	0.000472	0.0201617	0.0003089	0.0050813	0.000252	0	0.000644	0.000252	0.00051	5.65E-05	0.001142	0.001912	0	0.034323
82.5	0	0.001712	0	0.000236	0.0246755	0.0002691	0.0070257	0.000162	0	0.00071	0.000122	0.000662	3.49E-05	0.00092	0.004779	0	0.041308
87.5	0	0.00107	0	0.000472	0.034305	0.0001993	0.0097219	0.00018	0	0.000661	0.000156	0.000255	1.74E-05	0.001047	0.007009	0	0.055094
92.5	0	0.000535	0	0	0.0255783	0.0001893	0.012107	0.000522	0	0.000677	4.34E-05	7.64E-05	6.97E-06	0.000698	0.005098	0	0.045531
97.5	0	0.000214	0	0	0.0120368	0.0001395	0.009333	0.000468	0	0.000264	6.08E-05	0.000102	4.18E-06	0.000857	0.007646	0	0.031125
102.5	0	0	0	0	0.0060184	2.99E-05	0.0053924	0.00054	0	0.000132	2.6E-05	0	1.39E-06	0.000603	0.006053	0	0.018796
107.5	0	0	0	0	0.0030092	1.993E-05	0.0031369	0.000216	0	1.65E-05	2.6E-05	0	2.79E-06	0.000412	0.00446	0	0.0113
112.5	0	0	0	0	0	2.99E-05	0.0019703	0.000306	0	0	1.74E-05	0	0	0.000317	0.00223	0	0.004871
117.5	0	0	0	0	0	0	0.001037	3.6E-05	0	0	3.47E-05	0	6.98E-07	0.000381	0.00223	0	0.003719
122.5	0	0	0	0	0	0	0.0003889	3.6E-05	0	0	8.68E-06	0	0	0.000254	0.002867	0	0.003555
127.5	0	0	0	0	0	0	0.000363	1.8E-05	0	0	0	0	0	0.000159	0.003186	0	0.003726
132.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0.000222	0.004142	0	0.004364
137.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0.00019	0.00223	0	0.002421
142.5	0	0	0	0	0	0	0	0	0	0	0	0	0	9.52E-05	0.000956	0	0.001051
147.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0.000127	0.004779	0	0.004906
Total																	

addition, b_1 and b_2 indicate another range (i.e., $b_1 + b_2$) where there is an impact. Namely, beyond this range there will not be any impact and the traffic will remain the same.

The default values for these window parameters are used here: $c = 95\%$, $a_1 = a_2 = 10\%$, and $b_1 = b_2 = 20\%$. Note that these default values have been tested using available data, as discussed in this report. However, the user may select other values if warranted. On the other hand, without rigorous research the user will likely use these default values. Such rigorous research will require measured truck weight data before and after the implementation of weight limit change. In addition, the data need to be gathered according to appropriate design so that comparison can be made. For example, the sites selected need to be identical or compatible to avoid site-dependent issues; the time periods for data collection before and after the weight limit change also need to be compatible so that seasonable changes in truck weights will not adversely affect the comparison, etc.

Step C. Perform shifting, which includes the following substeps.

In concept, this step starts from the TWH of the truck type under the Base Case that has been identified to shift traffic away. According to the shifting window parameters (c, a_1, a_2, b_1, and b_2), each weight interval's traffic is examined to determine what fraction of it will shift away to which weight interval under the Alternative Scenario. Thus, this step ends with a TWH of the truck type receiving traffic under the Alternative Scenario and a residual TWH of the truck type shifting traffic away. The latter becomes the TWH of the same truck type under the Alternative Scenario. These steps are discussed in detail now.

(i) Identify the intervals $GVW_{k,BC}$ that will have traffic to shift away, as well as their traffic amounts. (ii) Determine the intervals $GVW_{i,AS}$ and $GVW_{i+1,AS}$ that will receive new traffic, as well as the amounts of the new traffic. Note that these traffic amounts are different from the traffic amounts shifting away because the new truck type (CS6) is allowed to carry higher payload. Tables 3.5 and 3.6 show the process of this shifting, respectively for 3S2T and 3S2S trucks. Table 3.7 shows the resulting TWH in the last column as the predicted TWH for AS. The process is further detailed next.

For Substep (i):

For example, for interval $GVW_{k,BC} = 62.5$ kips:

$GVW_{k,BC}/PMGVW_{BC} = 62.5/80 = 0.78125$ which is out of the area defined by $1 - b_1 = 0.8$ and $1 + b_2 = 1.2$. Thus no shifting will occur, or the percentage of traffic to be shifted away is zero, as shown in column $TT'_{GVWk,BC}$ in Table 3.5.

For the next interval $GVW_{k,BC} = 67.5$ kips:

$GVW_{k,BC}/PMGVW_{BC} = 67.5/80 = 0.84375$ which is within the area defined by $1 - b_1 = 0.8$ and $1 + b_2 = 1.2$. Thus a fraction of this interval's traffic will shift, as determined next.

This fraction is calculated according to Eq. 3.2.2.1 = $c(GVW_{k,BC}/PMGVW_{BC} - 1 + b_1)/(b_1 - a_1) = 0.95(.84375 - 1 + .2)(.2 - .1) = .41563$, as shown in column "window-f" in Table 3.5.

Thus, according to Eq. 3.2.2.1, $TT'_{GVWk,BC} = (.41563)TT_{GVWk,BC} = (.41563)(0.0276847) = 0.01151$, as shown in column "$TT'_{GVWk,BC}$" of Table 3.5.

The rest of the operating weight ($GVW_{k,BC}$) intervals are treated in the same way as illustrated. The results are given in the columns "window-f" and "$TT'_{GVWk,BC}$" in Table 3.5.

For Substep (ii):

The following calculation is carried out only for those $GVW_{k,BC}$ intervals that have a traffic amount to shift away.

For interval $GVW_{k,BC} = 67.5$ kips:

According to Eq. 3.2.2.2a, $GVW_{AS} = GVW_{k,BC}\,(PMGVW_{AS}/PMGVW_{BC}) = (67.5)(97)/80 = 81.844$ kips (column "GVW_{AS}" in Fig. 3.5).

According to Eq. 3.2.2.2b, $TT_{GVW,AS} = TT'_{GVWk,BC}\,(GVW_{k,BC} - TARE_{BC})/(GVW_{AS} - TARE_{AS}) = 0.01151\,(67.5 - 30)/(81.844 - 35) = 0.00921$ (column "$TT_{GVW,AS}$" in Table 3.5).

Determine the ratio of distributing $TT_{GVW,AS} = 0.00921$ to two weight intervals $GVW_{i,AS} = 77.5$ kips and $GVW_{i+1,AS} = 82.5$ kips (because $GVW_{AS} = 81.844$ is between these two values). It is done via solving Eqs. 3.2.2.4 and 3.2.2.5 for p_i and p_{i+1}. They are, respectively, 0.13125 and 0.86875 as shown in the column "pi and pi + 1 for GVW_{AS} = 81.844 Eq. 3.2.2.4 and Eq. 3.2.2.5" in Table 3.5.

Then perform the distribution: $p_i\,TT_{GVW,AS} = (0.13125)(0.00921) = 0.00121$ is obtained as shown in column "$TT_{GVW,AS}$" in Table 3.5 for interval $GVW_{AS} = 77.5$ kips. $p_{i+1}\,TT_{GVW,AS} = (0.86875)(0.00921) = 0.00800$ is obtained as shown in column "$TT_{GVW,AS}$" in Table 3.5 for interval $GVW_{AS} = 82.5$ kips.

The rest of the impacted weight intervals are treated in the same way as illustrated. Note that one weight interval under the alternative scenario $GVW_{j,AS}$ may receive traffic amounts from two adjacent weight intervals of $GVW_{k,BC}$. This is because each weight interval $GVW_{k,BC}$ shifts traffic to two weight intervals: $GVW_{i,AS}$ and $GVW_{i+1,AS}$. Also note that the same calculation is done for 3S2S trucks as shown in Table 3.6 in the same format.

TABLE 3.5 3S2T shifting calculations and results

GVWk,BC	TTGVWK,BC (column 3S2T of Table 3.4)	window-f	TT'GVWK, Eq.3.2.2.1	GVW AS Eq.3.2.2.2a	TTGVW,AS Eq.3.2.2.2b	Pi,Pi+1for GVWAS= 81.844 Eq.3.2.2.4 Eq.3.2.2.5	Pi,Pi+1for GVWAS= 87.906 Eq.3.2.2.4 Eq.3.2.2.5	Pi,Pi+1for GVWAS= 93.969 Eq.3.2.2.4 Eq.3.2.2.5	Pi,Pi+1for GVWAS= 100.031 Eq.3.2.2.4 Eq.3.2.2.5	Pi,Pi+1for GVWAS= 106.094 Eq.3.2.2.4 Eq.3.2.2.5	Pi,Pi+1for GVWAS= 112.156 Eq.3.2.2.4 Eq.3.2.2.5	TTGVWAS	GVWj,AS
2.5	0.00000												2.5
7.5	0.00000												7.5
12.5	0.00000												12.5
17.5	0.00030												17.5
22.5	0.00090												22.5
27.5	0.00241												27.5
32.5	0.01294												32.5
37.5	0.01896		-0.00230										37.5
42.5	0.02106		-0.00423									+0.000241	42.5
47.5	0.02588		-0.00383									+0.001600	47.5
52.5	0.02648		-0.00469									+0.003122	52.5
57.5	0.03611		-0.00652									+0.002455	57.5
62.5	0.03009		-0.00213									+0.002775	62.5
67.5	0.02768	0.415625	0.01151	81.844	0.00921							+0.003399	67.5
72.5	0.02227	0.95	0.02115	87.906	0.01699							+0.003908	72.5
77.5	0.02016	0.95	0.01915	93.969	0.01543	0.13125						+0.001604	77.5
82.5	0.02468	0.95	0.02344	100.031	0.01892	0.86875						0.00800	82.5
87.5	0.03430	0.95	0.03259	106.094	0.02636		0.91875					0.01561	87.5
92.5	0.02558	0.415625	0.01063	112.156	0.00861		0.08125	0.70625				0.01228	92.5
97.5	0.01204							0.29375	0.49375			0.01388	97.5
102.5	0.00602								0.50625	0.28125		0.01699	102.5
107.5	0.00301									0.71875	0.06875	0.01954	107.5
112.5	0.00000										0.93125	0.00802	112.5
117.5	0.00000												117.5
122.5	0.00000												122.5
127.5	0.00000												127.5
132.5	0.00000												132.5
137.5	0.00000												137.5
142.5	0.00000												142.5
147.5	0.00000												147.5
Total	0.35087		0.14218		0.09553							0.11463	

TABLE 3.6 3S2S shifting calculations and results

GVWk,BC	TTGVWK,BC (column 3S2S of Table 3.4)	window-f	TT'GVWK,BC Eq.3.2.2.1	GVW AS Eq.3.2.2.2a	TTGVW,AS Eq.3.2.2.2b	Pi,Pi+1for GVWAS= 81.844 Eq.3.2.2.4 Eq.3.2.2.5	Pi,Pi+1for GVWAS= 87.906 Eq.3.2.2.4 Eq.3.2.2.5	Pi,Pi+1for GVWAS= 93.969 Eq.3.2.2.4 Eq.3.2.2.5	Pi,Pi+1for GVWAS= 100.031 Eq.3.2.2.4 Eq.3.2.2.5	Pi,Pi+1for GVWAS= 106.094 Eq.3.2.2.4 Eq.3.2.2.5	Pi,Pi+1for GVWAS= 112.156 Eq.3.2.2.4 Eq.3.2.2.5	TTGVWAS	GVWj,AS
2.5	0.000E+00												2.5
7.5	0.000E+00												7.5
12.5	0.000E+00												12.5
17.5	4.983E-05												17.5
22.5	4.584E-04												22.5
27.5	5.182E-04												27.5
32.5	5.481E-04												32.5
37.5	4.784E-04		-3.893E-05										37.5
42.5	4.186E-04		-6.438E-05									+4.091E-06	42.5
47.5	3.488E-04		-5.870E-05									+2.708E-05	47.5
52.5	3.887E-04		-5.112E-05									+4.751E-05	52.5
57.5	4.285E-04		-3.787E-05									+3.759E-05	57.5
62.5	4.784E-04		-1.574E-05									+3.427E-05	62.5
67.5	4.684E-04	0.415625	1.947E-04	81.844	1.558E-04							+2.951E-05	67.5
72.5	3.388E-04	0.95	3.219E-04	87.906	2.586E-04							+2.289E-05	72.5
77.5	3.089E-04	0.95	2.935E-04	93.969	2.364E-04	0.13125						+1.187E-05	77.5
82.5	2.691E-04	0.95	2.556E-04	100.031	2.064E-04	0.86875						1.354E-04	82.5
87.5	1.993E-04	0.95	1.893E-04	106.094	1.531E-04		0.91875					2.376E-04	87.5
92.5	1.893E-04	0.415625	7.870E-05	112.156	6.375E-05		0.08125	0.70625				1.880E-04	92.5
97.5	1.395E-04							0.29375	0.49375			1.713E-04	97.5
102.5	2.990E-05								0.50625	0.28125	0	1.475E-04	102.5
107.5	1.993E-05									0.71875	0.06875	1.145E-04	107.5
112.5	2.990E-05										0.93125	5.937E-05	112.5
117.5	0.000E+00												117.5
122.5	0.000E+00												122.5
127.5	0.000E+00												127.5
132.5	0.000E+00												132.5
137.5	0.000E+00												137.5
142.5	0.000E+00												142.5
147.5	0.000E+00												147.5
Total	0.00610901		0.001601		0.001074							1.289E-03	

TABLE 3.7 Predicted TWH under alternative scenario (non-normalized)

GVWj,AS	SU3	SU4	CS3	CS4	3S2T	3S2S	CS6	CS7	CT4	CT5	CT6	DS5	DS6	DS7	DS8	TRP	TWH,AS
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7.5	0	0	0.00807	0	0	0	0	0	0.00036	0	0	0	0	0	0	0	0.0004
12.5	0	0	0.16138	2.05375	0	0	0	0	0.00188	6.6E-05	0	0	0	0	0	0	0.01187
17.5	0.79797	0.04773	0.91182	1.36916	0.0003	5E-05	0.00036	0	0.00218	0.00079	0.00015	0	1.4E-06	0	0	0	0.01785
22.5	3.14492	0.48778	1.66225	2.79099	0.0009	0.00046	0.00057	0	0.00319	0.00066	0.00023	0.00015	5.6E-06	0.00022	0	0	0.04265
27.5	3.56738	1.92241	2.62248	3.00163	0.00241	0.00052	0.0014	0	0.0037	0.00124	0.00037	0.00071	1.8E-05	0.00029	0	0	0.06048
32.5	3.37962	2.18065	2.08184	3.10695	0.01294	0.00055	0.00633	5.4E-05	0.00367	0.00164	0.00068	0.00155	4.5E-05	0.00025	0	0	0.0759
37.5	2.82593	2.06588	1.34755	3.26493	0.01896	0.00048	0.0055	0.00025	0.00236	0.00175	0.00076	0.00239	4.4E-05	0.00057	0	0	0.07567
42.5	2.10747	0.62055	1.00865	3.31759	0.02106	0.00042	0.00529	0.00074	0.00149	0.00187	0.00095	0.00288	8.7E-05	0.00051	0.00287	0	0.06978
47.5	2.63434	0.85922	0.71009	3.26493	0.02588	0.00035	0.00345	0.00103	0.00071	0.0013	0.0007	0.00313	7.7E-05	0.00073	0.0035	0	0.07435
52.5	2.15537	1.57523	0.21787	3.10695	0.02648	0.00039	0.00254	0.00138	0.00027	0.00144	0.00073	0.00262	7.9E-05	0.00089	0.00319	0	0.07164
57.5	1.5806	1.98097	0.12911	1.63247	0.03611	0.00043	0.00179	0.0009	0.00019	0.00139	0.00069	0.00209	7.9E-05	0.0007	0.00701	0	0.07523
62.5	0.71846	2.24351	0.02421	1.0532	0.03009	0.00048	0.00184	0.00056	0.00015	0.00147	0.00072	0.00219	6.3E-05	0.00073	0.00573	0	0.06214
67.5	0.47897	2.0287	0	0.31596	0.01618	0.00027	0.00228	0.00038	6.7E-05	0.00164	0.00031	0.00132	7.5E-05	0.00092	0.00637	0	0.04248
72.5	0.0479	1.50363	0	0.36862	0.00111	1.7E-05	0.00324	0.00032	4E-05	0.00134	0.00036	0.00099	8.5E-05	0.00067	0.0035	0	0.02029
77.5	0	0.78762	0	0.10532	0.00101	1.5E-05	0.00631	0.00025	0	0.00064	0.00025	0.00051	5.6E-05	0.00114	0.00191	0	0.01611
82.5	0	0.38187	0	0.05266	0.00123	1.3E-05	0.01516	0.00016	0	0.00071	0.00012	0.00066	3.5E-05	0.00092	0.00478	0	0.02575
87.5	0	0.23867	0	0.10532	0.00172	1E-05	0.02557	0.00018	0	0.00066	0.00016	0.00025	1.7E-05	0.00105	0.00701	0	0.03817
92.5	0	0.11934	0	0	0.01495	0.00011	0.02457	0.00052	0	0.00068	4.3E-05	7.6E-05	7E-06	0.0007	0.0051	0	0.04729
97.5	0	0.04773	0	0	0.01204	0.00014	0.02338	0.00047	0	0.00026	6.1E-05	0.0001	4.2E-06	0.00086	0.00765	0	0.04517
102.5	0	0	0	0	0.00602	3E-05	0.02253	0.00054	0	0.00013	2.6E-05	0	1.4E-06	0.0006	0.00605	0	0.03594
107.5	0	0	0	0	0.00301	2E-05	0.02279	0.00022	0	1.7E-05	2.6E-05	0	2.8E-06	0.00041	0.00446	0	0.03095
112.5	0	0	0	0	0	3E-05	0.01005	0.00031	0	0	1.7E-05	0	0	0.00032	0.00223	0	0.01295
117.5	0	0	0	0	0	0	0.00104	3.6E-05	0	0	3.5E-05	0	7E-07	0.00038	0.00223	0	0.00372
122.5	0	0	0	0	0	0	0.00039	3.6E-05	0	0	8.7E-06	0	0	0.00025	0.00287	0	0.00355
127.5	0	0	0	0	0	0	0.00036	1.8E-05	0	0	0	0	0	0.00016	0.00319	0	0.00373
132.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0.00022	0.00414	0	0.00436
137.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0.00019	0.00223	0	0.00242
142.5	0	0	0	0	0	0	0	0	0	0	0	0	0	9.5E-05	0.00096	0	0.00105
147.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0.00013	0.00478	0	0.00491
Total																	0.97679

Note: Traffic amount reduction = 1 - 0.976791 = 0.023209

Step D. Account for effects of tare weight changes.

The default ratio of empty to loaded trips is used for this example: $r_{E/L}$ = 0.2. It characterizes the amounts of traffic used to deliver payload and to return to the operation base with no payload. The default value is based on data collected in previous studies. The user may change this value if data are available to support such a change.

For weight interval $GVW_{k,BC}$ = 32.5 kips, the empty truck traffic will not change, as shown in Table 3.5 in column "$TT'_{GVWk,BC}$". This is because the lowest impacted interval is 67.5 kips. This leads to the lowest-impacted empty traffic interval being 67.5 – 30 = 37.5 kips using the selected tare weight of 3S2T of 30 kips. Accordingly, there will be no increase of empty traffic to the CS6 trucks, as shown in column "$TT_{GVW,AS}$" of Table 3.5.

For weight interval $GVW_{k,BC}$ = 37.5 kips, the empty truck traffic is determined as $\Delta TT_{TARE-GVW,BC} = -r_{E/L}\ TT'_{VWk,BC}$ = (0.2)(0.01151) = –0.00230, according to Eq. 3.2.2.9. This value is shown in column "$TT'_{GVWk,BC}$ Eq. 3.2.2.1" of Table 3.5 with a negative sign to distinguish it from those traffic amounts for loaded trips. Similarly, the increased empty truck traffic is calculated as $DTT_{TARE-GVW,AS} = +r_{E/L}\ TT_{GVWi,AS}$ = +(0.2)(0.0012) = +0.000241, shown in column "$TT_{GVW,AS}$" of Table 3.5.

For the rest of impacted weight intervals from 42.5 to 62.5 kips, the same calculations are repeated as shown in columns "$TT'_{GVWk,BC}$ Eq.3.2.2.1" and "$TT_{GVW,AS}$" of Table 3.5. Note also that interval 77.5 kips has two additional amounts. The first one (0.001604) came from the empty truck traffic due to the weight interval 112.5 kips ((0.001604 = (0.2)(0.00802)). The second amount (0.0012) came from the loaded traffic increase discussed above.

The calculation for the truck type 3S2S is given in Table 3.6 in the same format.

Step E. Summarize the entries to the TWH for Alternative Scenario.

The final resulting TWH is given in Table 3.7 in the last column "TWH_{AS}". It was summed using the normalized traffic amounts in Table 3.4, except those for truck types 3S2T, 3S2S, and CS6 that are impacted upon. For truck types 3S2T and 3S2S, traffic shifts away to type CS6. Columns "$TT'_{GVWk,BC}$" in Tables 3.5 and 3.6 show the decreased amounts, respectively, for 3S2T and 3S2S. Columns "$TT_{GVW,AS}$" in these two figures give the increased traffic amounts to truck type CS6. Table 3.7 gives the final results of these calculations.

Comparison of Tables 3.4 and 3.7 shows the impact of the considered Alternative Scenario on TWH. Also note that the TWH for the Alternative Scenario in Table 3.7 has not been normalized as the sum of the frequencies is not unity. The summed traffic amount 0.9768 has a difference from unity. This difference, 0.0232, is the traffic amount reduction as a result of weight limit change, because CS6 is allowed to carry a higher payload for each trip. The TWH needs to be normalized to perform other analyses that require a normalized TWH, such as the analysis to find the equivalent weight for steel fatigue life prediction.

3.2.5 Recommended Method for Predicting WWHs

For assessing RC deck fatigue, truck wheel-weight distributions are needed to estimate the effects of changes in truck weight limits. Also, although outside the scope of this project, wheel weight distributions are needed to estimate pavement impacts.

It is assumed that there is a correlation relationship between the wheel weights and the GVW. Accordingly, the concept of the recommended method is to estimate the wheel weights based on GVW. This assumption is particularly valid for trucks loaded to the limits, which is dominant to RC deck fatigue. When a TWH is available, possibly obtained using the method recommended above, the wheel weights can be estimated using the following empirical relationships:

$$\begin{aligned}\text{Wheel Weight} &= 0.5\ \text{Mean Axle Weight} + \text{Weight Residual}\\ &= 0.5\ (e + f * \text{Gross Vehicle Weight in kips})\\ &\quad \text{in kips} + X\end{aligned} \tag{3.2.5.1}$$

where e and f are regression coefficients. They form the first part at the right hand side of this equation expressing the mean wheel weight. X is a correction parameter to cover extreme wheel loads away from the mean loads. These relationships have been established for all individual axles of a variety of trucks, using WIM data from California. They are shown as Data Set A-5.2.2 in Appendix A, as the default database for e and f. Note that for each configuration, coefficients e sum to zero and coefficients f sum to one. This condition guarantees that the sum of the axle weights is equal to the gross weight. It is also recommended that agencies use their own WIM data to obtain those coefficients for typical truck types within the jurisdiction.

To demonstrate the application of the equations, a conventional 5-axle tractor–semi-trailer carrying 77,000 pounds would have 10,760 pounds on its first axle (the steering axle). Data set A-5.2.2 gives e = 7.603 and f = 0.041. Thus, the first axle's average weight is calculated here as 1,000 pounds * (7.603 + 0.041 * 77 (kips)) = 10,760 pounds = 47,860 kN.

It is noted, again, that the extremely high and extremely low wheel weights are not covered in the mean or average weight obtained using that regression relationship in the first part in Eq. 3.2.5.1. This is due to the nature of regression for predicting the conditional mean of a function (i.e., wheel weight here) of the independent variable (i.e., GVW here). As discussed in Section 2.3.2, RC deck fatigue is greatly influenced by the highest wheel loads. Thus, X is used in Eq. 3.2.5.1 to cover

this additional amount to be added to the average wheel weight.

Based on WIM data provided by the Idaho Transportation Department (ITD), this additional amount, or residual weight, is modeled here using a truncated skewed double exponential distribution (Benjamin and Cornell 1970, Johnson et al. 1994). The truncated probability density function $f'_X(x,\lambda)$ is expressed as follows.

$$f'_X(x,\lambda) = f_X(x,\lambda)/A \qquad \text{where} \quad \lambda > 0 \tag{3.2.5.2}$$

where X is a random variable to model the residual wheel weight. λ is its skew factor. A is the area of the skewed double exponential probability density function $f_X(x,\lambda)$ after truncation of the part for $x > x_0$. x_0 represents the maximum wheel weight on bridges. It is usually not the same as the legal maximum wheel load. It depends on the degree of compliance of truckers to wheel weight limits and effectiveness of enforcement. Using the WIM data from Idaho, x_0 is found to be at 18 kips.

The skewed double exponential probability density function $f_X(x,\lambda)$ is defined as follows

$$f_X(x,\lambda) = 2F_X(\lambda x)f_X(x) \qquad \text{where} \quad \lambda > 0 \tag{3.2.5.3}$$

and

$$f_X(x) = \frac{1}{2\beta}\exp\left[-\frac{|x-\mu|}{\beta}\right] \tag{3.2.5.4}$$

$$F_X(x) = \begin{cases} \frac{1}{2}\exp\left[\frac{(x-\mu)}{\beta}\right] & (x-\mu) < 0 \\ 1-\frac{1}{2}\exp\left[-\frac{(x-\mu)}{\beta}\right] & (x-\mu) \geq 0 \end{cases} \tag{3.2.5.5}$$

where μ is the mean value of random variable X, i.e.,

$$\mu = E[X] \tag{3.2.5.6}$$

E stands for expectation. Using the WIM data from Idaho used in the application example, it is found to be zero. This is expected, because X is the residual or deviation from the regression-predicted wheel weight. b is another model parameter related to the variance of X:

$$2\beta^2 = E[X^2] \tag{3.2.5.7}$$

The WIM data from Idaho were also used to estimate β and λ for both before and after a change in truck weight limit: β = 1.25 kips and $\lambda = 0.1$. They may be used as default data. Fortunately, they were found to be little influenced by the truck weight limit change. This perhaps is because the WIM data were from a case where the wheel weight limits did not change.

The above probabilistic model for the residual X is then used in generating WWHs using TWHs. The procedure is as follows: (1) For each weight interval in TWH, use Eq. 3.2.5.1 to find the mean wheel weight. (2) Then distribute the traffic of the GVW weight interval to a range of wheel weight intervals, with the obtained mean wheel weight at the center of the range. The traffic distribution follows the truncated skewed double exponential distribution discussed above. Note that this procedure has been implemented in the software module in Attachment 5 in the attached CD.

3.3 STEEL MEMBER FATIGUE ASSESSMENT AND FATIGUE TRUCK MODELS

The analysis for this cost-impact category typically consists of the following steps.

1. Identify possibly vulnerable bridges.
2. Sample the possibly vulnerable bridges to reduce the number of bridges to be analyzed in details, if Level I analysis is used.
3. For the analysis of each bridge in the sample (if Level I analysis is performed), generate the TWHs under the Base Case and predict the TWHs under the Alternative Scenario. Estimate remaining safe life and remaining mean life for both the Base Case and Alternative Scenario. Select the responding action based on the estimated remaining lives. Estimate the costs for the selected action.
4. Summarize the costs for all bridges.
5. Perform a sensitivity analysis to understand possible controlling effects of the input data.

The concepts for these steps are discussed in more details next.

3.3.1 Identifying Vulnerable Bridges and Sampling Bridges to Be Analyzed

Vulnerable bridges are defined here as those that have details of E and/or E′ fatigue strength category according to the AASHTO specifications (1990, 1996, 1998). (Section A-5.1.3 in Appendix A presents a set of general guidelines that can be used in this process.) Typically, the agency's bridge inventory can be used for identification of these bridges. The NBI can be used as the default database if the agency does not have more detailed bridge inventory. Most likely, the NBI is needed when a federal-level analysis is performed. It should be noted that if fatigue-prone details other than E and E′ categories are of concern to the agency, they can be added to the analysis process to be covered. Thus the software for the recommended methodology should reserve an option for the user to include these detail types. When this is the case, the agency will need to provide all other information needed

to reach a cost estimate, such as the procedure to obtain the stress range, the repair or replacement procedure, associated unit costs, etc.

When the network being analyzed is extensive including a large number of bridges, a smaller sample will be desired considering the resource constraint. This represents a Level I data requirement case because only the information on this bridge sample will be needed for detailed analysis. Such a sample should be representative for the entire population, as the estimated costs for this sample will be proportioned to the entire population. It is thus advised that sampling be done with respect to the characteristics of the bridges, because these characteristics influence costs, sometimes very significantly. These characteristics may include jurisdiction (state vs. local agency); functional class of the roadway; type of construction (plate girders vs. rolled beams); type of spans (simple vs. continuous); span length; and the year of original construction. Fortunately, this type of information is available in the NBI or a typical state agency's bridge inventory.

Note that the identified bridges as a result of this step are possibly vulnerable ones. They may or may not have the targeted E or E′ details. To confirm the presence of such targeted details, a detailed analysis needs to be performed for each possibly vulnerable bridge (for a Level II analysis) or for each bridge in the sample selected (for a Level I analysis). This detailed analysis should proceed as specified in (AASHTO 1990) or the new AASHTO manual after its adoption, as follows.

3.3.2 Bridge Analysis for Remaining Lives

For each bridge selected (resulting from the last step), the fatigue analysis should follow the AASHTO procedure (1990) to be consistent with current practice (or the new AASHTO manual after adoption). Namely, the following safe life estimation should be used:

$$Y = \frac{fK \times 10^6}{(T_a/T)TC(R_sS_r)^3} \tag{3.3.2.1}$$

where Y is the total life in years. K is a constant tabulated for each type of fatigue sensitive detail in the AASHTO specifications, and f equal to 1 for safe life and 2 for mean life. C is the number of cycles for a passage of the fatigue truck. R_s is a reliability factor. S_r is stress range in ksi for a passage of the fatigue truck whose weight can be more reliably determined using WIM data according to Eq. 3.2.2.11. For the Base Case and the Alternative Scenario, this stress range should be calculated using respective TWHs. The Base Case TWH is based on site-specific WIM data or the default VMT data whose sample is presented in Data Set A-5.2.1 of Appendix A. The Alternative Scenario's TWH is to be developed using the Base Cases' TWH and the prediction method discussed in Section 3.2 above. T is the current annual daily truck volume for the outer lane. T_a is an estimated lifetime-average daily truck volume in the outer lane. The AASHTO specifications (1990) provide values for these parameters or guidelines about determining them.

Note that the AASHTO procedure for T_a represents an approximation, which may lead to under- or overestimates. The following formula is recommended to improve this assessment. Its derivation starts from the definition of T_a/T:

$$\frac{T_a}{T} = \frac{\sum_{i=1}^{Y}(1+u)^i}{Y(1+u)^A} \tag{3.3.2.2}$$

where u is the annual traffic growth rate. It may be estimated using information in the agency's bridge inventory or the NBI (i.e., the latest recorded traffic volume and future traffic volume), if more specific information is not available. A is the current age of the bridge. The numerator in Eq. 3.3.2.2 represents the sum of the total traffic over the life span Y, using a constant annual growth rate u. The numerator divided by the fatigue life Y gives the life-average annual traffic, except that the initial traffic volume is not included. The denominator $(1 + u)^A$ represents the current traffic at the age of A years, except the initial traffic volume. These two missing terms actually cancel each other, and thus they are not shown. According to Eq. 3.3.2.2, it appears that finding Y needs an iterative approach because the unknown Y is in both sides of the equation in Eq. 3.3.2.1. This would require some computational effort.

Fortunately, the summation in Eq. 3.3.2.2 can be explicitly written as

$$\sum_{i-1}^{Y}(1+u)^i = (1+u)\times\left(\frac{(1+u)^Y-1}{u}\right) \tag{3.3.2.3}$$

Substituting Eq. 3.3.2.3 into Eq. 3.3.2.2 and then into Eq. 3.3.2.1 allows directly solving for Y as follows.

$$Y = \frac{\log\left[\frac{fK\times 10^6}{TC(R_sS_r)^3}u(1+u)^{A-1}+1\right]}{\log(1+u)} \tag{3.3.2.4}$$

This formula for Y is very helpful in simplifying the calculation as well as increasing its calculation speed in the software module for the recommended methodology.

3.3.3 Impact-Cost Estimation

The impact costs largely depend on the action to be selected in response to the calculated remaining life changes and other factors. These factors may be the current age of the bridge, agency's policy regarding fatigue repair failure, and so on. While the agency may decide whether more frequent

inspection (monitoring) is warranted, the expected repair and replacement costs are recommended below.

As discussed above, despite the research efforts spent over the past decades on steel fatigue, there is still uncertainty in the estimation process. Therefore, steel fatigue failure is considered to be a random process. Accordingly, it is recommended that a probability based approach be used for estimating the costs directly related to fatigue failure (cracking). These costs could be repair or replacement costs. The default decision in the recommended methodology will be repair, while other actions (such as monitoring) may be added. On the other hand, replacement costs for individual members depend on many factors that cannot be comprehensively covered in this project. They may include, for example, whether or how many other members are to be affected or replaced, the cost-effectiveness for repair compared with other options, etc. On the other hand, repair costs are much less scattered. Some default cost data are included in Appendix A in case more specific cost data are not available. A probabilistic approach is recommended here to estimate the expected repair costs.

The safe remaining life and the mean remaining life are needed in this approach, using Eq. 3.3.2.4. The following equation can be used to estimate the remaining life's standard deviation σ_Y

$$\sigma_Y/Y_{\text{Mean Life}} = -\beta + (\beta^2 + 2\,\mathrm{Ln}(Y_{\text{Mean Life}}/Y_{\text{Safe Life}}))^{1/2} \qquad (3.3.3.1)$$

where $Y_{\text{Safe Life}}$ and $Y_{\text{Mean Life}}$ are safe and mean lives calculated using Eq. 3.3.2.4. β is the target reliability index to which the AASHTO specifications (1990) have been calibrated (Moses et al. 1987). β is equal to 2 and 3, respectively, for redundant and non-redundant components. Then the probability of fatigue failure (cracking) P_f within the considered planning period PP (in years) can be estimated as follows using a truncated lognormal cumulative function LOGN:

$$\begin{aligned} P_f = (&\mathrm{LOGN}(PP + A, Y_{\text{MeanLife}}, \sigma_Y) \\ &- \mathrm{LOGN}(A, Y_{\text{MeanLife}}, \sigma_Y))(1 - \mathrm{LOGN}(A, Y_{\text{MeanLife}}, \sigma_Y)) \end{aligned} \qquad (3.3.3.2)$$

A is the current age of the bridge. The change in this failure probability from the Base Case to the Alternative Scenario is the impact on fatigue failure risk due to the Alternative Scenario being investigated. Thus, the expected impact costs can be estimated as follows

$$\text{Expected Impact Cost} = \text{Impact Cost}(P_{f,AS} - P_{f,BC}) \qquad (3.3.3.3)$$

where subscripts AS and BC indicate respectively the Base Case and the Alternative Scenario. When the expected impact cost turns out to be negative (i.e., the failure probability under the Alternative Scenario is smaller than that under the Base Case), then the expected impact cost is taken to be zero because not impact is expected. The impact cost here depends on the action selected in response to the life change. It may be for repair, replacement, monitoring, or their combinations. The default is recommended to be repair. Data Set A-5.2.4 in Appendix A provides steel fatigue repair cost estimates as the default costs data.

Note that this recommended approach is consistent with the concept of the AASHTO fatigue assessment procedure, using a probability based approach. It has the following advantages. (1) Using the concept of expected cost equal to the cost times the probability of cost incurrence, the likelihood of failure occurrence (i.e., reaching end of fatigue life) is clearly described. This reflects the nature of fatigue failure with uncertainty (including the RC deck fatigue to be discussed below). (2) It also avoids the difficulty in deterministicly deciding which responding action to use, in previously recommended deterministic approaches, in which different decisions could cause extremely large differences.

3.3.4 Validity of the AASHTO Fatigue Truck Model

The recommended procedure above suggests the use of the current AASHTO fatigue truck, which was developed based on WIM data collected many years ago. There is a concern that truck configurations may have changed and will change as a result of truck weight limit changes. This section addresses this issue, by considering a specific scenario of truck weight limit change and providing guidelines for examining the issue for a general Alternative Scenario.

3.3.4.1 Introductory Remarks

This investigation is to quantitatively evaluate the current AASHTO fatigue truck model (defined in *AASHTO Guide Specifications for Fatigue Evaluation of Existing Steel Bridges* 1990) under truck weight limit changes. The AASHTO fatigue truck model has a fixed configuration (with axle distances of 14 and 30 ft, and axle weights of $\frac{1}{9}$, $\frac{4}{9}$, and $\frac{4}{9}$ of GVW for 3 axles as shown in Fig. 2.1). The GVW is determined using the equivalent weight concept based on the truck weight histogram (TWH) if available:

$$GVW = W_{\text{equivalent}} = (\Sigma_{i=1,2,3,\ldots}\, f_{i} W_i^3)^{1/3} \qquad (3.3.4.1)$$

where W_i is the GVW for weight interval i which is taken at the mid-interval, and f_i is the frequency for that weight interval. The AASHTO fatigue truck model was developed using WIM data collected in the early 1980s by Fred Moses and his colleagues (1987). Present investigation is to evaluate whether this model would be valid when truck traffic changes (mainly in configuration or the distribution of GVW among axles) under changes in truck weight limits.

3.3.4.2 *Alternative Scenario*

A specific scenario of weight limit change is selected for this investigation: legalizing GVW of 431 kN (97 kips) on 6 axles. This scenario was selected because of the following considerations. (1) Only those scenarios that legalize new weights across the board could affect the validity of current fatigue truck model. Localized legal weight-limit changes or permit-limit changes would unlikely generate such an impact because the amount of traffic to be affected would be too small. (2) The 431 kN (97 kips) legal weight has the potential to be legalized. It has been, and still will be, a subject of debate at Congress. Further, this scenario is legal in Canadian provinces; this pressures border U.S. states to legalize this scenario (e.g., Michigan).

Because it is not certain what axle distances will be legalized for the selected scenario, two 6-axle configurations are considered here. Shown in Fig. B-2.1.1 in Appendix B, these two configurations represent upper and lower bounds for the reality with respect to axle distances, especially the axle distance between the tridem and the tandem. The two truck configurations are respectively referred to as 3S3A and 3S3B. Note that the steering axle weight is set to be constant because it does not vary significantly with GVW for this kind of configuration. When fully loaded to 431 kN (97 kips), these two configurations will have their tandem and tridem respectively weighing 151 kN (34 kips) and 227 kN (51 kips). It should be noted that, with respect to dimensions, the 3-S3A is likely more acceptable than the 3-S3B because it is shorter, requires less space, and thus is easier to be accommodated.

3.3.4.3 *Approach*

The validity of the AASHTO fatigue truck model is determined herein by understanding whether the model can adequately predict load effects, which relate to fatigue damage more directly than truck weights. Bending moment is used here for this purpose, because it is proportional to stress:

$$M_{equivalent} = (\Sigma_{\, i = 1,2,3,...}\, f_i\, M_i^3)^{1/3} \qquad (3.3.4.2)$$

where M stands for moment, and the rest of the symbols have been defined in Eq. 3.3.4.1. As discussed above, $M_{equivalent}$ can be used to better predict fatigue wear. In routine practice, $M_{equivalent}$ is not readily available because it requires knowledge of trucks' axle weights. A less direct way of estimating fatigue is the AASHTO method as described in Eq. 3.3.4.1. The difference between $M_{equivalent}$ and the moment induced by the AASHTO fatigue truck model (with GVW = $W_{equivalent}$ as defined in Eq. 3.3.4.1) is used here to indicate the model's validity:

$$\text{Error} = M_{\text{AASHTO fatigue truck model}}/M_{equivalent} - 1 \qquad (3.3.4.3)$$

WIM data collected in 1996 are used here from interstate rural highways in New York, which has a significant number of steel bridges. These are the latest data from that state available at FHWA. These data are used here as the load for the Base Case. The researchers then apply the recommended method for predicting TWHs presented in Section 3.2 to estimate the load under the Alternative Scenario (i.e., legalizing 431 kN or 97 kip GVW on 6 axles). For the midspan moment in simple spans, the results are compared using Eq. 3.3.4.3. The default window parameters are used for this investigation.

All "shifting" of loads between truck configurations due to the hypothetical weight-limit change is done for the individual 3-S3 trucks. Namely, each truck in the original WIM data is examined to determine whether its load will be hauled by a new 6-axle semi that could haul higher load. Only those 5-axle trucks that have a GVW close to the current weight limit are eligible to be shifted to 6-axle trucks because they are likely to be affected. Only a specified fraction of these trucks will be subject to such shifting, according to the default window parameters $a_1 = a_2 = 10\%$, $b_1 = b_2 = 20\%$, and c = 0.95. When a truck is confirmed to be eligible for shifting, deciding whether the particular truck will be shifted or not is based on random selection, which assures the specified fraction. If shifted, the truck is replaced by a new truck with 6 axles. As a result of shifting, the number of trucks is also reduced to maintain constant payload, as defined by Eq. 3.2.2.2. After all trucks have been examined and shifted if deemed necessary, they are used to find their maximum midspan moment for a simple span. A histogram of moment is then generated for calculation in Eq. 3.3.4.2, which is used to find $M_{equivalent}$ for Eq. 3.3.4.3.

3.3.4.4 *Results and Conclusions*

As discussed above, inadequate information exists on the real configurations of 6-axle semis. The 3S3-A and 3S3-B in Fig. B-2.1.1 are respectively used for this purpose to produce results as two bounds for the reality, as shown in Table 3.8 for a range of simple spans. For comparison, the Base Case data are also used in Table 3.8 to show the validity of the AASHTO fatigue truck model for current truck traffic. Several observations are made as follows for these results.

1. If most 6-axle semis have a configuration of 3S3-A, the AASHTO fatigue truck model is still valid, as shown by low errors in the column of 3S3-A in Table 3.8. The maximum error there is 3.66 percent for moment (or stress) and 11.39 percent (= $1.0366^3 - 1$) for fatigue wear according to the cube rule. This is because the 3S3-A configuration is close to that of the AASHTO fatigue truck. Further, this validity improves with span length, because the weight distribution among axles in a truck becomes less significant to moment for longer spans.

TABLE 3.8 Errors by using the AASHTO fatigue truck model under the alternative scenario of legalizing 431 kN (97 kips) GVW on 6 axles

		Error (%)(Using Eq.3.3.4.3)		
		Alternative Scenario Shifted to 3S3-A	Alternative Scenario Shifted to 3S3-B	Base Case Without Shifting
Span Length in m (ft)				
18	(59)	- 0.88	- 0.88	- 0.96
30	(98)	+3.66	+15.56	+5.22
42	(138)	+2.54	+11.29	+3.57
54	(177)	+2.34	+8.26	+2.71
66	(216)	+1.54	+6.50	+2.17

2. If most 6-axle semis have a configuration of 3S3-B, the AASHTO fatigue truck will be less valid. For some spans, this becomes severe, as shown by larger errors in the column of 3S3-B. For example, for a span of 30 m, the error is 15.56 percent. This error will cause an overestimation of fatigue by 54.3 percent according to the cube rule.
3. The reality is understood to be between the two bounds discussed above. Depending on how close the real truck configurations are to 3S3-A and 3S3-B, the real error would rest between the two bounds (Columns of 3-S3A and 3-S3B) given in Table 3.8.
4. For very short spans, the axle, tandem, or tridem weights become governing for moment. Thus the difference between different configurations becomes small, as shown in Table 3.8 for the 18-m span. This span cannot have all the axles on the bridge for the 3S3-B, and cannot have axles all contributing to moment significantly for the 3S3-A either. As a result, the tandem and tridem weights control the maximum moment.
5. For current truck traffic, the AASHTO fatigue truck still appears to be a reasonable model, as shown in the column of Base Case without shifting. The maximum error is 5.22 percent for moment, thus 16.49 percent for fatigue damage.
6. It should be noted that medium span lengths that are just long enough to have all the axles on the span and all of them making notable contributions to moment would suffer from highest approximation as shown in Table 3.8 for the 30-m (98-ft) span length. When the span length increases, this approximation becomes more acceptable.
7. In general, if the new trucks under the Alternative Scenario do not significantly differ from the current AASHTO fatigue truck in configuration (for a span length), the AASHTO fatigue truck would still be valid. Further, if the replaced traffic does not occupy a large percentage of the traffic traveling at the current weight limit (e.g., permit truck traffic or localized legal trucks), the current fatigue truck would still be valid. In other words, thereby caused approximation in fatigue assessment will be acceptable.

3.3.5 Sensitivity Analysis

As alluded to or directly discussed earlier, there is a level of uncertainty involved in the AASHTO fatigue assessment procedure. It is thus critical to understand the effects of this uncertainty on the final results, the estimated expected impact costs. Due to a large number of parameters used here in the recommended methodology, a general approach to this requirement is recommended to be as follows: (1) identify those parameters or assumptions that may significantly influence the final result and (2) alter the identified parameters in a realistic range and re-perform the analysis accordingly. This sensitivity analysis will help identify those parameters that have more dominant influence or higher sensitivity. These parameters may need re-examination and possibly adjustment for more reliable results. This concept is recommended for all four cost impact categories covered in the recommended methodology.

For the cost-impact category of steel fatigue, the following parameters may need to be examined for their effects on the final result in the sensitivity analysis. (1) The window parameters a_1, a_2, b_1, b_2, and c, and the parameter for exogenous shifting in the TWH prediction method, as defined in Section 3.2. (2) Load distribution factor used to calculate the stress range. (3) Impact factor. (4) ADTT. (5) Repair cost data. (6) Action selected by the agency user. (7) The sample bridges selected, if Level I analysis is used.

3.3.6 Secondary Bending

Fatigue failure caused by secondary bending is commonly observed in the field. It results from distortion of members and partial fixity at connections that are assumed to be pinned (Moses et al. 1987). Currently there are no general quantitative methods for identifying and analyzing them for fatigue assessment, mainly because this type of failure is a result of local condition, which may vary significantly over the nation. Thus, it is very difficult to develop general guidelines for identification and analysis to cover a large variety of situations. Section A-5.1.3 offers a general concept for addressing cost impact for this type of steel fatigue. The assumption used

there is that, within a jurisdiction, the variation of situation may be much smaller. This situation may make it possible to perform detailed analysis for several typical vulnerable details common within the jurisdiction.

3.4 RC DECK FATIGUE

3.4.1 The RC Deck Fatigue Model

The following formula has been recommended for predicting fatigue failure of RC decks:

$$\text{Log}(P/P_u) = A + B\ \text{Log}(N) \tag{3.4.1.1}$$

where P/P_u is the ratio of the repetitively applied load P and the static ultimate strength of the concrete deck P_u. N is the number of times (cycles) load P is repetitively applied. A and B are model parameters to be determined based on reported physical testing and statistical analysis of the test results. Note that this format is very similar to that for steel fatigue discussed above, known as S-N curves:

$$\text{Log}\ (S) = A + B\ \text{Log}(N) \tag{3.4.1.2}$$

where S is the stress range due to repetitively applied load. The rest of the symbols are defined the same as those in Eq. 3.4.1.1. Parameter B has been found approximately equal to −1/3 for steel fatigue, based on a large number of tests (Moses et al. 1987). Further, parameter A has been found to be dependent on the type of weld detail. The AASHTO bridge design codes (1996, 1998) classify these weld details into fatigue strength categories A through E′. Using the principle of Eq. 3.4.1.2 and the assumption of linear accumulation of damage (the Miner's Law), the AASHTO specifications (1990) include provisions on A for fatigue evaluation, which are being integrated into the new AASHTO evaluation manual under NCHRP Project 12-46 (AGLichtenstein & Associates 1999). (This AASHTO procedure has been included in the recommended methodology in Appendix A.)

The latest effort of investigating RC deck fatigue was reported in Perdikaris et al. (1993) and the study was conducted at Case Western Reserve University. In this study for Ohio DOT, a large number of rolling wheel tests were performed on 1/3– and 1/6.6–scale models, as well as static tests for the ultimate capacities of these deck models. The model system, including the deck, the beams, and the wheel load, was carefully scaled according to the similitude theorem. Three reinforcement ratios were used in the testing program: the AASHTO method (0.7% in the transverse direction and 0.35% in the longitudinal direction); the Ontario method (0.3% in both directions); and isotropic 0.2% in both directions. A scaled wheel load was used to apply moving (rolling) load to the deck. Two prototype beam spacings were included in the test program: 7 ft and 10 ft. The 1/3–scale models had two steel beams supporting a deck, and the 1/6.6 models had 4 steel beams simulating typical U.S. highway bridges. The test program of this ODOT project represents the most comprehensive research effort to date for RC deck fatigue behavior. Cracking damage was shown resembling that in real bridges, which was also observed by other researchers independently (Matsui 1991, Kato and Goto 1984, Okada et al. 1978).

The results reported in Perdikaris et al. (1993) can be summarized as

$$\text{Log}(P/P_u) = -0.1737 - 0.0557\ \text{Log}(N) \tag{3.4.1.3}$$

which is in the same format as Eq. 3.4.1.2 for steel fatigue, except that the stress S is replaced by a stress ratio P/P_u.

On the other hand, the ODOT study did not cover the effects of water presence. Using full-scale models for highway bridges in Japan, Matsui (1991) found that water worsens the situation and significantly accelerates the fatigue process. The fatigue life (number of cycles) may be reduced by as much as 1,000 times as a result of water presence because water "washes" cement and sand off the cracked surfaces, enlarges the crack, and in turn increases the rate of deterioration. As discussed above, the load–life (S-N) curves are shown as straight lines in the log-log scale. By comparing these straight lines under dry and wet conditions, Matsui (1991) found that the S-N curves for dry condition are "rigidly shifted" down by an amount, almost without a change in the slope. Namely, the interception of the straight line was lowered (i.e., parameter A in Eq. 3.4.1.1 is reduced by an amount).

3.4.2 The Recommended Fatigue Assessment Procedure

Based on Eq. 3.4.1.3 the following procedure is recommended for assessing RC deck fatigue using a similar format to that in Article 3.2 of the AASHTO specifications (1990) for steel fatigue assessment:

$$Y_d = \frac{K_d K_p}{(T_a/T)TC_d(R_d I P_s P/P_u)^{17.95}} \tag{3.4.2.1a}$$

where Y_d is the service life of the deck. Using the concept in Eq. 3.3.2.4, Y_d can be explicitly computed as follows

$$Y_d = \log \frac{\left\{\dfrac{K_d K_p}{TC_d(R_d I P_s P/P_u)^{17.95}}\, u(1+u)^{A-1} + 1\right\}}{\log(1+u)} \tag{3.4.2.1b}$$

Y_d will be the mean service life for the reliability factor R_d set equal to 1 and the evaluation life for R_d equal to 1.35. T_a is the life-average of daily truck volume and T is the current daily truck traffic volume for the outer lane, as used in Eq. 3.3.2.1 for steel fatigue. C_d is the average number of axles per

truck. P/P_u is the equivalent stress ratio caused by wheel load P defined as follows:

$$P/P_u = [\Sigma f_i (P_i/P_u)(P_i/P_u)^{17.95}]^{1/17.95} \qquad (3.4.2.2)$$

where P_u is the ultimate shear capacity of the deck. Eq. 3.4.2.1 uses the same linear damage accumulation assumption (the Miner's Law) as for steel fatigue. K_d is a coefficient that covers the model uncertainty (with respect to the assumed Miner's Law). For calculation convenience, the model constant A = –0.000762 in Eq. 3.4.1.3 and a constant of 365 days/year are also include in K_d. K_p addresses the difference between the time of deck failure (punch through) described by Eq. 3.4.1.3 and the time of real deck treatment. It also covers accelerated fatigue due to water presence, which is a variable over the country due to climate condition.

The parameters in Eq. 3.4.2.1 may be divided into three groups: (1) load magnitude related (I, P_s, and P/P_u); (2) number of stress cycles related (T_a/T, T, and C_d); and (3) model-related (K_d and K_p). These parameters are further discussed as follows.

The recommended approach is based on the following concept. The useful service life of a bridge deck is a random variable that is a function of a number of other variables: load magnitudes, number of load cycles, and decision as to when it should be renewed (by overlay or replacement). Note that patching is considered to improve service to the public by providing better riding quality, but it does not increase structural capacity against fatigue.

Note that deciding the end of service life inherently involves uncertainty. Weyers et al. (1994) has showed, in SHRP Project C-103, that the opinions of engineers making the decision on when to overlay a bridge deck are far from uniform. When they were given the same information about the top surface condition of the same decks, their answers had significant scatter as to whether these decks have reached the end of service life or whether they need treatment. The procedure in Eq. 3.4.2.1 has been designed to cover this factor to a certain extent. This is discussed below in more detail.

3.4.2.1 Load Magnitude Related Parameters

In Eq. 3.4.2.1, the nominal impact factor I from the AASHTO specifications is used here to cover dynamic effects of truck wheels. On the other hand, the real dynamic impact is a random variable, assumed to have a mean equal to the code specified nominal value and a standard deviation equal to the mean times the coefficient of variation (COV), which is set equal to 15% (Moses et al. 1987).

The parameter P_s is referred to as axle-group factor. It is to cover effective load increase due to closely spaced wheels in axle groups, such as tandems and tridems commonly used in heavy trucks. In general, this factor is deck dependent because it is a function of the deck's relative geometry related to the following parameters: (1) deck thickness; (2) spacing of the supporting beams (i.e., the span length of the deck); and 3) span length of the supporting beams, which could determine whether the deck is closer to a one-way slab or a two-way slab. Furthermore, the spacings of the wheels in a tandem or tridem are not constant. Therefore, parameter P_s describes an interactive relation between the wheel loads and the deck.

The finite element analysis method was used to understand the effects of the above variables to be covered by P_s for 6 RC decks in Arizona, Alabama, and Georgia. Note that many bridges in these states have not been subjected to de-icing salt, for which the recommended method for fatigue assessment is applicable. The finite element modeling was validated against field test data presented in Fu et al. (1997) using a bridge in New York whose deck was load tested several times over a period of 7 years. Figs. 3.4 to 3.9 show the finite element models for these bridges, with both the deck and supporting beams modeled. Two of these bridges have concrete beams and the rest have steel beams. It is found that the

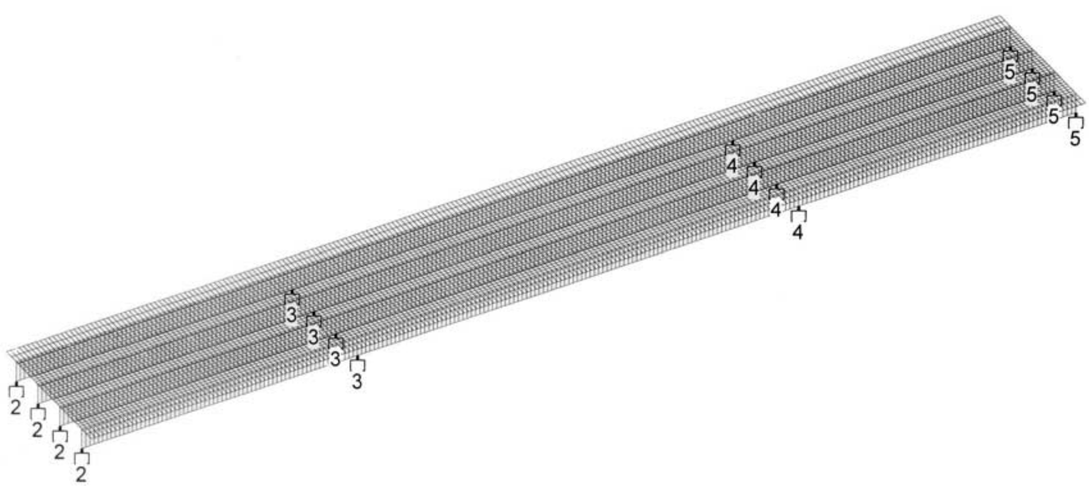


Figure 3.4. Finite element analysis model for Bridge 845 in Arizona.

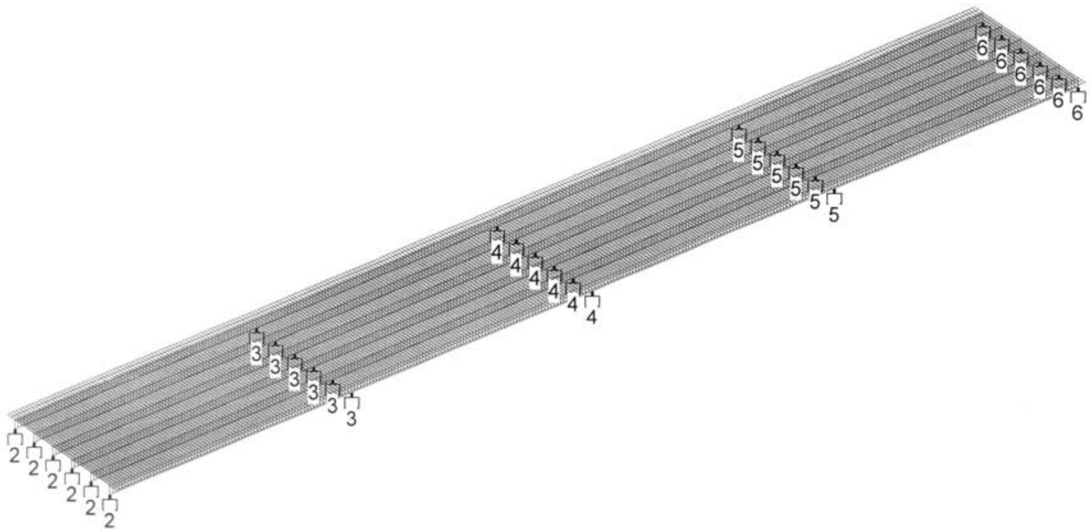


Figure 3.5.   Finite element analysis model for Bridge 1596 in Arizona.

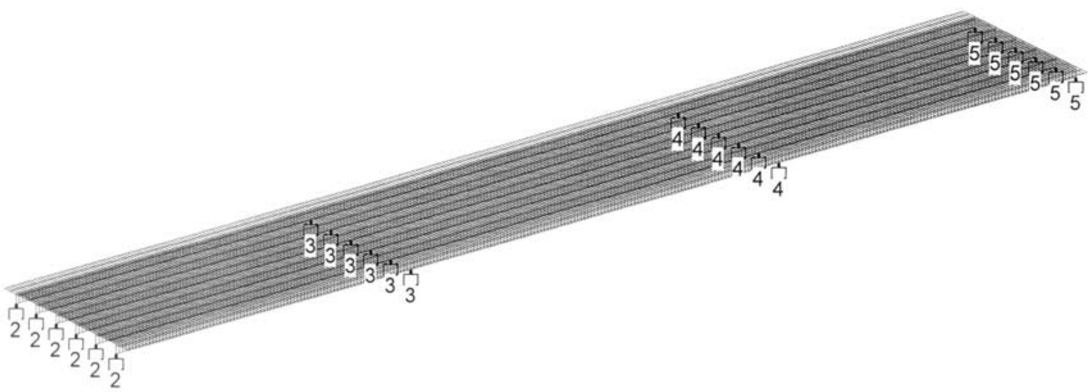


Figure 3.6.   Finite element analysis model for Bridge 12102420 in Georgia.

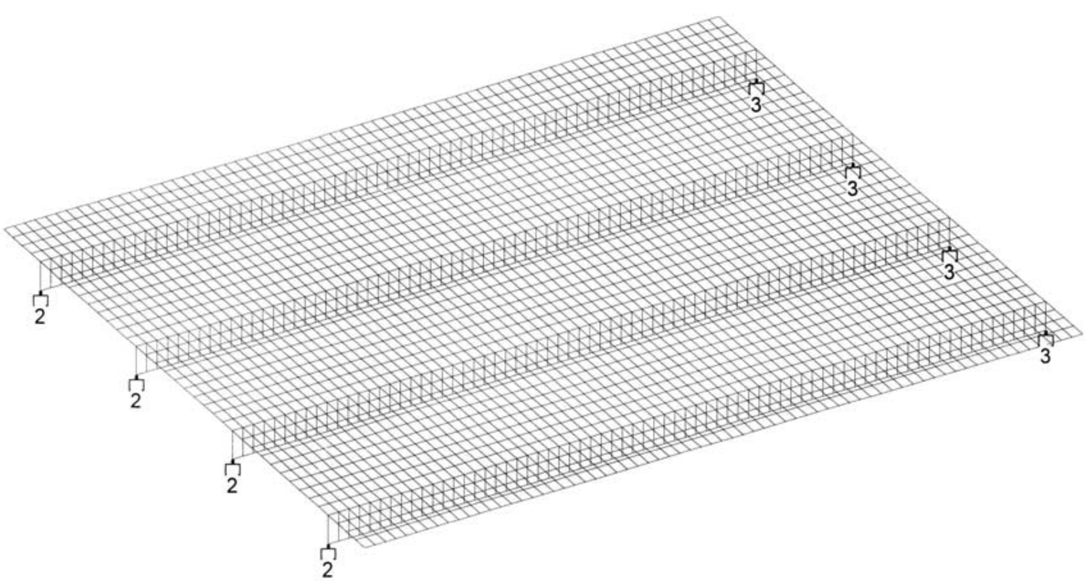


Figure 3.7.   Finite element analysis model for Bridge 7232 in Alabama.

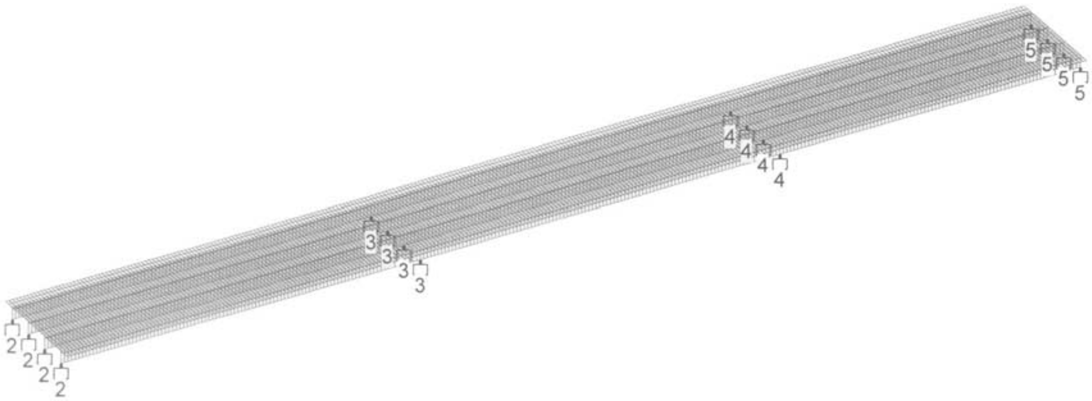


Figure 3.8. Finite element analysis model for Bridge 5360 in Alabama.

shear effect increase due to closely spaced wheels varies from 2 to 9 percent. Based on this set of analysis data, the recommended value for P_s is determined at 1.04 for Eq. 3.4.2.1. P_s actually can be modeled as a random variable having a mean equal to 1.04 and a COV equal to 3.5%.

P_u is the nominal ultimate shear strength of the deck to be estimated as follows, according to the ACI design code (Perdikaris et al. 1993) and the AASHTO design code (1996):

$$P_u = (2 + 4/\alpha)(f_c')^{1/2}\, b_0\, d\gamma < 4(f_c')^{1/2}\, b_0\, d\gamma \qquad (3.4.2.3)$$

where f_c' is the concrete compressive strength in psi. α is the ratio of the tire print's long side to short side, set equal to 2.5 for a nominal tire print of 0.508 m by 0.203 m (20 in. by 8 in.) for dual tires. d is the deck's effective thickness equal to the total thickness minus the bottom cover thickness. It is recommended to also subtract a 0.00635–m (0.25–in.) thick layer from the nominal thickness to account for wearing observed in bridge decks. b_0 is the perimeter of the critical section, which is defined by the straight lines parallel to and at a distance d/2 from the edges of the tire print used. γ is a model correction parameter, which is set at 1.55 based on the test data in Perdikaris et al. (1993). It should be noted that the above parameters are nominal values of respective variables with uncertainty, as in many other cases for strength or fatigue assessment. Thus P_u can be expressed as a random variable with its bias (nominal value divided by the mean value) equal to one and a COV equal to 23%, based on the data reported in Perdikaris et al. (1993).

P in Eq. 3.4.2.1 is an equivalent fatigue load that can be calculated as follows using a WWH.

$$P = (\Sigma f_i(P_i)\, P_i^{17.95})^{1/17.95} \qquad (3.4.2.4)$$

where P_i is the mid-interval value of the ith interval in the WWH, and $f(P_i)$ is the frequency for that interval. Eq. 3.4.2.4 is similar to Eq. 3.3.4.1 for steel fatigue, except that the model constant is 17.95 in the former and is 3 in the latter.

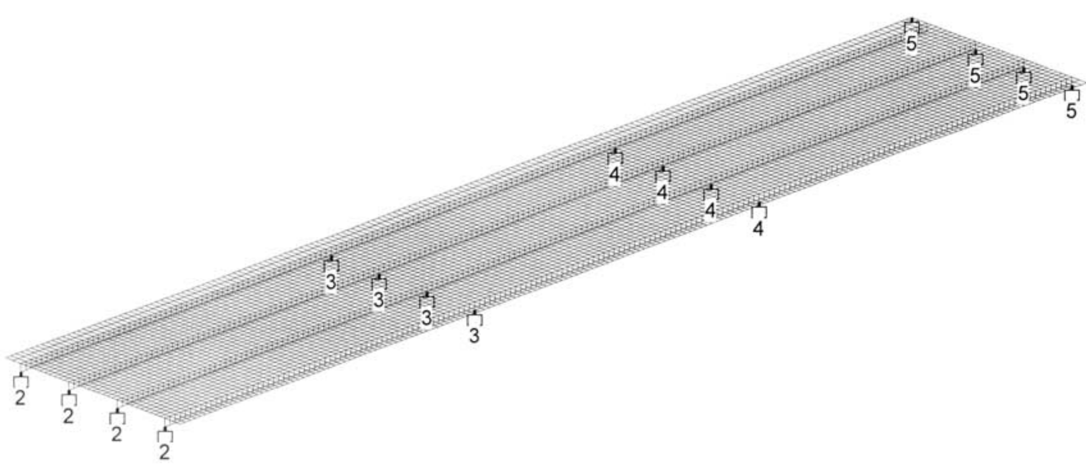


Figure 3.9. Finite element analysis model for Bridge 6446 in Alabama.

Here 17.95 = −1/B with B = −0.0557 taken from Eq. 3.4.1.3 based on reported physical testing results.

Further note that a steering wheel usually consists of a single tire not dual tires. The wheel acts on an area that is approximately half of the dual tire print. Thus the ultimate shear capacity P_u is reduced by about 33 percent. For calculation convenience, P_u can be kept as a constant with the load increased by 1/0.67. In other words, the steering wheel weights should be increased by 1/0.67 for P_u to be treated as a constant and taken out of the summation sign, as indicated in Eq. 3.4.2.4. According to previous research experience (Moses et al. 1987), the equivalent weight P can be described by a random variable with its bias equal to 1 and its COV equal to 0.15.

3.4.2.2 Load Cycle-Related Parameters

These parameters include T_a/T, T, and C_d. Eq. 3.3.2.2 gives the formula to calculate T_a/T, as the ratio of the life-average truck traffic to the current truck traffic for the outer lane. As discussed there, an iterative approach should be used to reach a reliable estimation. T is the current truck traffic for the outer lane, according to the procedure given in AASHTO (1990). This includes adjustment to the total truck traffic recorded in the agency's bridge inventory or the NBI, according to the number of traffic directions (one way or two way) and the number of lanes.

C_d is the average number of axles per vehicle, which may be obtained using appropriate WIM data or the default VMT data whose sample is given in Data Set A-5.2.1 of Appendix A. C_d can be calculated using WIM data according to the following formula:

$$C_d = \Sigma n_i f_i(\text{truck type}_i \text{ with } n_i \text{ axles}) \qquad (3.4.2.5)$$

where f(a) indicates the frequency of a. When appropriate WIM data are not available, the FHWA VMT data may be used to obtain the frequencies f(a) for Eq. 3.4.2.5. In this default data set, 18 vehicle types are included besides automobiles and light 4-tire trucks, which are usually excluded in bridge structure related analyses. Numbers of axles for these 18 vehicles are graphically shown in Data Set A-5.2.1 in Appendix A. They may be used to estimate C_d according to Eq. 3.4.2.5.

3.4.2.3 Model Related Parameters

The model underlying the recommended procedure in Eq. 3.4.2.1, as well as that in Eq. 3.3.2.1 for steel fatigue assessment, is based on Miner's law. It assumes that the fatigue life consumed by one application of a load P is inversely proportional to the number of cycles at which a constant repetitive load P will exhaust the fatigue life. Namely for RC decks:

$$\begin{aligned}\text{Fatigue consumption of one application of P} &= \frac{1}{N} \\ &= \frac{1}{c}(P/P_u)^{17.95}\end{aligned} \qquad (3.4.2.6)$$

where $N(P/P_u)^{17.95} = c$ is the S-N curve based on physical testing, which describes the same relationship as Eq. 3.4.1.1 except in a different format. This linear model is apparently a convenient approximation. Parameter K_d in Eq. 3.4.2.1 is to model the uncertainty in this prediction and is modeled as a random variable. A nominal value of $K_d = 2.09 \times 10^{-6}$ is recommended, based on reported test results (Perdikaris et al. 1993).

Furthermore, the fatigue S-N curve for RC decks shown in Eq. 3.4.1.3 refers to ultimate failure—a cone shaped concrete cracks off (i.e., is punched through) the deck. On the other hand, most RC decks in the United States are overlaid using new concrete or replaced before ultimate failure, except for a few incidents of real failure showing deck holes. This indicates that there is a clear difference in the definition of end of service life between what Eq. 3.4.1.3 describes and what is recognized in practice. The practice includes an apparent safety margin for preventing serious consequence of deck failure. This difference is covered by parameter K_p in Eq. 3.4.2.1. This parameter also covers the effect of water presence that accelerates deck fatigue. A nominal value of $K_p = 3.16 \times 10^{-7}$ is recommended, based on a calibration using 11 bridge decks in Alabama, Arizona, California, Georgia, Mississippi, Nebraska, and Washington. These decks have been overlaid or have been scheduled for overlay in the near future. This influence is also modeled by a random variable, with a bias equal to 1 (i.e., unbiased or mean value equal to the nominal value) and a COV of 2. This COV is relatively large as observed variation in deciding deck rehabilitation and water presence. As commented on above, there is a notable scatter among engineers who make decisions on when a bridge deck needs overlay or replacement for the same physical deck conditions (Weyers et al. 1994). Needless to say, there are other variables beyond the physical condition that contribute to the variation in the deck overlay or replacement decision process; for example, whether other bridges on the same route would need rehabilitation in order to save mobilization costs and user costs caused by traffic disturbance.

Furthermore, the reliability factor R_d in Eq. 3.4.2.1 is determined using the same approach used in Moses et al. (1987) for R_s in steel fatigue assessment according to Eq. 3.3.2.1. This process takes into account the variation of the random variables discussed above. The main purpose for R_d here is to provide a second point on the probability distribution curve for the deck service life being estimated, beside the mean life using R_d equal to 1. This second point is referred to as evaluation life.

R_d is set equal to 1.35 being the same as R_s for steel fatigue. This R_s value corresponds to a reliability index $\beta = 0.94$, due to higher uncertainty observed than that in steel fatigue. With these two points made available, the probability of failure (i.e, the probability of reaching the end of service life) can be computed for any time interval. This project is interested in this probability for the pre-selected PP as discussed next.

3.4.2.4 *Cost-Impact Estimation*

The recommended procedure defined in Eq. 3.4.2.1 offers two values of life: the mean life and the evaluation life. These two values define two points on the distribution curve for the service life of an RC deck, in the same fashion as the procedure for steel fatigue discussed earlier. The mean service life indicates the expected life. The true service life has equal probabilities (50%) to be higher or lower than the mean life. The evaluation life is defined to be associated with a probability value of approximately 0.174 (i.e., the probability of service life smaller than the evaluation life is 0.174). Thus the safety index $\beta = -\Phi^{-1}(0.174) = 0.94$ where Φ^{-1} is the inverse cumulative probability function for the standard normal variable. This procedure for RC deck fatigue has a consistent format with that for steel fatigue assessment.

The expected impact cost can then be estimated in the same way as Eqs. 3.3.3.1 to 3.3.3.3:

$$\text{Expected Impact Cost} = \text{Impact Cost}\,(P_{f,AS} - P_{f,BC}) \qquad (3.4.2.7)$$

where P_f is the probability of failure or probability of reaching the service life end during PP years. σ_Y is the standard deviation of the deck service life, to be calculated as follows, using Eq.3.3.3.1:

$$\sigma_Y/Y_{\text{Mean Life}} = -\beta + (\beta^2 + 2\,\text{Ln}(Y_{\text{Mean Life}}/Y_{\text{Safe Life}}))^{1/2} \qquad \beta = 0.8 \qquad (3.4.2.8)$$

The subscripts BC and AS in Eq. 3.4.2.7 indicate the Base Case and the Alternative Scenario, respectively.

Parameter K_p in Eq. 3.4.2.1 has been calibrated using seven RC decks that have reached the end of service life (recently overlaid or have been planned to be overlaid) in Alabama, Arizona, California, and Georgia. Thus the default responding action is concrete overlay and the default impact cost above is the overlay cost. Note that deck replacement in those states that are not subjected to much snow has been much less frequent than concrete overlay. Thus data are not adequate at this point to calibrate Eq. 3.4.2.1 against replacement need, although it may be performed at a later time when more data become available.

The mechanism of deck fatigue after concrete overlay is also not well understood at this point, particularly for those states not using much salt. As such, the following recommendations are made as to what action should be selected for cost estimation. Options of responding action may be (i) patching and then concrete overlay, (ii) immediate concrete overlay, (iii) patching and then asphalt concrete overlay, (iv) immediate asphalt concrete overlay, and (v) patching and then replacement. These options are discussed in more details next, offering guidelines useful for corresponding cost estimation.

As a principle of cost estimation stated earlier, the impact costs are those expected to incur within PP. Options (i) and (ii) correspond to the situation for which Eq. 3.4.2.1 has been calibrated. Option (i) includes patching in addition to concrete overlay, which is mainly to “buy” time but does not address the structural need of the deck. Thus it is considered to be an option of the agency, which may depend on whether funds for concrete overlay are available. Options (iii) and (iv) actually do not completely address the structural need either but they may buy more time than just patching. They are likely to be done before the deck reaches the condition needing a concrete overlay. They could be a less expensive responding action, compared with concrete overlay but for a shorter life span as a return. Assuming no further action is needed after the asphalt concrete overlay (within PP), this selection is expected to provide a conservative cost estimate or under-estimate. Option (v) is certainly an option even for a deck appearing to need a concrete overlay, because the different needs for overlay and replacement sometimes are not very well defined. Other factors may override their differences. For optimizing life cycle costs at the network level, replacement may be a more cost-effective option than concrete overlay. Thus this option is listed for the agency to decide according to the specific deck situation.

3.4.2.5 *Generation of WWHs for RC Deck Fatigue Assessment*

The required WWHs can be generated using the TWHs respectively for the Base Case and the Alternative Scenario. The starting point of this process could be the truck weight data for TWH as seen in Data Set A-5.2.1 in Appendix A or WIM data. Note that the data in that table are directly taken from the FHWA VMT data that have not been normalized to satisfy the definition of histogram that the summation of all the frequencies should be 1.0. For each vehicle type, Data Set A-5.2.1 in Appendix A offers an empirical way to find the individual wheel weights if the GVW and the configuration are known:

$$\begin{aligned}\text{Wheel Weight} &= 0.5\ \text{Axle Weight}\\ &= 0.5\,(e + f\ \text{GVW}) + X\end{aligned} \qquad (3.2.5.1)$$

where e and f are model parameters resulting from regression analysis of wheel weights and GVW. Data Set 5.2.2 in Appendix A provides the default values for e and f, which was obtained using a large number measured wheel weights and GVWs. When more site-specific data are not available, this data set may be used as the default data. It is recommended that

state agencies obtain jurisdiction specific values for these parameters, using available WIM data that include axle weights and distances. X is the residual from the average wheel weight predicted by the regression relationship 0.5 (e + f GVW), as modeled in Eqs. 3.2.5.2 to 3.2.5.6. This "back track" approach makes it possible to obtain WWHs based on TWHs.

3.4.2.6 An Illustration Example

For illustration, an application example for Eq. 3.4.2.1 is presented here. The RC deck studied here is on Bridge No. 15420 in the state of Idaho, built in 1966. It carries 2 lanes of traffic in two directions. For a total deck thickness of 0.175 m (6⁷/₈ in.), d is taken as 0.143 m (5⁵/₈ in.): $d = d' - c - w = 0.143$ m (5.625 in.) $d' = 0.175$ m (6⁷/₈ in.) is the total thickness, c is the bottom cover equal to 0.0254 m (1 in.), and w accounts for wearing of the thickness, taken as 0.00635 m (0.25 in.). For concrete compressive strength $f'_c = 20.68$ MPa (3000 psi), the ultimate strength P_u is found to be 600 kN (134.9 kips) using a tire print of 0.2032 m × 0.508 m (8 in × 20 in), according to Eq. 3.4.2.3:

$$\begin{aligned} P_u &= (2 + 4/\alpha)(f'_c)^{1/2} b_0 d\gamma \\ &= (2 + 4/2.5)(3000)^{1/2}\,(2(20 + 8 \\ &\quad + 2 \times 5.625))5.625(1.55) \\ &= (3.6)(54.77)(78.5)(5.625)(1.55) \\ &= 134.9 \text{ kips} = 600 \text{ kN} < 4\,(f'_c)^{1/2} b_0 d\gamma \\ &= 4(54.77)(78.5)(5.625)(1.55) \end{aligned} \tag{3.4.2.3}$$

The following recommended model parameters are used:

$P_s = 1.04$ (a constant, based on calibration using decks in several states)
$K_d = 2.09 \times 10^{-6}$ (model constant based on reported RC deck test results)
$K_p = 3.16 \times 10^{-7}$ (model constant calibrated for US field condition for water presence and practice in service life definition)
$R_d = 1$ for mean service life.

Other parameters are calculated as follows.

T = (AADT for the bridge)(Truck Percentage) × (Outer Lane Coefficient from AASHTO 1990)
= (6100)(0.08)(0.6)
= 292.8 trucks per day on outer lane
$C_d = 5.00$, using WIM data and Eq. 3.4.2.5
I = 1.2 (AASHTO 1990)
u = 0.0195 (annual traffic growth rate, estimated using current AADT and future AADT in the NBI)
P = 54.1 kN (12.7 kips) using the WWH shown in Fig. B-1.2.1 and Eq. 3.4.2.4 for the Base Case.
A = 1997 – 1966 = 31 years (current age)

Thus,

$$\begin{aligned} Y_d &= \log \frac{\left\{\dfrac{K_d K_p}{TC_d (R_d I P_s P/P_u)^{17.95}}\, u(1+u)^{A-1} + 1\right\}}{\log(1+u)} \\ &= \log \frac{\left\{\dfrac{(2.09)(3.16)10^{-13}}{292.8(5.00)[1.0(1.2)1.04(54.1/600)]^{17.95}}\right.}{\log(1.0195)} \\ &\quad \frac{\left.(0.0195)(1+0.0195)^{31-1} + 1\right\}}{\log(1.0195)} \\ &= 51.1 \text{ years} \end{aligned} \tag{3.4.2.1b}$$

It should be noted that due to uncertainty observed in reported physical test results and practice in determining end of service life, the real service life of the deck is not certain. Thus a probabilistic approach has been recommended above in Eq. 3.4.2.7 to estimate the expected impact cost as the product of the cost for the action and the probability of reaching the end of service life during the next PP years (which is the probability for that action to take place).

3.4.3 Sensitivity Analysis for the Recommended Procedure

The recommended RC deck fatigue assessment procedure in Eq. 3.4.2.1 is based on reported test results and a calibration for U.S. practice for deck renewal by concrete overlay. These test results and agency practice have notable uncertainty. It is important to understand the effects of such uncertainty in order to guide appropriate application and interpretation of results. This section addresses this issue, by performing a sensitivity analysis.

As discussed above, the terms in Eq. 3.4.2.1 with an exponent of 17.95 are in the same situation for their effect on the estimated life. These parameters include the reliability factor R_d, the dynamic impact factor I, the stress ratio P/P_u, and the axle-group factor P_s for closely spaced truck wheels. For the same bridge used in Section 3.4.2.6 above in the illustration example, the probability of failure (reaching the end of service life) is calculated for three cases of the dynamic impact factor value I, as follows.

Dynamic Impact Factor I	**Probability of Deck Life Exhausted in Next 20 Years**
1.25	0.279
1.20 (reference)	0.335
1.35	0.407

The reference case is that shown in the illustration example above, using the recommended value of 1.2 for I. That value is given in the AASHTO code (1990). It is seen that dynamic

impact factor plays an important role in the resulting probability of failure, using Eq. 3.4.2.7 for cost estimation.

Furthermore, the following results show the effects of the average number of axles per truck C_d in Eq. 3.4.2.1. This factor directly affects the number of load cycles.

Average Number of Axle per Truck C_d	Probability of Deck Life Exhausted in Next 20 Years
4.70	0.330
5.00 (reference)	0.335
5.30	0.340

As seen, its effect on the probability of failure is much smaller than that of the dynamic impact factor I. This is because I has an exponent of 17.95. It follows that the terms with this exponent dominantly contribute to the uncertainty associated with the service life. This observation indicates the importance of determining the stress ratio P/P_u, the dynamic impact I, and the axle-group factor P_s in RC deck fatigue assessment. It also indicates that appropriate wheel weight limits and their enforcement are critical to RC deck service life.

3.4.4 Application of the Recommended Procedure to a Network of Bridges

The Level II analysis requires that the recommended analysis of Eq. 3.4.2.1 be performed for every bridge deck in the network. However, when this becomes excessively costly, a sampling approach is recommended at Level I. The level of data requirement for this cost impact category is very similar to that for steel fatigue at the same level of data requirement and detail. It requires detailed analysis for only a small sample of bridge decks considered to be representative for the entire population. The results for this sample will be used then to project to the entire population.

Based on the sensitivity analysis discussed above, the dominant factors are those included in the relative load term raised to the 18^{th} power. The exponent of 17.95 here is equivalent to the exponent of 3 for steel fatigue. –1/17.95 and –1/3 are the slopes of the S-N curves in the log-log scale respectively for RC deck and steel fatigue. Graphically, a slope of –1/17.95 means a much "flatter" straight line than one with a –1/3 slope. Physically, it indicates that the relative stress range P/P_u is much more dominant or sensitive in life prediction. In other words, a small change in P/P_u could cause a large change in the number of stress cycles N. For example, a 10 percent increase in P/P_u could cause fatigue accumulation increase by 453% ($1.1^{17.95} - 1 = 4.53$), which will cause the predicted life to reduce by 82 percent ($1 - 1.1^{-17.95}$). In contrast for steel fatigue, the same amount of increase in the stress range causes fatigue accumulation increase by only 33 percent ($1.1^3 - 1 = 0.33$), and the predicted life is reduced by only 25 percent ($1 - 1.1^{-3}$). Thus, load is much more dominant for RC deck fatigue accumulation. It also indicates the importance of enforcement for wheel weight limits.

Accordingly, sampling the bridge deck population for Level I analysis should take into account these characteristics of RC deck fatigue. Sites with similar parameters as follows should be grouped together in the sampling process. (1) Sites subjected to heavy wheel loads (not necessarily GVW although wheel weights and GVW may be correlated to certain extent). (2) Bridges that have a rough road surface (perhaps with a low condition rating) causing higher dynamic impact. (3) Decks with a lower thickness and/or lower concrete strength, resulting in a lower P_u and thus relatively high P/P_u. (4) Bridges with similar age (year built) may have similar deck design in terms of thickness and materials. Thus, a bridge (or a few of them) from the same group can well represent the group, because they likely have similar deck thickness, concrete strength, similar deterioration on the driving surface, etc. On the other hand, traffic volume has become secondary, compared with other factors related to the relative load. This has been discussed above in the sensitivity analysis.

3.4.5 Limitation of the Recommended Procedure and Future Research Work

It should be noted that the above recommended procedure for RC deck fatigue assessment is still limited, due to the limited data available and the present state of knowledge. The limits are commented here, which may be used to appropriately interpret the results and to determine future research.

1. Replacement after Overlay as a Responding Action
   The recommended procedure addresses expected cost impact for RC deck fatigue calibrated to the need for concrete overlay. It seems that overlaying an RC deck at least once before replacing has been a popular choice if not a routine practice. This makes sense when it is realized that the deck's life span is usually shorter (some times much shorter) than the life span of the supporting beams. Based on this understanding, the first step of treatment is often overlaying instead of replacement, when the deck needs a significant renewal.
   While deck replacement is listed as an option for cost estimation in Section 3.4.2.4, replacement after one (or more) concrete overlay(s) has not been included as a possible option because there are no reliable data available to help quantify how an overlaid concrete layer works with an old concrete deck. This approach of ignoring the future replacement is conservative in producing an under-estimate. Further, such a replacement may not very likely take place within a typical PP of 20 years for the states that use no or little salt because the life spans of concrete overlay have been estimated around 15 years in states subjected to high rebar corrosion rates due to salt (Weyers et al. 1993, 1994), depending on the type of overlay material used. Thus, the

extended end of deck life by concrete overlay is expected to be beyond the typical PP of 20 years. In other words, within this default PP, the need will unlikely occur for replacing an overlaid deck.

Furthermore, an approach to this issue of estimating fatigue life for an overlaid deck is to assume that the overlaid concrete perfectly bonds to the old concrete and they form a monolithic deck. Further assuming that the renewed deck had a concrete strength equal to that of the new concrete. Then the recommended procedure in Eq. 3.4.2.1 can be applied to estimate the renewed life span staring at the end of the old life span. This would overestimate the deck's fatigue strength and therefore underestimate the cost impact.

2. Patching and Asphalt Overlay as Responding Action Options

Patching (using cement concrete or asphalt concrete) and then asphalt concrete overlay are considered "time buyers" without fundamentally improving the structural condition of the deck. Patching buys the agency less time than the overlay. These measures are taken often because the funds needed for a longer term solution are not available or other actions are not cost effective based on network-level considerations. Thus there should be an option in the recommended methodology for the user to select patching or asphalt overlay if desired. On the other hand, patching needs to be followed by an overlay (using cement concrete or asphalt concrete), as expected to take place within a typical PP of 20 years. After an asphalt-overlay is done, there may or may not be a need for deck replacement within the PP. This possibility is not included as an option for a conservative underestimation

3. Interaction of Rebar Corrosion Due to Salting and Load Related Fatigue

The interacting deterioration between steel reinforcement corrosion and load-related RC fatigue is not quantified in the recommended procedure, because data are not available with regard to the mechanism of such interaction. The steel rebar corrosion has been recognized as a dominant deterioration mechanism for RC decks in the areas where a large amount of salt is used in the winter for de-icing. Thus it should be a decision of the user whether steel rebar corrosion is significant for the concerned bridge network. Fig. 3.10 shows a map indicating typical salt usage for the states, as a result of a SHRP study (Weyer et al. 1994). The states are divided into three groups, referred to as groups of minimal salt usage, moderate salt usage, and severe salt usage. Based on this map, it is recommended that states with minimal salt usage should include RC deck fatigue assessment developed herein in their application of the recommended methodology. Note that there may be exceptions within these states, because salt usage is not uniform even within a state. Also note that in a state belonging to the moderate or sever salt usage, there may be bridges

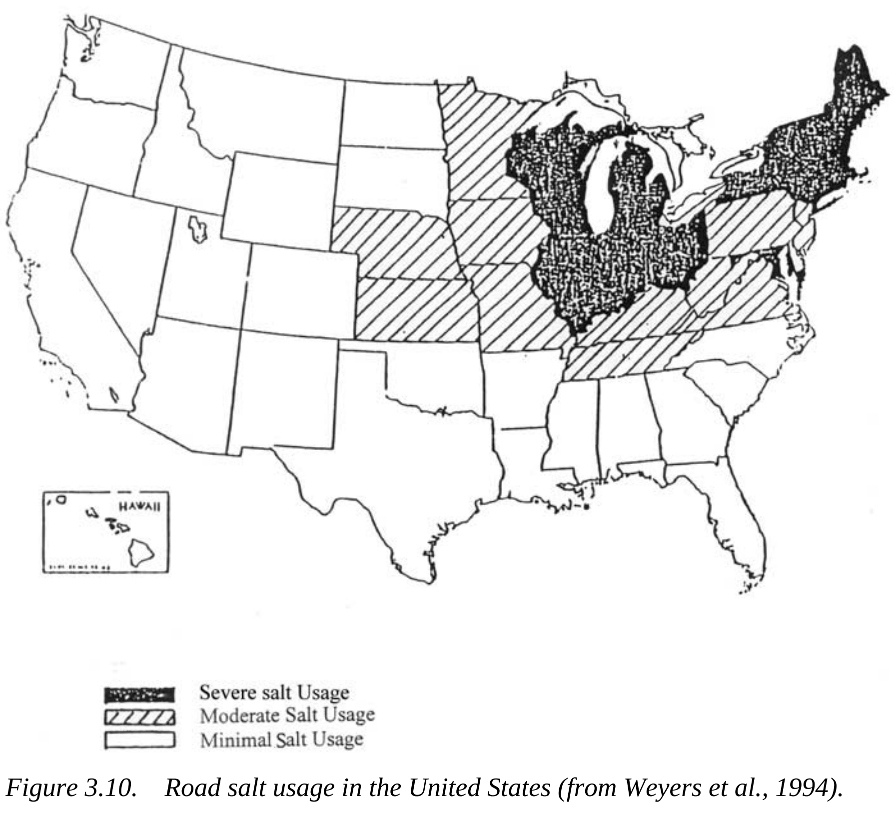


Figure 3.10. Road salt usage in the United States (from Weyers et al., 1994).

that have been subjected to much less salt than in the rest of the state. These bridges should also be analyzed for RC deck fatigue. The user of the recommended methodology needs to make that decision according to the jurisdiction specific or site specific data.

In addition, a conceptual model of interaction between the two factors is discussed below for future research.

3.4.6 Interaction Model of Fatigue Damage and Salt-Induced Rebar Corrosion

Conceptually, corrosion may cause volume increase of steel rebars and thus concrete cracking. This can worsen the fatigue process and increase the rate of damage accumulation. Vice versa, fatigue damage (concrete cracking) may also worsen the damage caused by rebar corrosion. Both situations can adversely change the service life of RC decks.

Accordingly, a model for the interaction is discussed here between load-induced fatigue and salt-induced corrosion in RC deck deterioration. A general format of the model is proposed first, with its rationality presented. In the following section, two major quantities, service lives subjected to load-induced fatigue or slat-induced corrosion only, are further elaborated including the concept of determining these quantities. Then the types of data needed are discussed to fully develop the interaction model. It is suggested that a separate data collection effort needs to be designed and carried out to accomplish this.

3.4.6.1 General Format of Interaction Prediction Model

A literature review was conducted in this task to identify candidate models for describing the interaction focused here. As a result, no models were found in the literature directly related to the subject of interaction between load-induced fatigue and salt-induced corrosion in RC deck deterioration. Furthermore, no research work was found in the literature that investigated the interactive chemical and physical mechanisms, except that general observation was reported that RC decks deteriorate at a higher rate when both salting and loading become severer or either one of them becomes severer.

Note that research efforts have been reported in the literature on load-induced fatigue alone, including the influence of presence of water (e.g., Perdikaris et al. 1993 and Matsui and Yonhee 1993). In addition, research was also done on salt-induced corrosion in RC decks (e.g., Weyers et al. 1993). However, the deterioration prediction model proposed by Weyers et al. was calibrated using data from decks in service condition, apparently also subjected to truck load. Furthermore, these bridges' identification was not reported nor were the truck loads and their volume, so that further tracking back becomes very difficult if not impossible. Although these available data do not meet all the requirements for this task of study interaction, researchers still use this current knowledge here in developing the interaction model concept.

On the other hand, in structural engineering practice, there have been satisfactory interaction models for, for example, interaction between moment and axial load effects to columns and between shear and moment to beams. These interaction models were developed using statistical concepts. Namely, statistical fitting was used in the model development to determine the best fitting parameters for pre-determined functions using physical testing data. These models are considered in this effort of developing the interaction model.

Based on this state of the art, a statistical approach is proposed here for modeling the focused interaction between load and corrosion, as opposed to physical and chemical description of the microscopic deterioration process for RC decks. This approach can be described using the following equation to indicate a deck's end of service life:

$$\left(\frac{A}{Y_d}\right)^a + \left(\frac{A}{Y_c}\right)^b = 1 \qquad (3.4.6.1)$$

where A is the deck's current age (in years). Y_d and Y_c are the predicted service lives (in years) respectively considering load fatigue and rebar corrosion due to salting only. The exponents a and b in Eq. 3.4.6.1 are model parameters. They are always positive. When the left hand side of the formula is equal to or larger than 1, the end of service life is reached. When it is less than 1, the service life has not been exhausted. Thus Eq. 3.4.6.1 indicates the surface of the deck's failure in a space of two dimensions—one for load-induced fatigue and the other for salt-induced rebar corrosion.

Figs. 3.11 to 3.13 show three examples of this kind of surfaces. Fig. 3.11 is for a = b = 1, Fig. 3.12 for a = b = 3, and Fig. 3.13 for a = b = 0.6. They show that when a and b are 1, Eq. 3.4.6.1 is a linear function in the space of A/Y_d and A/Y_c. When a and b are larger than 1, the surface of Eq. 3.4.6.1 is convex away from the origin (Fig. 3.12). When a and b are smaller than 1, it is concave to the origin (Fig. 3.13).

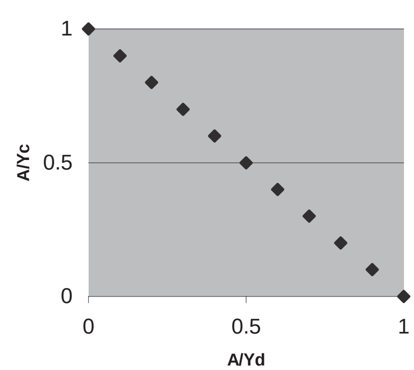


Figure 3.11. Interaction model for a = 1 and b = 1.

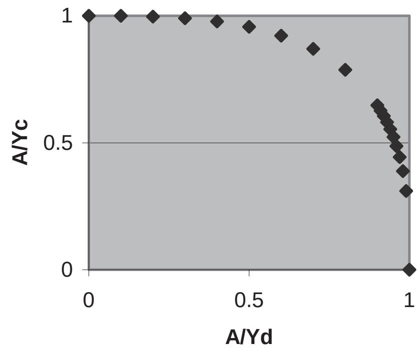


Figure 3.12. Interaction model for a = 3 and b = 3.

When one of the two failure mechanisms is considered to be irrelevant (i.e., having little influence), the respective service life can be accordingly set to infinity. Thus the model shows, mathematically, zero influence from that failure mechanism. For example, in areas where rebar corrosion due to salting is not significant (e.g., where no salting is ever performed), Y_c can be set to infinity. Thus A/Y_c is set equal to zero. The model becomes

$$\left(\frac{A}{Y_d}\right)^a = 1 \quad \text{or} \quad \frac{A}{Y_d} = 1 \tag{3.4.6.2}$$

to indicate end of service life. On the other hand, when load fatigue is deemed to be irrelevant (e.g., in areas where an extremely large amount of salt is used for deicing). Y_d can be viewed to be infinitely large so that A/Y_d can be set equal to 0. The model then becomes

$$\left(\frac{A}{Y_c}\right)^b = 1 \quad \text{or} \quad \frac{A}{Y_c} = 1 \tag{3.4.6.3}$$

to mark the end of service life.

3.4.6.2 Estimation of Yd and Yc

According to the above discussion, Y_d in Eq. 3.4.6.1 is the deck's service life under load fatigue only. Y_c is the deck's service life subjected to rebar-corrosion due to salting only. This project has developed a model to estimate Y_d as follows:

$$Y_d = \log \frac{\left\{\dfrac{K_d K_p}{TC_d (R_d IP_s P/P_u)^{17.95}} u(1+u)^{A-1} + 1\right\}}{\log(1+u)} \tag{3.4.2.1b}$$

This formula has been given earlier using the same identification in Section 3.4.2. The symbols have been defined there. It is listed here for convenience.

On the other hand, estimation for Y_c remains to be a subject of research. The latest comprehensive research work on relevant issues perhaps is (Weyers et al 1993) completed in the SHRP program. That project developed a model for estimating Y_c. However, the data used to determine the model parameters were not differentiated according to what and how much truck load had been applied to the decks that provided performance data. Thus, data from better controlled experiments are needed to complete the development of the model for Y_c for the purpose here.

3.4.6.3 The Need for Data to Complete Development of the Model in Eq. 3.4.6.1

The model parameters in Eq. 3.4.6.1 (exponents a and b, and other parameters in Y_d and Y_c) may be calibrated using statistical techniques applied to data of deck deterioration subjected to load-induced fatigue and salt-induced corrosion. Ideally, both laboratory and in-service data are needed which describe deck deterioration. The former refers to those obtained in the laboratory and the latter in the service condition for decks that have reached end of service life, i.e., have experienced renewal work, such as patching, overlay, or replacement. Laboratory experiments can provide data on deck deterioration in a controlled environment. Data from decks in service can include factors that the laboratory experiments cannot cover, such as temperature and humidity fluctuation.

With respect to truck load and salting, three types of data are needed to fully develop the interaction model. They are deck deterioration history data (1) under load fatigue only, (2) under salt-induced rebar corrosion only, and (3) under different severity combinations of both load fatigue and salt-induced corrosion.

The first and second types of environment conditions can provide data to determine the models to predict Y_d and Y_c in Eq. 3.4.6.1, respectively. Note that Y_d and Y_c are defined above as the service lives without the other deteriorating factor. In other words, they provide data points for areas where A/Y_d is close to 1 and A/Y_c is close to zero, and vice versa A/Y_d is close to zero and A/Y_c is close to 1.

The data under the third type of environment can be used to determine the model parameters (exponents) a and b in Eq. 3.4.6.1. They mainly describe which factor (load-induced fatigue or salt-induced corrosion) is more dominant in which

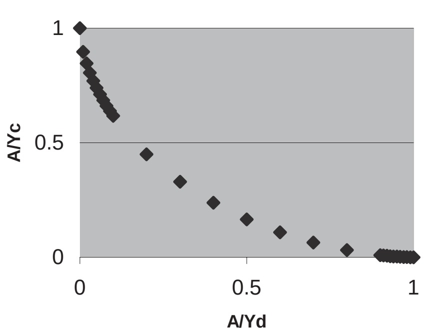


Figure 3.13. Interaction model for a = 0.6 and b = 0.6.

regions. They will provide data points to guide the surface's trend in Figs. 3.11 to 3.15 between the two points ($A/Y_d = 1$, $A/Y_c = 0$) and ($A/Y_d = 0$, $A/Y_c = 1$). Fig. 3.14 shows an example of the failure surface with a = 0.2 and b = 8. It displays a concave trend when A/Y_d is small and convex when A/Y_d is larger and closer to 1. This would be suitable for a behavior of dominant influence from load-induced fatigue. In other words, this case indicates relatively less influencing salt-induced corrosion. For comparison, Fig. 3.15 shows an opposite situation with salt-induced corrosion more dominant and thus the exponents a and b have their values switched as a = 8 and b = 0.2.

The first type of data used in this project was from Perdikaris et al. (1993) in the laboratory condition and from several state DOTs for the in-service condition. They were used to develop the model in Eq. 3.4.2.1 to estimate Y_d.

For the second and third types of data, the researchers have tried to use data gathered in Weyers et al. (1993) to complete a model for Y_c. As commented on above, unfortunately, these data do not have the bridge deck identified and truck load recorded. The researchers also contacted several DOTs and researchers experienced in this area, the following difficulties were encountered in the order of significance.

1. Salt usage is usually not recorded. The best data retrievable were based on very "rough" estimation.
2. WIM truck wheel weight data are not available for specific bridge decks (i.e., those that experienced renewal work).

Furthermore, no laboratory data were found in the literature or other sources for the second and third types of data. These data points should be in the region where A/Y_d is not close to 1 in Figs. 3.11 to 3.15 and A/Y_c varies from almost zero to 1.

Therefore, it is recommended that laboratory experiments and field data collection be designed specifically for the purpose of developing the subject interaction model. It appears that gathering data from efforts with other research purposes will not meet the need here.

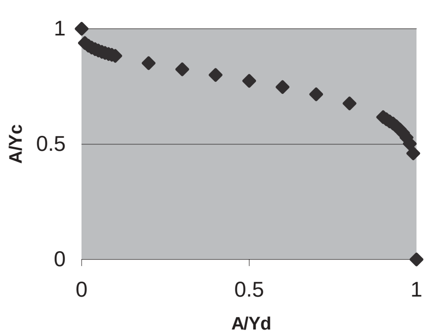


Figure 3.14. Interaction model for a = 0.2 and b = 8.

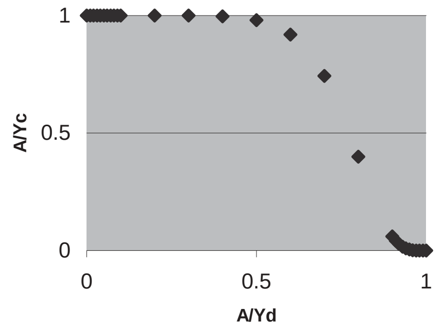


Figure 3.15. Interaction model for a = 8 and b = 0.2.

3.5 DEFICIENCY DUE TO OVERSTRESS FOR EXISTING BRIDGES

3.5.1 Level I Analysis

The concept of overstress criteria recommended here is consistent with that employed by state transportation agencies, based on the AASHTO load rating procedure. Namely, for a specific truck, if a bridge's rating factor is below 1.0, the bridge is considered to be overstressed for that truck. Typically, load rating requires detailed structure analysis, which would be resource consuming if every bridge needed to be analyzed in a large network. A conservative approach is adopted in the recommended methodology for Level I analysis, to reduce the amount of work needed for re-rating every bridge. It uses the existing rating factor as follows:

$$RF_{AS} = RF_{BC}(M_{BC,\ rating\ vehicle}/M_{AS,\ rating\ vehicle})/AF_{rating} \quad (3.5.1.1)$$

where RF_{AS} is the rating factor for the Alternative Scenario. RF_{BC} is the rating factor for the Base Case (likely the existing rating factor). $M_{BC,\ rating\ vehicle}/M_{AS,\ rating\ vehicle}$ is the ratio between the maximum moments due to the rating vehicle under the Base Case and due to the new rating vehicle under the Alternative Scenario. When continuous spans are analyzed, this is the maximum ratio of those for all critical sections. This ratio should not be larger than 1, otherwise it is set at 1. This is because when this ratio is larger than 1, it means that the moment effect of the new rating vehicle is smaller than that of the current rating vehicle. Thus, the new vehicle load effect would not govern in the process of load rating. Generic spans (without specific details from the plans) may be used to find these moment ratios for the interested spans. AF_{rating} is the ratio between the live load factors for the Base Case and the Alternative Scenario, according to Eq. 2.3.3.1:

$$AF_{rating} = [2W^*_{AS} + 1.41\,t(ADTT_{AS})\,\sigma^*_{AS}]/[2W^*_{BC} + 1.41\,t(ADTT_{BC})\,\sigma^*_{BC}] \quad (3.5.1.2)$$

where W* and σ* are the mean and standard deviation of the top 20 percent of the TWH, and t is a function of annual daily truck traffic (ADTT) as given in Section A-3 in Appendix A. Subscripts $_{BS}$ and $_{AS}$ respectively refer to the Base Case and the Alternative Scenario. The ADTT data for the Base Case can be taken from the agency's bridge inventory or the NBI according to the functional class of the roadway that the bridge carries. The ADTT for the Alternative Scenario results from the prediction for future TWH using the recommended method presented in Section 2.4. Eq. 3.5.1.2 also indicates that this ratio of load factors should not be less than 1. In case the calculated value of the ratio is indeed less than 1, then it is set equal to 1. This is because the new safety factor would not be lower than then current load factor for the purpose of cost estimation here.

In Eq. 3.5.1.1, $M_{BC,\ rating\ vehicle}/M_{AS,\ rating\ vehicle}$ reflects the adjustment to rating due to the new truck model. As discussed earlier in Chapter 2, the new truck model should represent the practical maximum truck loads under the Alternative Scenario. It is envisioned that determining this model would not be difficult at a state agency, using available expertise in both areas of bridge structures and transportation planning. The adjustment factor AF_{rating} is the ratio of the live load factors discussed in Section 2.3.3.2. The adjustment covers uncertainty changes in truck weight spectra, expected to result from the considered Alternative Scenario.

For cost estimation, those bridges that are inadequate with $RF_{BC} < 1$ under the Base Case should be excluded, because they do not contribute to the cost impact (additional costs). When a bridge is found to be inadequate or overstressed under the Alternative Scenario but adequate under the Base Case ($RF_{BC} \geq 1$ and $RF_{AS} < 1$), an action needs to be selected as the basis for cost estimation. It can be, for example, posting, strengthening, replacing, or a combination thereof. Note that, in reality, the decision-making process requires information on a number of other factors. Such information may not be available when an application of the methodology is conducted. For example, whether this bridge is on a road that has several other bridges needing repair at the same time can be such information. Thus, this decision is to be made by the user with available information as well as engineering judgment.

3.5.2 Level II Analysis

This level of analysis requires more data and more analysis effort. It requires re-rating every bridge in the network using the rating truck model under the Alternative Scenario. Then the resulting rating factor is modified as follows to arrive at the rating factor for the Alternative Scenario:

$$RF_{AS} = RF_{BC,\ using\ AS\ rating\ vehicle}/AF_{rating} \tag{3.5.1.3}$$

where $RF_{BC,\ using\ AS\ rating\ vehicle}$ is the rating factor using the new truck model under the Alternative Scenario and the live load factor under the Base Case. Comparison of Eqs. 3.5.1.3 and 3.5.1.1 indicates that the Level II analysis requires a bridge-by-bridge approach for re-rating, using the new rating vehicle under the Alternative Scenario. The exact critical sections will be identified and used in this process. It increases the accuracy of the result but possibly requires a larger amount of analysis work, if the network is extensive.

3.6 DEFICIENCY DUE TO OVERSTRESS FOR NEW BRIDGES

As discussed in Section 2.3.4, the bridge design load is required to statistically envelope current and future truck loads over the expected life spans of the bridges to be designed. The design load needs to be updated when a significant percentage of the trucks are to change in terms of their weight distribution and total weight. This is typically expected when the considered Alternative Scenario is to legalize certain types of trucks or to permit routine overweight trucks without controlling the number of trips they may make. Of course the degree of impact depends on the nature of the Alternative Scenario. Eq. 2.3.4.1 indicates that this can be quantified using the mean and standard deviation of the top 20 percent of the TWH.

When the design load is changed, new bridges will cost differently from those under the old design load. This contributes to the cost impact under this category. The recommended methodology uses the concept of incremental cost allocation (Saklas 1998). This approach attributes incremental costs to respective groups of vehicles that trigger the increments. Accordingly, the considered Alternative Scenario is viewed responsible for the incremental costs here for a new design load.

As in the cost impact category for deficient exiting bridges, a new truck model needs to be determined which is able to cover the practically possible legal or permissible vehicles. This can be the practical maximum vehicle under the Alternative Scenario. A similar approach to that used in Cost Impact Category 3 for deficient existing bridges can be used to determine this model. It may consist of several vehicles, depending on the considered Alternative Scenario.

The second step for this category is to generate the TWH for the entire network under the Base Case, and then predict the TWH for the network under the Alternative Scenario. These two TWHs will be used below to determine a live load factor ratio as part of the new design load. For a state agency, these TWHs should usually be representative for the entire state, not site specific or functional class specific as in Cost Impact Category 3, because, most likely the live load factor for design is uniform for the entire state. An exception may be that certain local bridges are considered not subject to

general heavy loads and higher truck traffic and therefore a lower design load is justifiable.

3.6.1 Level I Analysis

At this level of data requirement and the related amount of analysis, new bridges constructed in recent years are used to estimate the costs that are expected if the considered Alternative Scenario is implemented. This analysis will require the following further steps: (1) Identify the new bridges constructed in the past Q years. Q needs to be determined with consideration to the number of new bridges to be included. (2) Estimate the required design load for each of these bridges under the Alternative scenario. (3) Estimate the additional costs for each of these bridges under the new design load.

Step (1) is feasible using the agency's bridge inventory or the NBI as the default database, where the year built is recorded for each bridge in the system. Q, the number of years the analysis should track back largely depends on the number of bridges to be included. When a large number of bridges belong to this group for a relatively large network, fewer years may be used for a small Q. In contrast, if a small number of bridges were typically constructed in each year, a larger number of years would be desired to arrive at a reliable annual cost for new bridges. Thus Q may need to be determined by iteration combined with the sensitivity analysis to be discussed below.

Step (2) is to be accomplished using the following formula for the amount of design load change:

$$\text{DLCF} = (M_{\text{AS design vehicle}} / M_{\text{BC design vehicle}})\,\text{AF}_{\text{design}} \tag{3.6.1.1}$$

$$M_{\text{AS design vehicle}} / M_{\text{BC design vehicle}} \geq 1 \tag{3.6.1.2}$$

$$\text{AF}_{\text{design}} = (2W^*_{\text{AS}} + 6.9\,\sigma^*_{\text{AS}}) / (2W^*_{\text{BC}} + 6.9\,\sigma^*_{\text{BC}})$$

$$\text{AF}_{\text{design}} \geq 1 \tag{3.6.1.3}$$

where DLCF stands for design load change factor indicating the ratio between the design loads under the Base Case and the Alternative Scenario. $M_{\text{AS, design vehicle}} / M_{\text{BC, design vehicle}}$ is the ratio of the maximum moments due to the design vehicle under the Base Case and the same under the Alternative Scenario. Practically, it should not be lower than 1. Namely, when $M_{\text{AS design vehicle}}$ is smaller than $M_{\text{BC design vehicle}}$, the design vehicle under the Base Case would be the governing load and the ratio should be taken as 1 in Eq. 3.6.1.2. This will assure that the new design load will not be lower than the current design load. $\text{AF}_{\text{design}}$ is the ratio between the live load factors under the Base Case and the Alternative Scenario. It is an adjustment factor for design used to cover the change in uncertainty associated with the considered Alternative Scenario. It plays a similar role as $\text{AF}_{\text{rating}}$ in Eq. 3.5.1.2 for additional deficiency in existing bridges.

In Level I analysis, the AASHTO HS load is assumed to be the design load for the current norm of design load, although the HL93 has been adopted in the AASHTO LRFD design specifications (1998). It is because the HS load has been the design load for many bridges in service today, which provide the data needed to project to future bridges. One of the important sets of data here is the cost increase data for design loads beyond the HS-20. The default Data Set A-5.2.7 in Appendix A refers to HS-20 as the reference for possibly higher design loads. No such data are found available referring to the HL93. Thus Step (3) of this analysis is to compare the design load under the Alternative Scenario with that under the Base Case with reference to the HS load. This can be done using the DLCF resulting from Eq. 3.6.1.1, which is the multiples of the new design load compared with the Base Case design load. Appendix A includes cost data for additional design load with reference to HS-20 in Data Set A-5.2.7 in Appendix A. They can be used as the default data if more specific data are not available. Note that these data may be used to extrapolate to situations where the design load is beyond the ranges given there, as appropriate.

3.6.2 Level II Analysis

The recommended methodology at this level of analysis requires information about every individual bridge in the network. First of all, the new bridges to be constructed need to be identified using more specific information than the bridge inventory. Such information could include, but not be limited to, the agency's capital program for the next several years, the agency's long term plan for expenditure and candidates for new construction and replacement for the next 5 or 10 years.

Next, the configurations of these future new bridges need to be identified. This information is needed to perform the analysis defined in Eq. 3.6.1.1 to 3.6.1.3. Note that this analysis can be more accurate if more details about the configurations can be provided by the agency. The rest of analysis for the Level II requirement will be identical to that for the Level I requirement.

3.6.3 Sensitivity Analysis

The following parameters may need examination at the stage of sensitivity analysis. (1) The window parameters used in the TWH prediction method for the Alternative Scenario, defined in Fig. 3.1. (2) The bridge sample identified, if a Level I analysis is performed. More years of record of recent new bridges in the network may be included and averaged to produce an annual cost for this category of cost impact. (3) Possible increase of available resources due to economic

growth that result in more new bridges to be built. This may be covered at the network level by a growth factor to the total costs obtained.

3.7 TOTAL COST-IMPACT CALCULATION

When all the four cost-impact categories are covered using the concept described in this chapter, individual contributions from these categories need to be summarized to find the total cost. The summation process should use the principles of engineering economics. A discount rate will need to be predetermined for this purpose, which could be one of the factors subjected to sensitivity analysis. It is recommended that all costs be converted to the same format of expression. Options of this uniform format may be present worth, annual costs for the next PP years, etc. Note that these different forms are equivalent with a discount rate consistently included.

CHAPTER **4**

CONCLUSIONS

This research project developed the recommended methodology for estimating bridge network costs as a result of truck weight limit changes. A software module implementing the methodology has also been developed. This methodology's targeted users are state transportation agencies in the United States. However, federal and local agencies may also find it useful. Four cost impact categories are covered in this methodology, because their cost impact is found to be quantifiable based on the current state of knowledge. However, when the agency wishes to add other cost impact categories based on its experience, the recommended methodology and the software have a flexible structure to accept such an addition. Thus, the recommended methodology can be improved with increase of knowledge in related areas. The methodology has been designed to be flexible to meet the needs of various agencies, in terms of its required data input. A set of default data is also provided to facilitate application.

The procedure of the recommended methodology is presented in Appendix A along with a set of default data, and the concept of the methodology is presented in Chapter 3. Two application examples of the methodology are presented in Appendix B for illustration. The attachments include the software module and its users manual, which can stand alone without this report for routine application. The manual has a chapter of tutorial including Scenario 2 of the Idaho example and the Michigan example, as illustration examples.

The following conclusions can be drawn based on the results of this research effort.

1. Based on the two examples of application for the recommended methodology and previous research results, the cost impact category for deficient existing bridges is likely the dominant contributor to the total cost impact of a change in truck weight limits. This is mainly because there are no general effective methods to strengthen existing bridges for increasing the load ratings. The possible options of replacement and posting plus enforcement both could be costly.
2. The current AASHTO fatigue truck model is found valid based on the current WIM data used. The model can be still valid under the considered scenario of legalizing the 3S3 configuration for a GVW of 431 kN (97 kips) because the envisioned 3S3 configurations are similar to the AASHTO fatigue truck model. Thus this conclusion can be extended to that if the considered scenario legalizes truck types similar to the AASHTO truck model (i.e., typical 3S2 configurations); that model would still be valid for steel fatigue assessment.
3. The models for assessing structural material fatigue (for both steel components and reinforced concrete decks) have more uncertainty than the strength assessing models. Essentially it is because fatigue accumulation largely depends on microscopic original discontinuities and acquired damages, which are randomly distributed in location and severity. Predicting failure originating from such sources is inherently involved with notable uncertainty. Thus, more research work is recommended to reduce the uncertainty in the modeling and prediction. Such effort should focus on modeling loading, because it plays a dominant role, due to the load's higher powered term (cubic power for steel fatigue and the close to 18th power for the reinforced concrete deck fatigue). This also includes predicting the available strength of the material (e.g., shear capacity Pu for RC decks), the stress range for steel members subjected to out-of-plane bending, etc.
4. Wheel loads have a very significant effect on RC deck fatigue accumulation, according to the fatigue model introduced herein. This result has important implications to wheel load limit development and enforcement. More research is recommended to better understand the mechanism of RC deck deterioration due to combined efforts of load and steel corrosion.

SELECTED BIBLIOGRAPHY*

AASHTO (1998): LRFD Bridge Design Specifications, 2^{nd} Ed. Washington, DC.

AASHTO (1996): Standard Specifications for Highway Bridges, 16^{th} Ed. Washington, DC

AASHTO (1994): Manual for Condition Evaluation of Bridges, Washington, D.C.

AASHTO (1991) “Guide for Maximum Dimensions and Weights of Motor Vehicles and for the Operation of Nondivisible Load Oversize and Overweight Vehicles”, Nov. 1991

AASHTO (1990) “Guide Specifications for Fatigue Evaluation of Existing Steel Bridges”, 1990

AASHTO (1989a): Guide Specifications for Fatigue Design of Steel Bridges, Washington, DC.

AASHTO (1989b): Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges, Washington, D.C

AASHTO (1983): Manual for Maintenance Inspection of Bridges, Washington, D.C.

ACI “Building Code Requirements for Reinforced Concrete”, ACI 318-95, American Concrete Institute, 1995

Aderson, J.M., and Rhodes, A.H., “The Prediction of Traffic Loading at Specific Sites”, Heavy Vehicles and Roads: Technology, Safety and Policy, Thomas Telford, London, 1992, p.299.

AGLichtenstein & Associates (1999): NCHRP 12-46 Proposed Manual for Condition Evaluation and Load and Resistance Factor Rating of Highway Bridges—Draft proposed for AASHTO adoption, 1999

Albrecht, P., and Rubeiz, C.G., “Variable Amplitude Load Fatigue Task A-Literature Review Volume III-Variable Amplitude Fatigue Behavior”, FHWA-RD-87-0619, April 1990.

Al-Subhi, Kamal M. Isa, Johnston, David W., and Farid, Foad, “Optimizing System-Level Bridge Maintenance, Rehabilitation, and Replacement Decisions”, FHWA/NC/89-001, January 1989.

Australia Road Research Board Ltd., “Dynamic Interaction of Vehicles and the Infrastructure”, Proc. of DIVINE Special Session: 17^{th} ARRB Conference, Research Report ARR No. 260, (Ed.) by Sharp, K.G.

Azad, A.K., Baluch, M.Y., Al-Mandil, M.Y., Sharif, A.M., and Kareem, K., “Loss of Punching Capacity of Bridge Deck Slabs from Crack Damage”, ACI Structural Journal-Technical Paper, Title No. 90-S5, January 1993, p.37.

Barella, Sara, and Cantieni, Reto, “Vehicle/Bridge Interaction for Medium Span Bridges-Research Element 6 of the OECD IR6 DIVINE Project”, Road Transport Technology-4, Proc. of the Fourth International Symposium on Heavy Vehicle Weights and Dimensions, (Ed.) by Winkl, Christopher, 1995, p.355.

Batchelor, Barrington Dev., Hewitt, Brian E., and Csagoly, P., “An Investigation of the Fatigue Strength of Deck Slabs of Composite Steel/Concrete Bridges”, Transportation Research Record 664, Vol.1, 1978, p.153.

Batchelor, Barrington Dev., Hewitt, Brian E., Csagoly P., and Holowka M., “Investigation of the Ultimate Strength of Deck Slabs of Composite Steel/Concrete Bridges”, Transportation Research Record 664, Bridge Engineering Volume 1, 1978, p.162.

Beal, David B.: “Strength of Concrete Bridge Decks”, Report FHWA/NY/RR-81-89, New York State Department of Transportation, July 1981.

Beal, David B.: “Reinforcement for Concrete Bridge Decks”, report FHWA/NY/RR-83/105, New York State Department of Transportation, July 1983.

Benjamin, J.R. and Cornell, C.A. “Probability, Statistics, and Decision for Civil Engineers”, McGraw-Hill Book Company

Bridge Branch (1997) “The Effect of the NRTC Mass Limits Review on MRWA Road Bridges in Western Australia”, Volume 1, Bridges on National Highways and State Roads, Bridge Branch, Road Technology Directorate, Main Roads Western Australia, August 1997

Bridge Branch (1998) “The Effect of the NRTC Mass Limits Review on Local Government Road Bridges in Western Australia”, Volume 2, Bridge on Local Government Roads, Bridge Branch, Road Technology Directorate, Main Roads Western Australia, March 1998

BTML Division of Wilbur Smith Associates (1987) “Effects of Truck Weights on Deterioration, Operations, and Design of Bridges and Pavements, Final report to NYSDOT, ERDB, Oct. 1987

Cady, P.D., and Carrier, R.E., (1973) “Deterioration of 249 Bridge Decks”, Highway Research Record 423, Concrete Properties and Performance, p.46.

Cady, Philip D., and Weyers, Richard E., “Deterioration Rates of Concrete Bridge Decks”, Journal of Transportation Engineering, Vol. 110, No. 1, January 1984, p.34.

Calgaro, J.A., “Road Traffic Loads Calibration of the Main Loading System”, ENV 1991-3, Traffic Load on Bridges, Background Studies, February 1997.

Cambridge Systematics, Inc. “A Guidebook for Forecasting Freight Transportation demand”, NCHRP Report 388, TRB, 1997

Cantieni, R., “Investigation of Vehicle-Bridge Interaction for Highway Bridges”, Heavy Vehicles and Roads: Technology, Safety and Policy, (Ed.) by Cebon, D., and Mitchell, C.G.B.

Clayton,A., and Plett,R., “Truck Weights as A Function of Regulatory Limits”, CAN.J.ENG.VOL.17, 1990.

Clayton,A., and Thom, Robert G., “Gross Vehicle Weight Distributions as a Function of Weight Limits”, Transportation Research Record, No.1313, p.11.

Csagoly, P., Holowka, M., and Dorton, R., “The True Behavior of Thin Concrete Bridge Slabs”, Transportation Research Record 664, Bridge Engineering Volume 1, 1978, p.171.

Demers, Cornelia E., and Fisher, John W., “Fatigue Cracking of Steel Bridge Structures, Volume I:A Survey of Localized Cracking in Steel Bridges-1981 to 1988”, FHWA-RD-89-166, March 1990.

Dexter,R.J., Connor,R.J., and Kaczinski,M.R. “Fatigue Design of Modular Bridge Expansion Joints”, NCHRP Report 402, TRB, 1997

Dicleci, Murat, and Bruneau, Michel, “Fatigue-Based Methodology for Managing Impact of Heavy-Permit Trucks on Steel Highway Bridges”, Journal of Structural Engineering, November 1995, p.1652.

Downs,H.G. (1981) “Criteria for Evaluation of Truck Weight Enforcement Programs” NCHRP Synthesis 82, Nov. 1981

Euritt, Mark A., Walton, C. Michael, Goff, Zane A., and Burke, Dock, “Texas Highway Cost Allocation Analysis and Estimates”, 1992-1994, TX-94+1919-2/1910-3, November 1993.

Fang,I.-K., J.Worley, N.H.Burns, and R.E.Klingner, “Behavior of Isotropic R/C Bridge Decks on Steel Girders”, ASCE Journal of Structural Engineering, Vol.116, No.3, 1990, p.659

Farid, Foad, Johnston, David W., Laverde, Martha A., and Chen, Chwen-Jinq. “Application of Incremental Benefit-Cost Analysis for Optimal Budget Allocation to Maintenance, Rehabilitation, and Replacement of Bridges”, Transportation Research Record 1442, 1994, p.88.

Fekpe, Edward S.K., and Clayton, Alan M.(1993) “A Rational Approach to Predicting Pavement Loadings”, Canadian Society for Civil Engineering Annual Conference June 8-11 1993, p.597.

Fekpe,E.S.K. and Clayton,A.M. (1995) “Prediction of Heavy-Vehicle Weight Distributions”, ASCE Journal of Transportation Engineering, Vol.121, No.2, p.158

Fekpe,E.S.K. and Clayton,A. (1994) “Quantitative Assessment of the Effect of Enforcement on Violations of Vehicle Weight and Dimension Regulations”, Transport Planning and Technology Journal, Vol.18, No.2, p.143

Fekpe,E.S.K., A.M.Clayton, and R.C.G.Haas. “Evaluating Pavement Impacts of Truck Weight Limits and Enforcement Levels”, Transportation Research Record 1508, 1994, p.39

FHWA (1997) “1997 Federal Highway Cost Allocation Study final report”, US DOT, Federal Highway Administration, 1997, http://www.ota.fhwa.dot.gov/hcas/final.

FHWA (1995) “The Federal Highway Administration’s Highway Cost Allocation Workshop, Searching for Solutions, TD 2130/5/95-039, June 1995.

FHWA (1991) “The 1991 Status of the Nation’s Highways and Bridges”, FHWA-PL-91-015, Nov. 1991

FHWA (1982) “Final Report on the Federal Highway Cost Allocation Study”, Report of the Secretary of Transportation to the United States Congress, Pursuant to Section 506, Public Law 95-599, Surface Transportation Assistance Act of 1978, May 1982

FHWA (1981) “An Investigation of Truck Size and Weight Limits”, Final Report, 1981

Fisher, J.W, Kaufmann, Eric J., and Pense, Alan W., “Effect of Corrosion on Crack Development and Fatigue Life”, Transportation Research Record 1624, Paper No. 98-1038, p.110.

Fisher, J.W., Yen, B.T., and Wang, D., “Fatigue and Fracture Evaluation for Rating Riveted Bridges”, National Cooperative Highway Research Program Report 302, December 1987.

Fisher, John W, Albrecht, Pedro A., Yen, Ben T. Klingerman, David J., and Mcnamee, Bernard M., “Fatigue Strength of Steel Beams with Welded Stiffeners and Attachments”, National Cooperative Highway Research Program Report 147, 1974.

Fisher, John W, Yen, Ben T., and Kaufmann, Eric J., “Cracking and Fracture Assessment of the Green River Bridge, I26 in Henderson County, N.C.”, Advanced Technology for Large Structural Systems, Lehigh University, April 1993.

Fisher, John W, Yen, Ben T., and Wang, Dayi, “Fatigue Cracking of Steel Bridge Structures, Volume II:A Commentary and Guide for Design, Evaluation, and Investigation of Cracking”, FHWA-RD-89-167, March 1990.

Fisher, John W., Jin, Jian, Wagner, David C., and Yen, Ben T., “Distortion-Induced Fatigue Cracking in Steel Bridges”, National Cooperative Highway Research Program Report 336, December 1990.

Fisher,J.W., Mertz,D.R. and Zhong,A. (1983) “Steel Bridge Members under Variable Amplitude Long Life Fatigue Loading”, NCHRP Report 267, 1983

Fitch, Michael G., Weyers, Richard E., and Johnson, Steven D., “Determination of End of Functional Service Life for Concrete Bridge Decks”, Transportation Research Record 1490, 1995, p.60.

Frith, B.A, Mitchell, C.G.B., and Newton, W.H., “Impacts of Increased Goods Vehicle Weight Limits: A Europe Case Study”, Road Transport Technology-4, Proc. of the Fourth International Symposium on Heavy Vehicle Weights and Dimensions, (Ed.) by Winkl, Christopher, 1995, p.55.

Fu, Gongkang, Alampalli, Sreenivas, and Pezze, Frank P. (1994) “Lightly Reinforced Concrete Bridge-Deck Slabs on Steel Stringers: A Summary of Field Experience”, Report FHWA/NY/RR-94/161, June 1994.

Fu,G. and Hag-Elsafi,O. (1996) “New Safety Based Checking Procedure for Overloads on Highway Bridges”, TRB Transportation Research Record 1541, 1996, p.22

Fu,G., S.Alampalli, and F.P.Pezze (1992) “Long-term Serviceability of Isotropically Reinforced Concrete Bridge Deck Slabs”, TRB Transportation Research Record 1371, 1992, p.17

Fu,G. and Moses,F. (1998) “Cost Estimation for Steel Bridge Fatigue”, (presented) TRB Annual Meeting 1998, Washington, DC

Hahin, Christopher, “Effects of Corrosion and Fatigue on the Load Carrying Capacity of Structural and Reinforcing Steel”, FHWA/IL/PR-108, March 1994.

Hahin, Christopher, and South, Jeffrey M., “Accurate and Rapid Determination of Fatigue Damage in Bridge Superstructures”, FHWA/IL/PR-106, December 1992.

Hajek, Jerry J., Tighe, Susan L., and Hutchison, Bruce G., “Allocation of Pavement Damage Due to Trucks Using a Marginal Cost Method”, Transportation Research Board 77th Annual Meeting January 11-15, 1998, Paper No. 981283.

Hawking, Neil M., “Fatigue Design Consideration for Reinforcement in Concrete Bridge Decks”, ACI Journal, Title NO. 73-9, February 1976, p.104.

Heywood,R.J. and Person,R.A. “Adequacy of Australian Bridges”, Department of Transport and Regional Development, 1997

Horn, Burkhard, “International Co-operation on Infrastructure and Heavy Freight Vehicles within OECD”, Road Transport Technology-4, Proc. of the Fourth International Symposium on Heavy Vehicle Weights and Dimensions, (Ed.) by Winkl, Christopher, 1995, p.3.

Hudson,S.W., Carmichael,R.F., Moser,L.O., Hudson,W.R., and Wilkes,W.J. (1987) “Bridge Management Systems”, NCHRP Report 300, 1987

Humphrey, Thomas F., “Uniformity Effects in Oversize/Overweight Permits”, National Cooperative Highway Research Program Synthesis of Highway Practice 143, December 1988.

Humphrey,T.F. (1988) “Uniformity Efforts in Oversize/Overweight Permits” NCHRP Synthesis 143, 1988

IBI/ADI (1994) “Impacts of Canada’s Heavy Vehicle Weights and dimensions research and Interprovincial Agreement”, TAC-CTRI, October 1994.

Illinois Department of Transportation (1992) “The Effects of Increased Truck Size and Weight in Illinois”, 1992

Illinois Department of Transportation, “Oversize and Overweight Permit Movements on State Highways”, Title 92, Chapter I, Subchapter f, Part 554, June 1988.

Imbsen, Roy A., “Bridge Weight-Limit Posting Practice”, National Cooperative Highway Research Program Synthesis of Highway Practice 108, August 1984.

James, Ray W., Noel, James S., Furr,H.L., Bonilla,F.E. "Proposed New Truck Weight Limit Formula", Journal of Structural Engineering, Vol. 112, No. 7, July 1986, p.1589.

James,R.W., Zimmerman,R.A., and McCreary,C.C.,Jr. "effects of Overloads on Deterioration of Concrete Bridges", Transportation Research record 1118, p.65

Johnson, N.L., Kotz, S., and Balakrishnan, N. "Continuous Univariate Distributions, 2nd Edition, Wiley Series in Probability and Mathematical Statistics, 1994

Johnson,R.J. (1980) "Motor Vehicle Size and Weight Regulations, Enforcement, and Permit Operation", NCHRP Synthesis 68, 1980

Kato, Takeaki and Goto, Yuji (1984) "Effect of Water Infiltration of Penetration Deterioration of Bridge Deck Slabs", Transportation Research Record 950, p.202

Kilareski, Walter P., "Heavy Vehicle Evaluation for Overload Permits", Transportation Research Record 1227, Rigid and Flexible Pavement Design and Analysis, Unbound Granular Materials, Tire Pressures, Backcalculation, and Design Methods, p.194.

Kim, K.H., Dominguez, J.M., Klinger, R.E., and Burns, N.H., "Behavior of Ontario-type Bridge Decks on Steel Girders", CRT 3-5-83-350-4F, January 1988.

Kim, Sangjin, Sokolik, Andrej F., and Nowak, Andrzej S., "Measurement of Truck Load on Bridges in Detroit, Michigan, Area", Transportation Research Record 1541, p.58.

Klaiber, F.W., Dunker, K.F., Wipf, T.J., and Sanders, W.W., "Methods of Strengthening Existing Highway Bridges", National Cooperative Highway Research Program Report 293, September 1987.

Kostem, Celal N., "Overloading of Highway Bridges - Initiation of Deck Damage", Transportation Research Record 664, Bridge Engineering Volume 1, 1978, p.207.

Krauss, P.D. and Rogalla,E.A. (1996) " Transverse Cracking in Newly Constructed Bridge Decks", NCHRP Report 380, TRB, 1996

Kuhlman, U., "Steel Bridges, progress in Structural Engineering and Materials", Vol.1 No.1, (Ed.) by Lam, Dennis, September 1997, p.42.

Kumar, Sanjeev V., and GangaRao, Hota V.S., "Fatigue Response of Concrete Decks Reinforced with FRP Rebars", Journal of Structural Engineering, January 1998, p.11.

Laman, Jeffrey A., and Nowak, Andrzej S., "Fatigue Load Spectra for Steel Girder Bridges Research Report", December 1992.

Laman, Jeffrey A., Boothby, Thomas E., and Ashbaugh John R., "Fatigue Impacts on Bridge Cost Allocation and Truck Size and Weight Limits", PTI 9707, Pennsylvania State University, January 1997.

Leidy, Joseph Paul, Lee, Clyde E., and Harrison, Robert, "Measurement and Analysis of Traffic Loads Across the Texas-Mexico Border", FHWA/TX-95+1319-1, March 1995.

Li, Wulin, Albrecht, Pedro, and Saadatmanesh, Hamid, "Strengthening of Composite Steel-Concrete Bridges", Journal of Structural Engineering, December 1995, p.1842.

Lybas, J.M., Hays, C.O., and Guevara, J.O., "Further Tests of the Punching Shear Strength of Lightly Reinforced Isotropic Bridge Decks, Structure and Materials Research Report NO.90-2", Volume 1 of 2, FL/DOT/RMC/0413-2486, State Project No. 99700-7375-119, WPI No. 0510413, UF Project Nos. 4910450420712 and 4910450423512, June 1990.

Mamlouk, Michael S., "A Rational Look at Truck Axle Weight", Transportation Research Board 70th Annual Meeting January 13-17, 1991, Paper No. 910093.

Martin, Tim C., "Estimating Australia's Heavy Vehicle Road Wear Cost Responsibilities(Load Related Road Wear)", Road Transport Technology-4, Proc. of the Fourth International Symposium on Heavy Vehicle Weights and Dimensions, (Ed.) by Winkl, Christopher, 1995, p.67.

Massie, Dawn L, Campbell Kenneth L., and Blower, Daniel F., "Large Truck Travel Estimates from the National Truck Trip Information Survey", Transportation Research Board 72th Annual Meeting January 10-14, 1993, Paper No. 931162A.

Massie, Dawn L., Campbell, Kenneth L., and Blower, Daniel F., "Comparison of Large Truck Travel Estimates from Three Data Sources", Transportation Research Board 72th Annual Meeting January 10-14, 1993, Paper No. 931162B.

Matsui,S. and Muti,K. (1992) "Rating of Lifetime of A Damaged RC Slab and Replacement by Steel Plate-Concrete Composite Deck", Technology Reports of the Osaka University, Vol.42, No.2115, p.329, October, 1992

Matsui,S. (1991) "Fatigue Deterioration Mechanism and Durability of Highway Bridge RC Slabs", Report to Hanshin Expressway Public Corporation, 1991 (in Japanese)

Matsui,S. and Yonhee,K. (1993) "Deterioration and Repair of RC Slabs on Highway Bridges", Proceedings of the Fourth East Asia-Pacific Conference on Structural Engineering & Construction, Seoul, Korea, September 20-22, 1993, p.911

McCarten,P. "Fatigue Life of R.C. bridge Decks", Proce. 8th Annual International Bridge Conference, Jun 10-12, 1991, Pittsburgh, PA, p.303

McGuire, J.F., and Burt, E.J., "Heavy Vehicles Economics Bridges", New Zealand Roading Symposium 1987 Vol. 4.

Miller, Douglas, and Munse, W.H., "Mathematical Modeling of Loading Histories for Steel Beams or Girder Highway Bridges", FHWA/IL/UI-204, March 1983.

Ministry of Transportation of Ontario (1997) "Impact of the Highway Infrastructure of Existing and Alternative Vehicle Configurations and Weight Limits", May 1997

Minnesota DOT (1991) "Truck Weight Limits and Their Impact on Minnesota Bridges", Task Force Report, response to Special Report 225, Truck Weight Limits: Issues and Options, TRB, Sept. 1991

Mitchell, C.G.B, "The Effect of the Design of Goods Vehicle Suspensions on Loads on Roads and Bridges, Research Report 115", Australia Road Research Board Symposium on Heavy Goods Vehicle Suspension Character, Canberra, 23-25 March 1987.

Mohammadi,J., Guralnick,S.A., and Polepeddi,R. "The Effects of Increased Truck Weights Upon Illinois Highway Bridges", Final Report to IDOT, FHWA/IL/RC-013, Illinois Institute of Technology, Sept.1991

Moses,F., Schilling,C.G., and Raju,K.S,. Fatigue Evaluation Procedures for Steel Bridges. NHCRP Report 299, TRB, Washington, DC, Nov. 1987

Moses,F. (1989) "Effects on Bridges of Alternative Truck Configurations and Weights" Draft Final Report to NCHRP - TRB, 1989

Moses,F. (1992) "Assessment of Bridge and Structure Related Costs for An Ohio Truck Weight and Fee System", Final Report to ODOT, June 1992

Natori, Tohru, Fukazawa, Makoto, and Terada, Hiromasa, "Method of Repair for Fatigue Damage in Steel Bridges, Durability of Structures", (Ed.) by Int'l Assoc. For Bridge and Structural Eng., 1989, p.901.

NBR, Prasad, White, Donald W., Ramirez, Julio A., and Kuczek, Thomas, "Statistical Analysis of Overload Vehicle Effects on Indiana Highway Bridges", FHWA/IN/JHRP-93/1, August 1994.

NCHRP, "Concrete Bridge Deck Durability", National Cooperative Highway Research Program Synthesis of Highway Practice 4, 1970.

Nishikawa, Kazuhiro, Murakoshi, Jun, and Matsuki, Takayuki, "Study on the Fatigue of Steel Highway Bridges in Japan", Construction and Building Materials, Vol. 12, No. 2-3, 1998, p.133.

Nix, F.P., and Boucher, M., "Long Combination Vehicles in Quebec and Ontario", Transportation research Forum, Vol.29, No.2, 1989, p.301.

Nix, Fred P., Billing, John R., and Delaquis, Michele, "Impact of Size and Weight Regulation on Trucks Crossing the Canada-United States Border", Transportation Research Board 77th Annual Meeting January 11-15, 1998, Paper No. 980247.

Nowak,A.S. "Calibration of LRFD Bridge Design Code – NCHRP Project 12-33", University of Michigan, Ann Arbor, Michigan, 1992

OCDE (1982) "Impacts of Heavy Freight Vehicles" Dec.1982

O'Connor,D.S. and Hyman,W.A. (1989) "Bridge Management Systems", FHWA-DP-71-01R, Oct.1989

Okada,K., Okamura,H., and Sonoda,K. (1978) "Fatigue Failure Mechanism of Reinforced Concrete Deck Slabs", Transportation Research record 664, p.136.

Perdikaris,P.C. Petrou,M.F., and Wang,A. (1993) "Fatigue Strength and Stiffness of Reinforced Concrete Bridge Decks", Final Report to ODOT, FHWA/OH-93/016, March 1993, department of Civil Engineering, Case Western Reserve University, Cleveland, OH

Perdikaris, Philip C., and Beim, Sergio R., "Design of Concrete Bridge Decks", Research report No. FHWA/ OH-88/004, August 1988.

Perdikaris, Philip C., and Beim, Sergio, "RC Bridge Decks under Pulsating and Moving Load", Journal of Structural Engineering, Vol. 114, No. 3, March 1988, p.591.

Perdikaris, Philip C., and Petrou, Michael, "Code Predictions Versus Small Scale Bridge Deck Model Test Measurements", Transportation Research Record 1290, 1991, p.179.

Petrou, Michael F., and Perdikaris, Philip C., "Punching Shear Failure in Concrete Decks as Snap-Through Instability", Journal of Structural Engineering, September 1996, p.998.

Petrou, Michael F., Perdikaris, Philip C., and Duan, Mingzhu, "Static Behavior of Non-composite Concrete Bridge Decks under Concentrated Loads", Journal of Bridge Engineering, November 1996. p.143.

Petrou, Michael F., Perdikaris, Philip C., and Wang, Aidong, "Fatigue Behavior of Non-composite Reinforced Concrete Bridge Deck Models", Transportation Research record 1460, p.73, 1994

Puckett J.A., Naiknavare, R.D., and Lohrer, J.D., "Evaluation of Bridge Deck Utilizing Ontario Bridge Deck Design Method Vol. No.1 Review of Experimental and analytical Research in Lightly Reinforced Bridge Decks-Final Report", Project No. 38717, FHWA-WY-89-002, July 1989.

Ricketts, N.J., and Page, J., "Traffic Data for Highway Bridge Loading", TRL Report 251.

RJHansen Associates (1979) "State Laws and Regulations on Truck Size and Weight", NCHRP Report 198, Feb.1979

Roeder, Charles W., Hildahl, Mark, and Lund, Van John A., "Fatigue Cracking in Modular Bridge Expansion Joints", Transportation Research Record 1460, p.87.

Roy Jorgensen and Associates (1969) "Oversize-Overweight Permit Operation on State Highways", NCHRP Report 80, 1969

Rudra, Taposh "Written History" http://www.marion.lib.in.us/history/indtrucks/report.html.

Sahli, Adnan H., Albrecht, Petro, and Vannoy, Donald W., "Fatigue Strength of Retrofitted Cover Plates", Journal of Structural Engineering, Vol. 110, No. 6, June 1984, p.1374.

Sakano, M., Mikami, I., and Miyagawa, K., (1993) "Simultaneous Loading Effect of Plural Vehicles on Fatigue Damage of Highway Bridges, Reliability and Optimization of Structural Systems", V (B-12), (Ed.) by Thoft-Christensen, P., and Ishikawa, H., 1993, p.221.

Saklas,J. (1998) Background Material for Previous FHWA Studies.

Saklas, James G., Albrecht, Pedro, Sidani, Maher, and Schelling, David R., "Bridge Fatigue Damage Cost Allocation", DTFH61-85-R-00157, 1988

Sanders, David H., and Zhang, Yuqing Jane, "Bridge Deterioration Models for States with Small Bridge Inventories", Transportation Research Record 1442, p.101.

Scanlon,A. abd Saklas,J. (1996) "Bridge Cost Allocation: Current Issues" TRB Presentation Preprint, Jan. 1996

Schelling,D.R. (1985) "The Influence of Heavy Trucks on the Overload and allocation of User Costs on Bridge Structures", University of Maryland, March 1985

Schelling, David R., "Effects of Maryland Legal Vehicle Weight Limits on Highway and Bridges", FHWA/MD-81/1, February 1981.

Schilling, C.G., Klippstein, K.H., Barsom, J.M., and Blake G.T., "Fatigue of Welded Steel Bridge Members under Variable-Amplitude Loadings", National Cooperative Highway Research Program Report 188, 1978.

Schilling, Charlies G., "Variable Amplitude Load Fatigue Task A-Literature Review Volume I-Traffic Loading and Bridge Response", FHWA-RD-87-059, July 1990.

Scottish Development Department, "Damage to Bridges by Road Vehicles Traffic Signs at Bridges", ISBN 0 7480 0164 6.

Snyder,R.E., Likins,G.E. and Moses,F. (1985) "Loading spectrum Experienced by Bridge Structures", FHWA/RD-85/012, Feb.1985

Sorensen, Harold C., and Robledo, Francisco Manzo, "Turner Track Impact on Bridges", WA-RD 287.1, December 1992.

Stephens, J.E., Scoles, J., Patterson, S., and Schillings, P., "Impact of Adopting Canadian Interprovincial and Canamex Limits on Vehicle Size and Weight on the Montana State Highway System", FHWA/MT-96/8113, October 1996.

Tawfiq, Kamal, "Fatigue Fracture in Concrete-Final Report-Volume II", Project # 99700-7582-119, WPI 0510623, Contract # C-4068, Fl/DOT/RMC/0623-4068, March 1994.

Tanihira,T., Sonoda,K., Horikawa,T., and Kitoh,H. "Low Cycle Fatigue Characteristics of Bridge Deck RC Slabs under the Repetition of Wheel Loads", Proc. Pacific Concrete Conference, New Zealand, Nov. 8-11, 1988, p.381

Tawfiq, Kamal, "Fatigue Fracture in Concrete-Final Report-Volume I", Project # 99700-7582-119, WPI 0510623, Contract # C-4068, Fl/DOT/RMC/0623-4068, March 1994.

Terrell, Ronald L., and Bell, Chris A., "Effects of Permit and Illegal Overloads on Pavements", National Cooperative Highway

Research Program Synthesis of Highway Practice 131, September 1987.
Thompson, Paul D., and Markow, Michael J., “Collecting and Managing Cost Data for Bridge Management Systems”, National Cooperative Highway Research Program Synthesis of Highway Practice 227, 1996
TRB (1990a) “Truck Weight Limits: Issues and Options”, Special Report 225, 1990
TRB (1990b) “New Trucks for Greater Productivity and Less Road Wear - An Evaluation of the Turner Proposal”, Special Report 227, 1990
TRB (1980), “Motor Vehicle Size and Weight Regulations, Enforcement, and Permit Operations”, National Cooperative Highway Research Program Synthesis of Highway Practice 68, April 1980.
Turner, Richard K., “The operators’ View of Emerging Size and Weight Policy in Europe”, Road Transport Technology-4, Proc. of the Fourth International Symposium on Heavy Vehicle Weights and Dimensions, (Ed.) by Winkl, Christopher, 1995, p.15.
The Urban Institute, “Pavement and Bridge Impacts of Longer Combination Vehicles”, 91-20075, June 1990.
USDOT “The Intermodal Transportation and Inventory Cost Model Documentation”, April 1999
USDOT (1998) “Comprehensive Truck Size and Weight Study”, Draft Volume III, Scenario Analysis, Dec.30, 1998
USDOT Secretary (1991a) “Overweight Vehicles - Penalties & Permits, An Inventory of State Practices for Fiscal Year 1989”, March 1991
USDOT Secretary (1991b) “The Status of the Nation’s Highways and Bridges: Condition and Performance”, Report to Congress, Sept. 1991
USGAO, “Commercial Trucking Safety and Infrastructure Issues under the North American Free Trade Agreement”, GAO/RCED-96-61, February 1996.
USGAO (1979) “Excessive Truck weight: An Expensive Burden We Can No Longer Support”, Report to Congress, July 16, 1979
Walton,C.M., Yu,Chien-Pei, and Ng, Paul (1983) “Procedure for Assessing Truck Weight Shifts that Result from Changes in Legal Limits”, Transportation Research Record 920, p.19.
Weissman, Jose, and Harrison, Rob, “Impact of 44,000-kg(97,000-lb) Six-Axle Semitrailer Trucks on Bridges on Rural and Urban U.S. Interstate System”, Transportation Research Record 1624, Paper No. 98-0481.
Weissman, Jose, and Harrison, Rob, “Impact of Turnpike Doubles and Triple 28s on the Rural Interstate Bridge Network”, Transportation Research Record, p.32.
Weissman, Jose, and Harrison, Rob, “Increasing U.S Truck Size and Weight Regulation under NAFTA”, Journal of the Transportation Research Forum Vol. 37, No. 1, p.1.
Weissman, Jose, Reed, Robert L., and Feroze, Ahmed, “Incremental Bridge Construction Costs for Highway Cost Allocation”, Transportation Research Record 1460, p.19.
Weyers,R.E., Fitch,M.G., Larsen,E.P., and Al-Qadi,I.L. “Concrete Bridge Protection and Rehabilitation: Chemical and Physical Techniques”, SHRP-S-668, National Research Council, 1994
Weyers,R.E., Prowell,B.D., Sprinkel,M.M., and Vorster,M. “Concrete Bridge Protection, Repair, and Rehabilitation relative to Reinforcement Corrosion: A Methods Application Manual”, SHRP-S-360, National Research Council, 1993
Weyers, Richard E., and Cady, Philip D., “Deterioration of Concrete Bridge Decks from Corrosion of Reinforcing Steel, Concrete International: Design & construction” Vol.9#1 January 1987, p.15.
Whiteside, R.E., Chu,T.Y., Cosby,J.C., Whitaker,R.L., and Winfrey,R. (1973) “Changes in Legal Vehicle Weights and Dimensions: Some Economic Effects on Highways”, NCHRP Report 141, Highway Research Board, 1973
Wojakowski, John, and Hossain, Mustaque, “Twenty-Five –Year History of Interlayer Membranes on Bridge Decks in Kansas”, Transportation Research Record 1476, p.180.
Yen, Ben T., Huang, Ti, Lai, Lung-Yang, and Fisher, John W., “Manual for Inspecting Bridges for Fatigue Damage Conditions”, FHWA-PA-89-022 +85-02, January 1990.
Yoder, Eldon J., Colucci-Rios, Benjamin, Fraczek, John, and Skees, James A. (1979) “Effects of Raising Load Limits on Pavements and Bridges in Indiana”, Joint Highway Research Project, IN/JHRP-79-23, December 1979.
Yu, Chien-Pei, and Walton, C. Michael (1982) “Estimating Vehicle Weight Distribution Shifts Resulting from Changes in Size and Weight Laws”, Transportation Research Record 828, 1982, p.16.
Zaghloul, Sameh, and White, Thomas, “Effect of Overloaded Vehicles on the Indiana Highway Network”, FHWA/IN/JHRP-93/5, June 1994.
Zwerneman, Farrel J., “Fatigue Assessment of Bridge Members Based on in-Service Stresses-Final Report: Executive Summary”, FHWA\OK-97(06), July 1997.
Zwerneman, Farrel J., Kelly, John W., and Johnson, Orin A., “Fatigue Assessment of Bridge Members Based on In-Service Stresses”, FHWA\OK-95(05), June 1997.
Zwerneman, Farrel J., Poynter, Paul G., Abbas, Michael D., Rauf, Abdul, and Yang, Jie, “Fatigue Assessment of Bridge Members Based on In-Service Stresses”, FHWA\OK-95(07), August 1996.
Zwerneman, Farrel J., West, Adam B., and Lim, Kee Seong, “Fatigue Damage to Steel Bridge Diaphragms”, FHWA/OK 89(10), December 1989.

*Unedited Bibliography

APPENDIX B TWO APPLICATION EXAMPLES OF THE RECOMMENDED METHODOLOGY FOR ESTIMATING BRIDGE NETWORK COSTS DUE TO TRUCK WEIGHT LIMIT CHANGES

Two examples are presented in this appendix for illustrating application of the methodology in Appendix A. The first example is an application to two routes in Idaho, and the second one to the entire state of Michigan. The levels of detail for these examples were selected with consideration to the time and cost constraints for the project. They may or may not represent the same levels of detail that the respective agencies would take when applying the methodology. In reality, this level of detail is determined based on available resources for the task. However, these examples illustrate typical steps that would be taken to accomplish the task of estimating bridge network costs resulting from truck weight limit changes. These steps follow the procedure presented in Appendix A, using some of the default data included there. More background information for the recommended methodology is presented in Chapter 3 of this report.

B-1. The Idaho Example

This example is to estimate the cost impact for highway bridges in the State of Idaho due to a weight limit increase for permit loads. This change includes lifting the cap on the bridge formula from 467 kN (105 kips) to 574 kN (129 kips). Note that the bridge formula is still effective with the up limit increased.

The following three scenarios of impact extent have been investigated in this example. (1) The cost impact for the bridges on two routes under the experimented regulation (i.e., the new regulation is applicable to only these two routes). These two routes are indicated in Fig. B-1.1.1. An annual permit is needed for trucking under this new regulation. This regulation has been experimented on the two routes from July 1998 to June 2000. It is referred to as the pilot project, and the routes as the pilot routes hereafter in this report. (2) The cost impact on the same bridges under the same regulation but hypothetically applicable to the entire state. It is assumed that the permit regulation also applies. (3) The cost impact on the same bridges under a regulation legalizing the new cap of 574 kN (129 kips). These scenarios are respectively referred to as Scenarios 1, 2, and 3 in this appendix.

For Scenario 1, the weight limit change is effective only for the two routes. Thus, most likely not all truckers would take advantage of it, because it requires some investment in new vehicles for those truckers who do not have the permissible trucks with more axles and longer axle distances. In addition, the relevant state legislation had a clearly defined two-year effective period. It can be therefore challenging to fully predict the effect of this weight limit change, because only a section of the trucking industry responded to the new and short-lived regulation. Since weigh-in-motion (WIM) data have been collected from on of the routes before and after the implementation of the weight limit change, these data are used below for cost estimation for this scenario.

Scenario 2 assumes that the considered weight limit change is implemented in the entire state, although only those bridges on the pilot routes are of interest for cost estimation. Thus, it is assumed that the effect of this new weight limit is fully developed. Shifting of truck traffic is assumed to be as follows. 1) The traffic of type CT5 trucks with two boxes will be subject to shifting to triples TRP. 2) The traffic of type DS7 will still stay in the same type, but with its GVW limit increased to 512 kN (115 kips). 3) The traffic of type DS8 will be subject to shifting to a vehicle type shown in Fig. B-1.3.1, whose up limit is 574 kN (129 kips). The Idaho Transportation Department (ITD) developed this vehicle and the other in the same figure. These vehicles were envisioned to be the practical maximum loads at the two GVW levels under the new regulation.

For Scenario 3, it is assumed that the traffic of 5-axle truck types (Types 3S2T and 3S2S according to the FHWA definition) subject to current legal weight limit of 80 kips will change their behavior and shift to truck types that allow heavier GVW under the new regulation. This assumption is based on the scenario definition that the 129 kip limit will be legalized statewide. In addition, the shifting of truck types included in Scenario 2

is also assumed to take place here in this scenario, because Scenario 3 actually encompasses Scenario 2.

Since it is not absolutely certain which new truck types will receive the traffic of the 5 axle trucks (3S2T and 3S2S), the Scenario 3 analysis uses two extreme cases to provide an upper and a lower bound for the impact costs. The first extreme case assumes that 5-axle trucks will shift to a 9-axle configuration with a maximum GVW of 467 kN (105 kips), as shown in Fig. B-1.3.1. This is the current maximum GVW for permit load, which has been available. If a 5-axle truck does not take that option in current operation, it would at least take this option under the new regulation of Scenario 3 being considered here, provided that the truck traffic will indeed shift. Thus, this truck load is considered the lower bound for shifted truck loads. The other extreme case assumes that the 5-axle trucks will shift to a 10-axle configuration with a maximum GVW of 574 kN (129 kips), also shown in Fig. B-1.3.1. This truck load represents an upper bound for new truck loads, because this is the highest load allowed in the considered scenario of new regulation. Both truck models were provided by ITD, as a result of their effort to identify the impact of the considered truck weight limit change.

In this example, a 20-year planning period is assumed (PP=20 years). Namely, impacts (such as fatigue failure) will be included in the estimation only if they are expected to take place within 20 years from 1998 when Scenario 1 started to be effective. Namely, the Base Case (BC) refers to 1998 before Scenario 1 was implemented and the Alternative Scenario (AS) is the condition after the considered truck weight limit change is effective.

It may be of interest to note that a 20-year period is typically used in transportation planning. The planning period should be a user-input in application of the recommended methodology. It is perhaps important to realize that significantly longer planning periods may not be recommended because uncertainty associated with the input data could become large and thereby adversely affect the reliability of the final result. For example, such uncertainty may be associated with predicting the growth of economy that affects the truck traffic volume.

B-1.1 Cost Impact Category 1: Steel Fatigue

1. Identify possibly vulnerable (possibly impacted) bridges

For this cost impact category, it is possible to perform a Level II analysis, because the number of bridges to be impacted is not very large. Using the database of ITD, a total of 7 steel bridges (besides 2 steel culverts) were identified to be on the two pilot routes shown in Fig. B-1.1.1, considered to be possibly vulnerable. Vulnerable bridges are defined here as those that may have E and E' fatigue category details. (Note that agencies may add other fatigue categories when applying the recommended methodology, if adequate information is available to enable reasonable estimation for the

remaining lives under the Base Case and the Alternative Scenario and associated cost impacts.) The identified bridges for this category are listed below.

ID	Span type	Structure Type	Year Built
12560	Continuous	Rolled beams	1980
12565	Continuous	Rolled beams	1980
13705	Simple	Truss	1949
14520	Continuous	Plate girders	1966
15220	Simple	Rolled beams	1956
16641	Continuous	Plate girders	1995
17570	Simple	Rolled beams	1956

Using available plans, each of these bridges was examined to identify E and E' details. As a result, only one bridge (No.14520) was identified to have fatigue prone details of category E --- horizontal longitudinal web stiffeners ending at a transverse stiffener and close to the bottom flange. The other rolled beam bridges have several welded connections with stronger fatigue strength than E and E'. It also appears that the recently built plate-girder bridge (No.16641 built in 1995) has been carefully designed in avoiding details with low fatigue strength. (Note that once this bridge-by-bridge examination is complete for the network, the results can be used in future application analyses for other Alternative Scenarios. In other words, this identification of impacted bridges need to be done only once for a network. Also note that this indeed has been done in some state agencies in the country, such as the New York State Department of Transportation.)

2. Estimate the truck weight histograms and truck volumes for both the Base Case and the Alternative Scenarios for the impacted bridge.

Scenario 1: Practiced situation of lifted GVW limit applicable to only the pilot routes

Note that this scenario was actually implemented. It is a situation where the weight limit is increased for a small group of large and heavy trucks (annual permits) on two specific routes. Legally, the trucks hauling the newly permitted overweight may be used for other trips on other routes at legal or permit weights, provided that they meet other applicable requirements. For example, a triple truck can be easily changed to a double and haul a different legal weight which does not need a permit. This kind of situation can make it difficult to predict trucking behavior.

Nevertheless, WIM data were found available for the Base Case (before the weight limit change) and the Alternative Scenario (after the weight limit change) for an impacted site. The WIM station happens to be right on one of the routes and about 25 miles away from the impacted bridge, as shown in Fig. B-1.1.2.

Scenario 2: Hypothetical situation of lifted GVW limit applicable to entire state

The Base Case TWH for Scenario 2 is generated using the FHWA VMT data (recommended as the default Base Case data whose sample is shown in Data Set A-5.2.1 of Appendix A). The load shifting method presented in Section A-5.1.1 of Appendix A is then applied. The equivalent AASHTO fatigue truck weight W_{eqv} (according to the *AASHTO Guide Specs for Fatigue Evaluation of Existing Steel Bridges*, 1990) was calculated as follows for both Scenarios 1 and 2.

	W_{eqv} in kN(kips) Scenario 1 (practiced)	W_{eqv} in kN(kips) Scenario 2 (129 kip-cap for entire state)
Base Case	302(67.8)	317(71.3)
Alternative Scenario	313(70.4)	325(73.0)

Note that the WIM data used for the Bases Case of Scenario 1 include 8,651 trucks recorded in October 1997. Those for its Alternative Scenario include 10,170 and 14,323 trucks respectively recorded in October of 1998 and 1999 after the weight limit change was implemented. W_{eqv} for the Scenario 1's Alternative Scenario (313 kN or 70.4 kips) is the average of two W_{eqv}'s respectively based on the two TWH for 1998 and 1999 (315 and 313 kN or 70.9 and 69.9 kips). Fig. B-1.1.3 shows the TWH for these years for comparison. It is seen that 1998 and 1999 data show increase of heavier truck traffic. To display this increase more clearly, Fig. B-1.1.4 shows percentage changes in the 1998 and 1999 data from the 1997 data. When the 1997 data are zero for weights higher than 800 kN (180 kips), the percentage change is supposed to be infinity but is replaced by 2000% for indication. It is seen that, in the range of 700 to 800 kN (157 to 180 kips), heavy truck traffic has been increased at least by 250%. Apparently much heavier trucks were traveling on these routes, as a result of the truck weight limit change.

It is also interesting to note that in the area of GVW 350 to 525 kN (79 to 118 kips) in Fig. B-1.1.3, the 1998 and 1999 truck traffic has been reduced from the 1997 level. This is due to the effect of load "shifting" from lighter trucks to heavier trucks. Thus, more heavier trucks and fewer lighter trucks traveled on these routes when the new regulation was experimented. The mid-point of this weight range is 437.5 kN (98 kips), which is close to the original GVW cap of 467 kN (105 kips). It is also where the percentage decrease (negative change) reaches the maximum (-48% and –24% respectively for 1999 and 1998), as shown in Fig. B-1.1.4. It appears that Scenario 1 of lifting GVW limit from 467 kN (105 kips) to 574 kN (129 kips) has effected a well observable increase in truck operating weight.

More importantly, this observed truck-weight increase is not consistent with the fact that only three trucking companies have been issued special permit under the pilot project (i.e., the new regulation). These companies registered for 8 vehicles --- 4 for GVW of 516 kN (115 kips) and 4 for 574 kN (129 kips). According to their travel logs filed with ITD as required, these permit trucks have not traveled extensively. Actually, one of these trucks has traveled only about 500 loaded miles from July 1998 to end of

1999, according to the filed record and the research contractor's conversation with the trucker.

Scenario 3: Hypothetical situation of lifted GVW limit legalized in entire state

The Base Case TWH for Scenario 3 is the same as that for Scenario 2, generated using the FHWA VMT data. The same load shifting method was used here, as presented in Section A-5.1.1 of Appendix A. The equivalent AASHTO fatigue truck weight W_{eqv} (according to the *AASHTO Manual for Condition Evaluation of Bridges,* 1994) was calculated as follows for both the lower bound and the upper bound cases. They show a clear increase in the fatigue load.

	W_{eqv} in kN(kips) Scenario 3 lower bound (105 kip truck)	W_{eqv} in kN(kips) Scenario 3 upper bound (129 kip truck)
Base Case	317(71.3)	317(71.3)
Alternative Scenario	350(78.7)	373(83.8)

3. Estimate the mean and safe remaining lives for both the Base Case and the Alternative Scenario.

It is found that the infinite life requirement of the AASHTO specs is not met. Thus, finite remaining lives are calculated as shown in Table B-1.1.1. It should be noted that the AASHTO estimation method for the total number of stress cycles represents an approximation. It may lead to both over- and under-estimates, as discussed in Section 3.2. The above results were obtained using an explicit approach presented in Section 3.3 of this report, instead of the AASHTO approach. The calculation is based on a traffic growth rate of 1.95%, which is estimated using current AADT and future AADT recorded in the NBI. These long remaining lives are mainly due to very low truck traffic volume (ADTT=488 for 2-way and 2-lane), using data provided by the NBI as the default database.

4. Select responding action and estimate costs

Note that a 20-year planning period (PP=20 years) has been selected. Thus, only impacts within 20 years are included to contribute to the total impact costs. Based on these remaining lives, no preventive measure (i.e., do-nothing) appears to be justifiable. It results in zero cost for this cost impact category.

However, for illustration purpose, Eq. 3.33.3 is used here to estimate the expected impact costs as follows for repair cost:

Expected Impact Costs = Impact Costs ($P_{f,AS} – P_{f,BC}$) ≥ 0

(3.3.3.3)

For Scenario 1:

Expected Impact Costs =Repair Costs ($P_{f,AS}$ - $P_{f,BC}$)
=\$1360($6.03x10^{-13}$-$6.81x10^{-14}$)
=\$0.000

For Scenario 2:

Expected Impact Costs =Repair Costs ($P_{f,AS}$ - $P_{f,BC}$)
=\$1360($1.31x10^{-11}$-$2.58x10^{-12}$)
=\$0.000

For Scenario 3 (lower bound – 5 axle truck shifting to 105 k):

Expected Impact Costs =Repair Costs ($P_{f,AS}$ - $P_{f,BC}$)
=\$1360($1.55x10^{-10}$-$42.58x10^{-12}$)
=\$0.000

For Scenario 3 (upper bound – 5 axle trucks shifting to 129 k):

Expected Impact Costs =Repair Costs ($P_{f,AS}$ - $P_{f,BC}$)
=\$1360($1.08x10^{-8}$-$2.58x10^{-12}$)
=\$0.000

The unit repair cost of \$1360 is taken from Data Set A-5.2.4 in Appendix A for repairing fatigue cracking for a longitudinal stiffener's end weld. The resulting cost impact for this category appears to be negligible for all three scenarios as shown.

5. Sensitivity analysis

Based on the results above, it appears that possible variations in the input data likely will not change the fundamental nature of the result. In other words, the expected costs due to the three scenarios of weight-limit change will be negligible or practically zero. For illustration, Scenario 2 is used here for a sensitivity analysis according to the guidelines given in the recommended methodology in Appendix A.

The sensitivity analysis results are obtained with the shifting window parameter and impact factor perturbed, as shown in Tables B-1.1.2 and B-1.1.3. Table B-1.1.2 shows the impact cost as a function of the shifting window parameters, compared with the reference case of a_1 = a_2 = 10% and b_1 =b_2 =20%. The definition of the window parameters is given in Fig. 3.1. For the lower and upper bounds for the window

parameters, Table B-1.1.2 shows that the cost impact may change negligibly from the reference case (i.e., the default values of the window parameters). In a similar fashion, Table B-1.1.3 demonstrates the sensitivity of the dynamic impact factor on the expected cost impact. While for this particular example the influence of the impact factor and the window parameters is not noticeable, the impact cost is more sensitive to the dynamic impact factor than to the shifting window parameters. This appears to be because the dynamic impact factor is part of the stress range as loading. The load is raised to the cubic power and is more dominant than many other parameters. Although the shifting window parameters are also related to loading, they are not as directly as the impact factor. Thus, less sensitivity is observed of these parameters.

6. Conclusions for this cost impact category

Based on the details presented above, it can be concluded that the lifted GVW cap from 467 kN (105 kips) to 574 kN (129 kips) for the three considered scenarios is not expected to change costs to the state agency within next 20 years, with respect to steel bridge fatigue. This is mainly because of very low current (and thus future) truck traffic. It should also be noted that no transportation modal shifting (e.g., from railway to highway or vice versa) is included in the presented example. The two specific routes are local routes transporting goods within the area. Thus truck behavior change as a result of these scenarios is not expected to have an impact on railway transportation.

B-1.2 Cost Impact Category 2: RC Deck Fatigue

1. Identify vulnerable bridges and group

Using the ITD's bridge inventory, a total of 73 bridges were identified on the two pilot routes. Of these bridges, 39 were identified to have an RC deck-on-beam superstructure. The agency's inventory contains more details than the NBI, which was helpful in identifying these bridges according to their characteristics. A Level I analysis is performed here, because a Level II analysis would require an excessive amount of detailed analysis for each of the 39 bridges. These 39 bridges were grouped and for each group one bridge was selected for detailed analysis. Following the guidelines discussed in Section 3.4, the major factor considered here in grouping is the age, i.e., the year when the bridge was constructed. This also largely determines the deck's ultimate shear strength P_u. It is because P_u is mainly a function of the concrete's compressive strength and deck thickness, and bridges built in the same time period likely have similar values for these parameters. Tables B-1.2.1 provides an overview of the groups selected, according to age, i.e., the year built.

2. Generate wheel weight histograms (WWHs)

For generating WWHs, the Scenario 1 analysis used the measured WIM data to generate the WWHs. In this case, wheel weights are available in the WIM data by dividing the recorded axle weights by 2. The procedure of using the axle and gross weight relations in Section A-5.1.2 of Appendix A was not needed. Figs. B-1.2.1 and B-1.2.2 show the WWHs as the result respectively for the Base Case and the Alternative Scenario. They show that the heaviest wheels weigh 80 to 85 kN (18 to 19 kips). These wheel loads include the effective load increase for steering wheels that act on a reduced area of tire-print, discussed in Section 3.4. Scenarios 2 and 3 used the FHWA VMT data and the recommended procedure in Section A-5.1.2 of Appendix A to estimate wheel weights using the gross vehicle weight. Then WWHs were generated using these results.

3. Find increased probability of failure, select responding action, and estimate impact costs for sampled bridges

The increased probability of failure P_f is calculated according to Eq. 3.3.2.4 and also listed in Tables B-1.2.1, B-1.2.2, and B-1.2.3 for each bridge deck representing a group of decks, respectively for Scenarios 1, 2, and 3. The ITD is overlaying decks for the first time of their respective service life, and is gradually increasing this treatment in the State. Thus, overlaying costs would be most likely the costs for these bridges and they are therefore used here to estimate the cost impact. A $19 per square foot (in Y1998 dollars) is estimated as the unit cost for this selected treatment. This cost was estimated based on a $20 per square foot in Y2000 dollars, based on ITD personnel’s experience. The total costs are in Y1998 dollars shown in Tables B-1.2.1 to 1.2.3 respectively for Scenarios 1, 2, and 3.

For Scenario 3, Table B-1.2.3 shows estimated zero costs for both cases of assumed truck load shifting. Numerically, this is due to that the probability of failure increased by the weight limit change is always zero for all the bridges in this scenario. Actually the failure probabilities in AS was lowered from those in BC. Physically, this is due to the new distribution of GVW to a larger number of axles. A 5-axle truck of 80 kips now is shifted to either a 9-axle truck of 105 kips (the lower bound) or a 10-axle truck of 129 kips (the upper bound). Roughly, an increase of GVW of 25 kips or 49 kips is now distributed to 4 or 5 additional axles respectively for the two extreme bounds. On average, each new axle takes 25 kips/4 = 6.25 kips or 49 kips/5 = 9.8 kips. These axle loads under the Alternative Scenario certainly are not higher than those highest axle loads in the Base Case, while fatigue damage is dominated by the highest wheel loads in the spectrum due to a high exponent of almost 18 (Eq. 3.4.2.1). Thus, the Alternative Scenario here actually represents a less severe load to RC decks.

Please also note that this analysis is based on an implicit assumption that wheel loads under the Alternative Scenario would not exceed current wheel weight limit any differently from current practice. At this point, there is no way of verifying or rejecting this assumption. This is especially true for Scenario 3 of extensive scale (i.e., legalizing a new load limit over the entire state) and large magnitude (from 80 kips to 129 kips). We believe that reliable verification or rejection of this assumption can only be done using

data obtained in a process of implementing the considered scenario. This process can be staged to avoid drastic changes that may cause adverse effects on the bridges.

4. Sensitivity analysis

In Section 3.4, the sensitivity of the parameters involved in RC deck fatigue assessment has been examined. It is shown that the terms with the exponent of 17.95 are more sensitive than other parameters. Here we examine another variable related to decision making for treatment.

In practice, patching may be used before rehabilitation of concrete overlay, to "buy" some time for lower dynamic load effect and to immediately address the need for improving riding condition. As commented in Chapter 3, this would not change the structural strength of the deck but it does improve the riding surface condition. We may assume two times of patching for a cost of 10% of the concrete overlay cost.

This estimate of 10% increase from the concrete overlay cost for patching was made based on limited data from California and Arizona. It should be acknowledged that this figure varies over the country. In many states, minor patching is done by in-house maintenance force. It could be difficult to differentiate patching costs from other costs because these personnel have other duties. However, agencies should gather relevant data for more accurate results.

5. Conclusion for this cost impact category

The above results show that expected cost impact due to the increased weight limit to 129 kips for these are $37,100, $10,700, and $0 respectively for Scenarios 1, 2, and 3. There may be a 10 percent increase from these costs for 2 times of additional patching, depending on the agency's practice.

<u>B-1.3</u> Cost Impact Category 3: Deficiency of Existing Bridges

1. Deficiency criterion

For this cost impact category, Level II analysis requires an approach of detailed re-rating for every bridge in the network, except for those already deficient. It needs detailed structural analysis for these bridges under the Alternative Scenario. When Virtis is fully loaded and operational for the agency, Level II analysis may become accomplishable. Otherwise, needed information has to be gathered from different databases (such as plans and the bridge inventory database). Level I analysis, instead, uses current ratings available in the agency's bridge inventory (or the NBI) to estimate new ratings under the Alternative Scenario. Nevertheless, with assistance from ITD, a Level II analysis was made possible and presented here.

All the identified 73 bridges on the two pilot routes are listed in Table B-1.3.1. Also listed are the inventory and operating ratings for these bridges in the second through fifth columns. A rating factor greater than 1.0 indicates an adequate bridge. These ratings are obtained using the AASHTO Type 3S2 and Type 3-3 vehicles, as indicated. Note that a five-percent allowance is included when determining the bridge's need in terms of repair or replacement. This is more justifiable when the operating rating is acceptable (>1) and only the inventory rating is of concern, because the operating rating should be the absolute maximum allowable load. For example, Bridge 12585 is considered to be inadequate under the Base Case, with the inventory rating factors equal to 0.890 and 0.867 (and operating ratings of 1.484 and 1.445) for Type 3S2 and Type 3-3 vehicles, respectively. For comparison, Bridge 12654 is adequate since the rating factors are all higher than 1.0. Nevertheless, Bridge 13190 is also considered practically to be acceptable to remain in service without needing any work, because its inventory rating factors are 0.992 and 0.969 for the two vehicle types, within five percent from 1.0. Its operating ratings are 1.674 and 1.634 respectively for these two rating vehicles.

It was felt that using ratings based on these more realistic vehicles (as opposed to the H and HS vehicles) is appropriate in determining the need for addressing overstress, although there are States that include the H and HS vehicle ratings in their decision making for overstress and determining bridge needs.

According to the concept presented in Section 3.5, the Alternative Scenarios considered here (Scenarios 1, 2, and 3) are applicable to annual renewable permits or legal weights. Thus the following adjusting factor (AF_{rating}) is used, calculated for the respective truck weight histograms discussed above:

$$AF_{rating} = [2W_{AS}* + 1.41\ t(ADTT_{AS})\ \sigma_{AS}*] / [2W_{BC}* + 1.41\ t(ADTT_{BC})\ \sigma_{BC}*]$$

Eq.3.5.1.2 in Section 3.5

where W* and σ* are the mean and standard deviation of truck weight for the top 20 percent of the truck weight histogram (TWH). The coefficient t(ADTT) is a function of truck traffic ADTT at the site. Typically t is in the range of 2.0 to 4.5, as discussed in Section 3.5.3 and tabulated in Section A-3 of Appendix A. The subscripts BC and AS respectively refer to the Base Case and Alternative Scenario. The evaluation truck model is taken as the practical maximum load of 574 kN (129 kips) developed by the ITD as shown in Fig. B-1.3.1. Note that, developing such a truck model often requires considerations to factors other than truck weight. For example, it may depend on the dimensions of anticipated trucks that the designated roadways have to be able to accommodate. Thus, assistance may be needed from personnel with experience in commercial vehicle traffic and regulations.

2. Calculate the load ratings under Alternative Scenario and identify deficient bridges

For Scenario 1, practiced situation of lifted GVW limit applicable to only two routes, the same WIM data used for above cost impact categories are used. The WIM station happens to be on one of the two routes. For Scenario 2 and (hypothetical situation of lifted GVW limit applicable to the entire state) and Scenario 3 (hypothetical situation of legalization of 129 kips), the TWH's for the Base Case and Alternative Scenario are generated using the same procedure as that used for other cost impact categories above. The process of truck traffic shifting has been discussed above for these two scenarios.

The AF_{rating} values as discussed above are then used to find the new rating factors under the alternative Scenarios using the following equation

$$RF_{AS} = RF_{BC,\ \text{using AS rating vehicle}} / AF_{rating} \qquad (3.5.1.3)$$

where $RF_{BC,\ \text{using AS rating vehicle}}$ is the rating factor using the live load factor for the Base Case and the new rating vehicle in Fig.B-1.3.1 for the Alternative Scenario. This rating factor was calculated for each bridge's critical section in the network. The resulting rating factor for the Alternative Scenario RF_{AS} for each bridge is shown in Table B-1.3.1. Note that in these eight columns of rating factor under Alternative Scenario, NA is used to indicate those bridges that are deficient under the Base Case. Thus these bridges do not contribute to the total impact cost. The additional deficient bridges under the Alternative Scenario are indicated by rating factors lower than 0.95, considering an allowance of 5 percent as sometimes practiced.

Here, we will use the operating rating for making decisions as to which bridges need work for impact cost estimation. In Table B-1.3.1, new inadequate bridges under Alternative Scenario have their operating rating bolded and colored gray. For Scenario 1, one bridge is found to become inadequate (Bridge ID 16645) with an operating rating of 0.974 and inventory rating of 0.585. For Scenario 2, all bridges have operating rating greater than 1. For Scenario 3, the lower bound case (in the column marked as Scenario3 – 105 kips) also has no additional inadequate bridges. The upper bound case (in the column marked as Scenario3 – 129 kips) has three bridges identified as inadequate.

3. Select responding action and estimate costs

For Scenario 1, the resulting situation that one bridge has operating rating lower than 1 may be treated in several different ways. Since the operating rating is the absolute maximum load a bridge can carry, it is not recommended that the 5 percent allowance be applied. One option is to replace the bridge. The total cost would be

14,737 sq ft deck area x $68 per sq ft in Y1997 dollars
= $1.03 millions in Y1998 dollars.

A 3 percent discount rate has been included in the calculation. The unit cost for new bridges is taken from Data Set A-5.2.6 in Appendix A provided by FHWA.

Another option could be a physical load testing to the bridge in order to identify possible reserve strength that is not accounted for in analytical rating. This may cost approximately $6,000 in Y1998 dollars. For the operating rating very close to 1, it is highly likely that a load test will be successful in identifying such reserve strength. Note that the AASHTO evaluation code (1994) has provisions regarding such testing for load rating.

For Scenario 3, the upper bound case has identified three bridges to be inadequate: Bridges 12560, 12565, and 16645. Since their rating factors are clearly lower than 1, the costs for replacing these bridges are calculated. A total of $2.08 millions is estimated to be the cost impact for this category. As discussed above, the impact cost may be reduced if other mitigation measures are effectively taken. For example, physical load testing may be still an option to more accurately determine the load rating of these bridges. Such testing may indicate that replacement of these bridges is not necessary.

4. Sensitivity analysis

As an illustration example, several variables are further studied here in this section of sensitivity analysis. These considered variations from the parameters or assumptions used above may not be valid for ITD, however, this demonstration could be helpful to the reader for other situations.

Assume that load posting is selected as an appropriate reaction to this situation for the 10 bridges with low inventory rating. (Note that some States post at a level between the operating and inventory ratings, while ITD does so at the operating rating.) Such load posting requires enforcement and therefore the enforcement cost should be accounted. For this type of cost, the only available data that can be used as default input come from a Minnesota DOT study (Task Force Report "Truck Weight Limits and Their Impact on Minnesota Bridges" 1991). If we assume that their experience is applicable to the situation and that one enforcement-crew is needed, the following annual cost in 1998 dollars is arrived.

$89,200 per enforcement crew in Y1991 dollars

= $109,700 per enforcement crew in Y1998 dollars

where a discount rate of 3 percent is used.

If, on the other hand, another extreme of responding action is selected, i.e., to replace the bridges with inventory rating lower than 1. For Scenario 1, 10 bridges have this situation. Then the following estimate is arrived.

Total deck area of the 10 bridges = 6185 m^2 = 66574 ft^2

Total Cost = 66574 ft^2 ($68/$ft^2$ in Y1997 dollars)

= $4.66 Millions in Y1998 dollars

The unit bridge cost per deck area is taken form the FHWA average data, which are recommended to be the default data (Data Set A-5.2.6) in Appendix A. A 3 percent discount rate is included in this cost estimation. These 10 bridges were built between 1956 to 1995. In general, it is not expected that they would need to be replaced within 20 years under the Base Case (without the impact of the Alternative Scenario). Thus the $ 4.66 million cost is attributable to this scenario.

If agency specific cost data are available, this estimation can be improved. For example, Table B-1.3.2 shows a list of new bridges constructed in Idaho for a recent three-year period. The average bridge cost per deck area is found there to be $72.38 per sq ft, being 3.4 percent away from the FHWA cost of $70.04 per sq ft, both in Y1998 dollars. When the agency specific data is used, a total of $4.82 millions are computed.

Note that, in reality, responding action needs to be selected with consideration to many factors. Some of these factors are beyond the scope of this project and information on these factors may need to be supplied or identified by the user. Thus this selection will be input by the user into the software module implementing the recommended methodology. Note again that including the above (replacement and enforcement) options here is to illustrate possible applications, not to indicate necessary actions for the agency.

5. Conclusions for this cost impact category

Based on the details presented above, it can be concluded that the considered scenarios of lifting GVW cap may incur the following costs. For Scenario 1, a total of $6,000 is estimated to address the deficiency of Bridge 16645, with respect to overstress-caused deficiency. It is illustrated, however, that if a different responding action is selected, the result will be different and possibly very much different. This indicates that the decision on responding action is a strong influencing parameter to the final result. For Scenario 2, the impact cost is estimated as zero. For Scenario 3, if load testing is selected, a total cost of $18,000 is estimated for the three inadequate bridges identified. If replacement is implemented, a total of $2.08 million is arrived.

B-1.4 Cost Impact Category 4: Deficiency of New Bridges

Level I analysis is used here for this cost impact category, because the information on future new bridges was not readily available.

1. New design load requirement

Using the recommended methodology presented in Appendix A, the adjustment factor AF_{design} for design is defined as follows.

$$AF_{design} = [2W_{AS}^* + 6.9\,\sigma_{AS}^*] / [2W_{BC}^* + 6.9\,\sigma_{BC}^*] \quad AF_{design} \geq 1$$

(3.6.1.3) in Section 3.6

The same truck model with a GVW of 574 kN (129 kips) in Fig.B-1.3.1 is used here to estimate the new requirement for new bridges to be built within the 20-year planning period. This truck was considered to represent the practical maximum loads to be included for design checking. The current design load for Idaho is HS25, which is used here as the reference to identify additional strength requirement and thereby induced incremental costs. Eq.3.6.1.1 in Section 3.6 is used here to estimate the amount of change in the design load:

$$DLCF = (M_{AS,\,129K} / M_{BC,\,HS25})\,AF_{design} \qquad (3.6.1.1)$$

$$M_{AS,\,129K} / M_{BC,\,HS25} \geq 1 \qquad (3.6.1.2)$$

where DLCF has been defined as the design load change factor. Practically, the minimum value for $M_{AS,\,129K} / M_{BC,HS25}$ should be 1, because when the quotient becomes less than 1 (i.e., $M_{AS,\,129K}$ is smaller than $M_{BC,HS25}$) the existing design load HS 25 will govern. The following values of W_{AS}^*, σ_{AS}^*, W_{BC}^*, and σ_{BC}^* were found using the TWHs for the Base Case and the Alternative Scenario. They are used in Eq.3.6.1.3 above to estimate AF_{design}. When AF_{design} is computed to be less than 1, it is set equal to one because any lower live load factor would not be used.

	Scenario 1 (practiced)		Scenario 2 (129 kip-cap for permit in entire state)	
	W* kN(kips)	σ* kN(kips)	W* kN(kips)	σ* kN(kips)
Base Case	443(99.6)	74(16.6)	409(91.9)	45(10.1)
Alternative Scenario	463(104.0)	108(24.0)	415(93.2)	54(12.2)

	Scenario 3 – 105 kips (129 kips legalized)		Scenario 3 – 129 kips (129 kips legalized)	
	W* kN(kips)	σ* kN(kips)	W* kN(kips)	σ* kN(kips)
Base Case	409(91.9)	45(10.1)	409(91.9)	45(10.1)
Alternative Scenario	479(107.7)	34(7.7)	559(125.6)	42(9.4)

The above values for W* and σ* may cause AF_{design} to become less than 1. Physically, it is because when more trucks carry heavier load, the higher end of the TWH becomes less scattered. In ratio AF_{design}, the increase in W* from BC to AS is offset by decrease in σ* from BC to AS Then the denominator becomes smaller than the numerator, which leads to $AF_{design} < 1$. This actually happens in Scenario 3 where the majority of total truck traffic (i.e., the 5-axle truck traffic) is assumed to shift to more heavily loaded trucks. In both lower and upper bound cases, σ* noticeably reduces from BC to AS.

2. Identify impacted bridges

The types and sizes of new bridges to be built in the next 20 years are assumed to be similar to those built in recent years, for a Level I analysis. The bridges on the two routes that were built over the past 6 years are identified in Table B-1.4.1 (from 1993 to 1998 inclusive). This period of 6 years was selected by iteration, to reach a number of bridges that can help determine a reliable estimate of average annual cost impact.

3. Estimate the required design loads and incremental costs

The new design load for each of these bridges is estimated as shown in Table B-1.4.1 using DLCF defined in Eq.3.6.1.1. Accordingly, the incremental costs to meet the new load-carrying requirement are estimated as follows

Cost Impact

= Deck Area x Cost Increase Coefficient

x Unit Cost in Y1998 Dollars per Deck Area

The cost increase coefficient is found using the default incremental cost data in Appendix A (Data Set A-2.7.5). The cost increase coefficient is also listed in Table B-1.4.1 for each bridge for Scenarios 1 and 2. It shows that the average annual cost for new bridges due to design load increase is respectively about $43,000 and $27,000 for these two scenarios. For the PP = 20 years. They lead to total of $861,080 and $542,600. The difference between them is apparently due to the loads used. Again, Scenario 1 used WIM data as the load, and Scenario 2 used the FHWA VMT data for the Base Case and thereby predicted TWH for the Alternative Scenario.

Table B-1.4.2 shows the same information for Scenario 3 for the two limiting cases, referred to as Scenario 3- 105 k and Scenario 3 – 129 k, respectively. They give the total costs as $521,180 and $1,010,500 for 20 years.

Note that, when we consider the fact that only three trucking companies registered under Scenario 1 for a few trucks, it may not appear to justify design load increase. However, the WIM data used for Scenario 1 indicate that a remarkable amount of heavier

trucks now are traveling on these two pilot routes, as shown by the WIM data in Figs.B-1.1.3 and B-1.1.4. This requires a higher design load quantified here. The estimated costs for this category appear to be a major contributor to the grand total costs.

4. Sensitivity analysis

It is seen in Table B-1.4.1 that the number of years to look back in the analysis could be sensitive to the result for a relatively small network as this one in Idaho (i.e., two routes). For example, if 1 year is used instead of 6 years to look back, the annual cost for new bridges would be zero because no bridge was built on these two routes in 1998, as shown in Table B-1.4.1. This situation continues until 1995 is reached when two new bridges were built, the annual cost would suddenly change to cover these two bridges over a period of 4 years (from 1995 to 1998 inclusive). It is envisioned that this may be the most sensitive variable to the final result. Therefore, it is advisable that this sample of new bridges should be sufficiently large to provide a reliable annual cost for new bridges.

5. Conclusions for this costs impact category

The expected cost impact for this category is found to be about $43,100 and $27,100 per year in Y1998 dollars, respectively for Scenarios 1 and 2. The total costs for this category for PP=20 years are $861,100 and $542,600 for these two scenarios, respectively.

B-1.5 Summary for the Idaho Example

Table B-1.5.1 summarizes the cost estimation for this Idaho example for the three scenarios considered. It is shown that Cost Impact Category 4 for new bridges is the dominant contributor to the total impact costs. The example also shows that selecting which action to respond to the identified fatigue increase and additional load-carrying-capacity deficiency may be sensitive to the final result. It indicates that application of the recommended methodology needs adequate attention to what should be the appropriate reaction.

Furthermore, Scenarios 1 and 2 do not exhibit significant difference. This is mainly because that actually a significant number of trucks are now operating under the new regulation, although the registration data do not show this fact. The WIM data before and after the weight limit change clearly indicate that this is the case.

B-1.6 Truck traffic in Scenario 3

For the Scenario 3 analysis, a subtask was to estimate the percentage of trucks that are expected to exceed the inventory rating of the bridge. It should be noted that load ratings indicate safe load carry capacities based on truck load effects not just truck weights. While load shifting modeled here only uses information of truck weights (GVW), some assumptions about the relation of the truck weight and its load effect are needed to accomplish what is required here.

It is assumed here that the flexural moment of each truck is proportional to the truck's gross weight, according to the truck models in Fig. B-1.3.1. Further, the inventory rating is determined using the 129 kip truck model in that figure. Based on these assumptions, Table B-1.6.1 gives the percentage of trucks exceeding the inventory rating for each bridge in Scenario 3. It is shown that some bridges have more than 10 percent of the truck population exceeding their inventory rating. For the upper bound case of 5-axle trucks shifting to the 129 k truck model, 17 of these bridges have almost 20 percent of the truck traffic above the inventory rating. This is expected to certain extent, because these bridges have low inventory ratings.

B-2 The Michigan Example

This example applies the recommended methodology in Appendix A to the state of Michigan for an Alternative Scenario of legalizing the 3S3 configuration with a gross weight of 431 kN (97 kips). Fig. B-2.1.1 shows two truck configurations under this scenario as the practical maximum truck weights, representing the upper and lower bounds for the total length of these trucks. The shorter one is referred to as 3S3A and the longer one as 3S3B, respectively. This scenario has been considered in several studies at the federal and state levels. It still remains to be a candidate for a change to the current truck weight limits. In addition, since this configuration is legal in Canada, bordering states with Canada such as Michigan are receiving increasing pressure to legalize it as an effort of implementing the North American Free Trade Agreement (NAFTA).

There are approximately 12,400 bridges in the State of Michigan. Thus, going through every one of these bridges as done for steel fatigue in the Idaho example does not appear to be realistic. Level I analysis is therefore used for all four cost impact categories in this example. As discussed in Chapter 3, the main difference between the Level II and Level I analyses is that the former requires input data specific for the bridge (including factors controlled by the site and the jurisdiction). The latter samples the population and then uses the sample to project the costs for the entire network. As to be seen below, sampling and/or approximation are used here in all the analyses for the three cost impact categories investigated.

B-2.1 Cost Impact Category 1: Steel Fatigue

1. Identifying representative samples of vulnerable bridges

Among the bridges in Michigan, approximately 5,600 have a steel superstructure. Analyzing every one of them was not feasible considering the time and cost constraint for this project. Sampling was thus used here for a Level I analysis. As discussed in Chapter 3, this sampling approach will use a small number of bridges to project to the entire population. The bridge population we have here is not uniform with respect to possible cost impact. Thus the stratified sampling concept is used. This is to avoid such an error that a bridge is selected to represent a group of very much different bridges with respect to estimated fatigue accumulation and induced costs. Stratifying is done here according to the following parameters. 1) Jurisdiction (state versus local agencies). 2) Type of steel beams (plate girders versus rolled beams). 3) Type of span (simple versus continuous spans). 4) Age (built before 1940, between 1941 and 1950, between 1951 and 1960, … up to the respective years the targeted fatigue prone details likely fell off favor). 5) Maximum span-length. Within each stratum (group), random sampling was used to select a number of bridges for detailed remaining life analysis and the following cost analysis.

Two fatigue prone details are focused in this example's screening: 1) Partial

length cover-plate toe welds (for rolled beams). 2) Longitudinal stiffener or attachment plate welds to the web close to the tension flange (for plate girders). Both of these targeted details have either E or E' category fatigue strength, which are the weakest strength categories. The guidelines given in Section A-5.1.3 of Appendix A are used for this screening using the Michigan DOT's bridge inventory. 3,200 steel bridges resulted from this screening process. They were grouped or stratified according to the 5 factors discussed above.

Table B-2.1.1 shows the randomly selected 93 sample bridges, representing the 3,200 steel bridges identified to possibly have the targeted fatigue prone details of E or E' strength. The table also shows the characteristics of the bridge groups that the sample bridges respectively represent. For example, the second bridge in the table, Bridge 994, represents a group of plate girder bridges of continuous spans constructed between 1961 and 1970, and with the maximum span length between 40 m and 50 m (131 ft to 164 ft). Two bridges from the population belong to this group represented by Bridge 994. When the number of bridges in a group is 50 or smaller, one bridge is used. When this number is larger than 50, two bridges are included in the sample. This sampling resulted in the 93 bridges that were thought manageable within this project. This sampling rate may also represent a typical resolution of analysis for a state agency with its size of the network similar to that of Michigan. Of course, for application of the recommended methodology by agencies, the sampling rate will depend on the available resources for the application.

2. Estimate the remaining lives for both the Base Case and Alternative Scenario.

For the sample included in Table B-2.1.1, plans for each of these bridges were requested from the agencies with jurisdiction. The state agency is Michigan DOT, and the local owner can be a town, city, or a county. Except a few cases where no plans were available (for example, Bridge 6556 listed as No.22), plans were acquired and reviewed for identification of the two focused types of fatigue-prone detail. Table B-2.1.1 also shows the result of this identification in the last column. Then a detailed stress range analysis was performed for each of these bridges identified. The results are used in the remaining fatigue life calculation according to the AASHTO procedure. Note that the calculation used the formula in Eq. 3.3.2.4, instead of the chart in AASHTO (1990). This has been discussed in Section 3.3.

Table B-2.1.2 summarizes the result for this step of analysis. It includes 16 bridges that were found to have either of the targeted details, out of the sampled 93 bridges. They represent a total of 1,005 bridges, as indicated by the number of bridges in the group for each bridge. Note that Bridges 7833 and 9158 represent the same group of 339 bridges. For each bridge, Table B-2.1.2 exhibits the resulting remaining lives under the Base Case (BC) and the Alternative Scenario (AS). They are estimated using the AASHTO procedure and the default VMT data for the needed TWHs. Each functional class has a VMT data set as the sample in Data Set A-5.2.1 of Appendix A, and thus a pair of TWHs for the Base Case and the Alternative Scenario. The TWH for the Alternative Scenario is generated using the prediction method in section A-5.1.1 of

Appendix A and discussed in more details in Section 3.1. That method is applied to the VMT data for the impacted types of vehicles. In this case, 3S2 truck traffic is considered to be impacted and will shift to the 3S3 configuration. Other vehicle types, such as single unit with 3 or 4 axles, will not change.

3. *Select responding action and estimate costs*

The remaining lives in Table B-2.1.2 are used to compute the increased probabilities of failure using Eq.3.3.3.2, which are also shown in Table B-2.1.2 as increased P_f. Failure is defined here as fatigue cracking during the planning period PP set as 20 years here. The expected repair costs are estimated as

Expected Impact $Cost_i$

= Increased Probability of Failure $P_{f,i}$ x Action $Costs_i$

where subscript i indicates the specific detail analyzed, the repair cost for the fatigue-prone detail is calculated using Data Set A-5.2.4 in Appendix A's default database. It is $1360 for each longitudinal attachment weld and $1800 for each cover plate weld. Note that they represent approximately costs of Year 1998. Then the expected impact costs for each bridge group are calculated as the product of the expected impact for the representative bridge and the number of bridges in the group. When there are two bridges representing the same group, their respective costs are averaged first before multiplied by the number of bridges in the group. The expected repair costs for these groups are also included in Table B-2.1.2. At the end of the table, the total cost is projected to dollars in Year 2000, to be consistent with costs from other categories.

4. Sensitivity analysis

Several input parameters are examined here to understand their effects on the estimated costs. They include 1) the window parameters (a_1, a_2, b_1, and b_2) for estimating the TWH for the Alternative Scenario, defined in Fig. 3.1 and Eq. 3.2.2.1; 2) the traffic growth factor u; and 3) the impact factor for dynamic effects increasing stress.

1) The window parameters in estimating TWH for the Alternative Scenario (a_1, a_2, b_1, and b_2 defined in Fig.3.1). These parameters describe how much truck traffic could be impacted upon and change their behavior. Although these parameters have been tested as presented in Section 3.2, there is a possibility that a specific case deviates from what is modeled by these values. Thus, an upper bound and a lower bound are used to understand the sensitivity of these parameters to the final cost result.

Table B-2.1.3 shows the same result as in Table B-2.1.2 except for a_1=a_2=5% and b_1=b_2=10%, different from the default values used in Table B-2.1.2: a_1=a_2=10% and b_1=b_2=20%. This case represents very minor impact of a weight limit change on trucking behavior with respect to the change in the weight carried. Apparently, this small impact is estimated to cause a cost impact lower by about 60% compared with the case in Table

B-2.1.2.

Table B-2.1.4 represents another extreme of truck behavior change in the weight carried. The window parameters are set as a_1=a_2=15% and b_1=b_2=30%, which increase the cost due to more extensive response of trucks to the considered weight limit change. However, the cost increase is not as much as the increase from the case in Table B-2.1.3 to that in Table B-2.1.2. This appears to be because when the GVW is increased as considered here, the amount of traffic is reduced, due to assumed constant payload. The higher load is expected to increase the stress and thus to shorten the remaining life, while fewer load cycles caused by less traffic will permit longer life. This situation will cause the increased failure probability to be small and result in lower expected impact costs. As seen, these two effects counteract to each other here.

2) Dynamic impact factor. In the analysis results shown in Tables B-2.1.2, a typical value of 15% has been included to account for dynamic impact of vehicles in motion. In reality, this impact factor has been found to be a function of the road's roughness, which may be a function of time over the life of the bridge. This step is to examine the effect of this parameter. Since no specific information is available regarding the variation of this factor with time, a maximum value of 30% and a minimum value of 10% are used to estimate the influence of this parameter on the cost.

Table B-2.1.5 shows that the total steel fatigue cost reduces about 22 percent for the dynamic impact factor I reduced from 15 percent to 10 percent. Table B-2.1.6 shows that it increases about 73 percent when I increased from 15 percent to 30 percent.

3) Truck traffic growth factor u. The truck traffic growth factor used in this analysis is based on the two traffic amounts for two times (usually 20 years apart) recorded in the NBI. The earlier time is the time when this data was updated by the agency. In the 1998 version of the NBI, the future time referred to is often 2013 or later. It is possible that the resulting growth factor is not the same as the real life-average growth factor. Thus adding an additional 3% to the traffic growth rate u is thought to give its possible maximum value. Based on this new growth factor, the analysis for this cost impact category is performed again and the results are shown in Table B-2.1.7. It shows that the considered amount of traffic growth increases the cost by about 55 percent. It should be noted that a 3 percent increase in the traffic growth is considered an absolute upper bound for this parameter, and it is unlikely to occur.

5. Conclusions for this costs impact category

The expected additional cost for steel fatigue due to the considered scenario of truck weight limit change is estimated at a total of about $6,200 in Y2000 dollars. In general, the variation of the parameters examined is not expected to alter the final cost result's order of magnitude.

<u>B-2.2</u> Cost Impact Category 2: RC Deck Fatigue

Michigan is located in the Snow Belt and typically uses a large amount of salt for deicing. The recommended methodology does not provide a quantitative method to assess RC deck fatigue's contribution interactive with rebar corrosion due to salt. Based on the recommendation in Section 3.4, cost impact for this category for Michigan is conservatively set to zero.

B-2.3 Cost Impact Category 3: Deficiency in Existing Bridges

This cost impact category is covered here using the Level I analysis. This level of analysis requires the agency's bridge inventory database or the NBI. The maximum moment due to the new rating vehicle under the Alternative Scenario is also needed for a range of generic spans of interest. The TWHs used in this cost impact category are the same ones used in the category for steel fatigue. Thus their generation does not need to be repeated here.

1. Identify the criterion for deficiency

Michigan DOT uses several rating vehicles for load rating. A review of the state's bridge inventory indicated that for the vast majority of the bridges in the State, the governing vehicle is the 3S8 configuration, as shown in Fig. B-2.3.1. Furthermore, Michigan DOT uses the operating rating to determine inadequacy of bridges. Accordingly, this analysis is conducted using the 3S8 vehicle as the reference for comparison to find new load ratings under the Alternative Scenario, as follows:

$$RF_{AS} = RF_{BC}\ (M_{3S8} / M_{3S3})\ /AF_{rating} \qquad \text{Eq.3.5.1.1 in Section 3.5}$$

$$AF_{rating} = [2W_{AS}^* + 1.41\ t(ADTT_{AS})\ \sigma_{AS}^*] / [2W_{BC}^* + 1.41\ t(ADTT_{BC})\ \sigma_{BC}^*]$$

$$\text{Eq.3.5.1.2 in Section 3.5}$$

where M_{3S8} and M_{3S3} are the maximum moments due to the 3S8 vehicle in Fig. B-2.3.1 and the 3S3 vehicles in Fig. B-2.1.1 for the span. For continuous spans, the M_{3S8} / M_{3S3} ratio is calculated for several commonly critical sections, then the maximum ratio is used in the above equations to find the new rating factor RF_{AS} under the Alternative Scenario. As defined in Section 3.5.1, RF_{BC} is the operating rating factor for the Base Case (the existing operating rating factor in the inventory database). AF_{rating} is the ratio between the live load factors for the Alternative Scenario and the Base Case. W* and σ* are the mean and standard deviation of the top 20 percent of the TWH, and t is a function of annual daily truck traffic (ADTT) as given in Section A-3 of Appendix A. Subscripts $_{BS}$ and $_{AS}$ respectively refer to the Base Case and Alternative Scenario. The ADTT values are taken from the agency's bridge inventory database.

2. Identify deficient bridges

The new rating factor RF_{AS} under the Alternative Scenario is calculated for every bridge in the inventory as long as its current rating factor RF_{BC} is equal to or greater than 1.0. This will count only the cost impact or cost increment.

Further, since site-specific WIM data are not available to calculate RF_{AS} in Eq. 3.5.1.1, the FHWA VMT data (sampled in Data Set A-5.2.1 of Appendix A) are used to generate TWHs for the Base Case. These data are meant to be functional class specific. In other words, bridges that carry roads belonging to the same functional class will use the same TWH, provided by the FHWA VMT data. The result is then used to predict the TWH under the Alternative Scenario. The prediction method is also presented in Appendix A. Section 3.2 provides more details of the concept for the method. Note that decrease of traffic volume due to truck weight limit change is also covered in the analysis. This is reflected in the change from $ADTT_{BC}$ to $ADTT_{AS}$ in Eq. 3.5.1.2. This decrease is expected to result from that fewer trips will be required to transport the same amount of payload if each trip can transport more under the considered scenario of weight limit change. The amount of decrease for this example is found to be typically about 1 to 2 percent.

Table B-2.3.1 lists 15 bridges from the State's network that are not deficient under the Base Case (i.e., $RF_{BC}\geq 1.0$) and are found deficient under the Alternative Scenario (i.e., $RF_{AS}<1.0$). These are simple span bridges with a relatively short span length, except Bridge 7301, which has two short continuous spans. For these short spans, the 3S3 rating vehicles in Fig. B-2.1.1 produce a higher maximum moment than the current governing rating vehicle 3S8 in Fig. B-2.3.1. Mainly it is because the 3S3 vehicles have more severe axle groups (a tandem and a tridem), which govern the moment effect of short spans where not all axles can be on the span simultaneously. In addition, the AF_{rating} values larger than 1 indicate increase of uncertainty and the need to increase the live load factor. Collectively, these two factors bring down the rating factor for these bridges under the Alternative Scenario.

For other bridges in the network, the existing reserve strength (reflected by RF_{BC}) and the larger-than-1 ratios between the maximum moments due to the rating vehicles under the Base Case and the Alternative Scenario (M_{3S8} / M_{3S3}) combine to cancel the requirement for a higher live load factor (AF_{rating}>1). The net result is that RF_{AS}>1. This can be seen in Eq. 3.5.1.1 copied above.

For this network of approximately 12,400 bridges, the number of additional deficient bridges under the Alternative Scenario (15) is relatively small. In addition, the severity of deficiency, indicated by how far the rating factor RF_{AS} is away from 1.0 in Table B-2.3.1, is also not large. This situation is mainly due to two factors. 1) The operating ratings are used here, while this is consistent with the agency's practice. 2) The Michigan DOT currently uses the severe 3S8 load model to perform load rating. Relatively, the new rating truck model (3S3s in Fig.B-2.1.1) apparently is not as severe as the current governing load 3S8 for most spans. The bridges in Table B-2.3.1 are typically short spans where the weight of the most severe wheel-group controls the maximum

moment effect.

3. Select responding action and estimate the costs

Based on the results in Table B-2.3.1, one option of response to the situation is to replace these bridges. The replacement costs for these 11 bridges are calculated using the default new bridge cost data included in Data Set A-5.2.6 of Appendix A. That data set is provided by FHWA based on recorded expenditures. A $79 per square foot of deck area is used here for cost estimation as the 1997 cost. This unit cost is factored to $86.33 per square foot for Year 2000 using a 3 percent discount rate. The cost calculation results are also given in Table B-2.3.1. A total of about $1.13 million is estimated. It should also be noted that the RF_{AS} in Table B-2.3.1 is not significantly far from 1. If a 5 percent allowance is given, the total cost reduces to about $0.73 million as shown in the last column.

4. Sensitivity analysis

Two parameters are examined in this step for illustration. 1) The window parameters for truck load shifting under the Alternative Scenario, and 2) variation in what action to take for the Alternative Scenario in response to the identified deficiency.

The default values for the window parameters in Fig. 3.1 are given as a_1=a_2=10% and b_1=b_2=20%. As discussed in Section 3.2, these parameters describe the degree of impact on truck operation behavior. Namely, larger a_1 and a_2 model that more trucks are expected to change their behavior to use the truck weight limit change. Larger b_1 and b_2 describe that more trucks will be affected in terms of the weight they carry in response to the weight limit change. The default values are used in obtaining the results in Table B-2.3.1.

Table B-2.3.2 shows the bridges identified to be deficient under the Alternative Scenario for the window parameters set at a_1=5%, a_2=5%, b_1=10%, and b_2=10%. Other parameters are kept unchanged from the case shown in Table B-2.3.1. Note that based on the data discussed in Section 3.2, these values would represent an extreme situation of lower bound. Thus, it is unlikely to take place in reality, as shown in Section 3.2.3 by the Arkansas and Idaho WIM data before and after a truck weight limit change. Comparison between Tables B-2.3.1 and B-2.3.2 indicates that the total replacement costs required to eliminate the deficiency under the Alternative Scenario would reduce by about 30 percent, while the number of deficient bridges would reduce to 10 from 15.

For the window parameters increased to a_1=15%, a_2=15%, b_1=30%, and b_2=30%, Table B-2.3.3 shows that the number of bridges to be deficient will increase to 23 from 15. The total replacement costs will be close to $1.65 millions. Note that these values for the parameters model a wider spread change of trucking behavior as a result of the considered scenario of truck weight limit change. They represent an upper bound for these parameters.

Furthermore, the impact cost is a function of the action selected in response to the identified deficiency. Other options may be feasible. For example, posting and enforcement for the posting could be one of them. If this option is selected, the enforcement cost will be the only significant cost. Using the default data included in Data Set A-5.2.3 of Appendix A, one enforcement crew is estimated at $89,200 per year (in Y1991 dollars). Using a 3 percent discount rate, this cost is converted to $116,400 per year (in Y2000 dollars).

5. Conclusions for this cost impact category

The operating rating represents the absolute maximum load that can be permitted to cross a bridge. Thus the 5 percent allowance is not recommended here. Further, a $116,400 per year enforcement cost (in Y2000 dollars) would result in a total cost of $2.33 millions for the PP of 20 years ($116,400 x 20 = $2.33 millions). This does not appear to be the most cost effective measure. Thus, the expected cost for this category is judged to be close to a total of $1.13 million in Y2000 dollars.

B-2.4 Cost Impact Category 4: Deficiency in New Bridges

1. Identify the criterion for new design load

In concept, determining this criterion is similar to identifying the same for deficiency discussed above. As discussed in Section 3.6, the following design load change factor DLCF should be used.

$DLCF = AF_{design}\ (M_{3S3} / M_{HS25})$ Eq.3.6.1.1 in Section 3.6

$M_{3S3} / M_{HS25} \geq 1$ Eq.3.6.1.2 in Section 3.6

$AF_{design} = [2W_{AS}^* + 6.9\ \sigma_{AS}^*] / [2W_{BC}^* + 6.9\ \sigma_{BC}^*]$

$AF_{design} \geq 1;$ Eq.3.6.1.3 in Section 3.6

M_{HS25} / M_{3S3} is the ratio of the maximum moments between the HS25 (current design load for Michigan) and the 3S3 trucks in Fig.B-2.1.1. W^* and σ^* are the mean and standard deviation of the top 20 % of the TWH for the entire state. The subscripts AS and BC stand for Alternative scenario and Base Case, respectively. Unlike the Cost Impact Category 3 for rating deficiency where TWHs are site dependent (or functional class dependent if site dependent TWHs are not available), the TWH used here covers the entire network of the State. This is because the design load is usually applicable to the entire network and is not site dependent.

Based on Eq.3.6.1.1, 25 times DLCF will indicate the magnitude of the new sign load using the HS designation, because the HS-25 load is the reference load. The HS designation is used here because relevant cost data are more available, as discussed in Chapter 3. Some default cost data are given in Data Set A-5.2.7 of Appendix A along with the recommended methodology.

2. Identify the impacted bridges

The Michigan DOT's current bridge inventory database is searched to identify those bridges constructed in the last year (1999). 30 bridges were found to have been designed for HS20, which is lower than the current statewide design load HS-25. These bridges carry local roads subjected to apparently much lower loads and smaller volumes of traffic. Most likely, the investigated Alternative Scenario will not impact those bridges. Thus, they are excluded from cost estimation. The rest of the identified bridges are listed in Table B-2.4.1.

3. Estimate incremental costs

Most of these bridges in Table B-2.4.1 have a prestressed concrete beam superstructure as indicated. The DLCF factor is identical for these bridges because 1) the AF_{design} is identical for the entire State and 2) the ratio M_{3S3} / M_{HS25} is 1, subject to the requirement of minimum value equal to 1. For these cases, the maximum moment ratio M_{3S3} / M_{HS25} is actually lower than one by analysis, and it is set equal to 1 according to the minimum value requirement. It is because the 3S3 vehicle's moment effect is lower than the HS25 moment effect for these short span bridges. The DLCF determines the increase of design load for the bridge, which in turn determines the increased costs. A value of 1 means no change in design load effect including the live load factor. Thus, the cost for each individual bridge is zero and so is the total cost for this category, as shown in Table B-2.4.1.

4. Sensitivity analysis

It is felt that the sample used to project the future annual cost may have a noticeable effect on the final cost result. This sample includes the bridges constructed in 1999 listed in Table B-2.4.1. Apparently, the number of bridges constructed in a year and their costs vary from year to year. Thus, this parameter is studied here in the sensitivity analysis. Table B-2.4.2 shows the new bridges constructed in 1998 in Michigan, in the same format as in Table B-2.4.1. The last column "cost impact" gives the additional costs estimated for these bridges as a result of the considered Alternative Scenario. The total impact cost is not different from that in Table B-2.4.1 for 1999.

Furthermore, the parameters used in the TWH prediction for the Alternative Scenario, a_1, a_2, b_1, and b_2, are examined for their influence on the cost impact. Two cases of these parameters are included in this sensitivity analysis. The first one has a_1=a_2=5% and b_1=b_2=10%. The second one has a_1=a_2=15% and b_1=b_2=30%. The results are compared with those presented in Table B-2.4.1 where a_1=a_2=10% and b_1=b_2=20%.

The mean and standard deviation values for the top 20 % of the TWHs studied are shown in Table B-2.4.3 for these cases. Little difference is seen in DLCF among these three cases of the parameter values. These changes in a_1, a_2, b_1, and b_2 for this Michigan example increase W* and decrease σ*. As a result, AF_{design} defined in Eq.3.6.1.3 changes very little. Thus, the total costs remain almost constant for these cases of the window parameters.

Physically, shifting of payloads from a group of vehicles with lower GVW to another group with higher GVW caused the top 20 % of the TWH to be less scattered. Thus, when the mean value W* increases, the variation described by σ decreases. Note that this is observed in the Michigan data used here and it is not necessarily always true for other bridge networks and for other scenarios.

5. Conclusions for this cost impact category

It appears that the cost for this category is zero. It is because current design load (HS25) induces higher load effect than the 3S3 vehicles, and the TWH for the Alternative Scenario will reduce variation in truck loads to these bridges.

B-2.5 Summary

Table B-2.5.1 summarizes the cost results for this example, considering legalizing the 3S3 configurations for the state of Michigan. It is shown that Cost Impact Category 3 for treating deficient bridges is the dominant contributor to the total impact costs. This is consistent with an earlier TRB study (Moses 1989), covering the entire nation for several scenarios of truck weight limit changes. It indicates that application of the recommended methodology should spend adequate attention on this item in acquiring reliable input data, unless there are data to indicate otherwise. Further, additional costs for new bridges may also become significant as shown in the Idaho example.

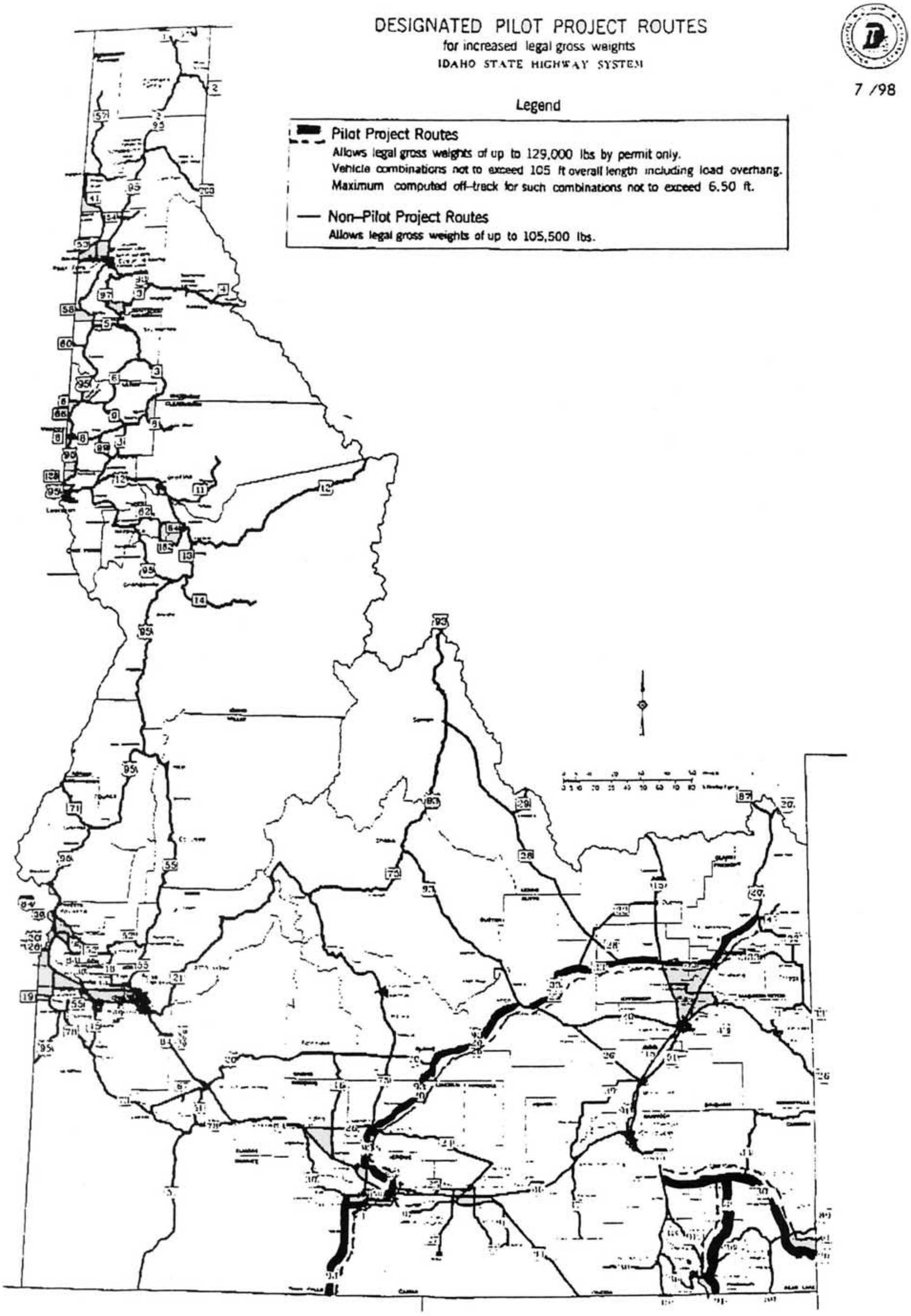


Fig.B-1.1.1 Designated Routes for Permit Weight Limit Increase in Idaho

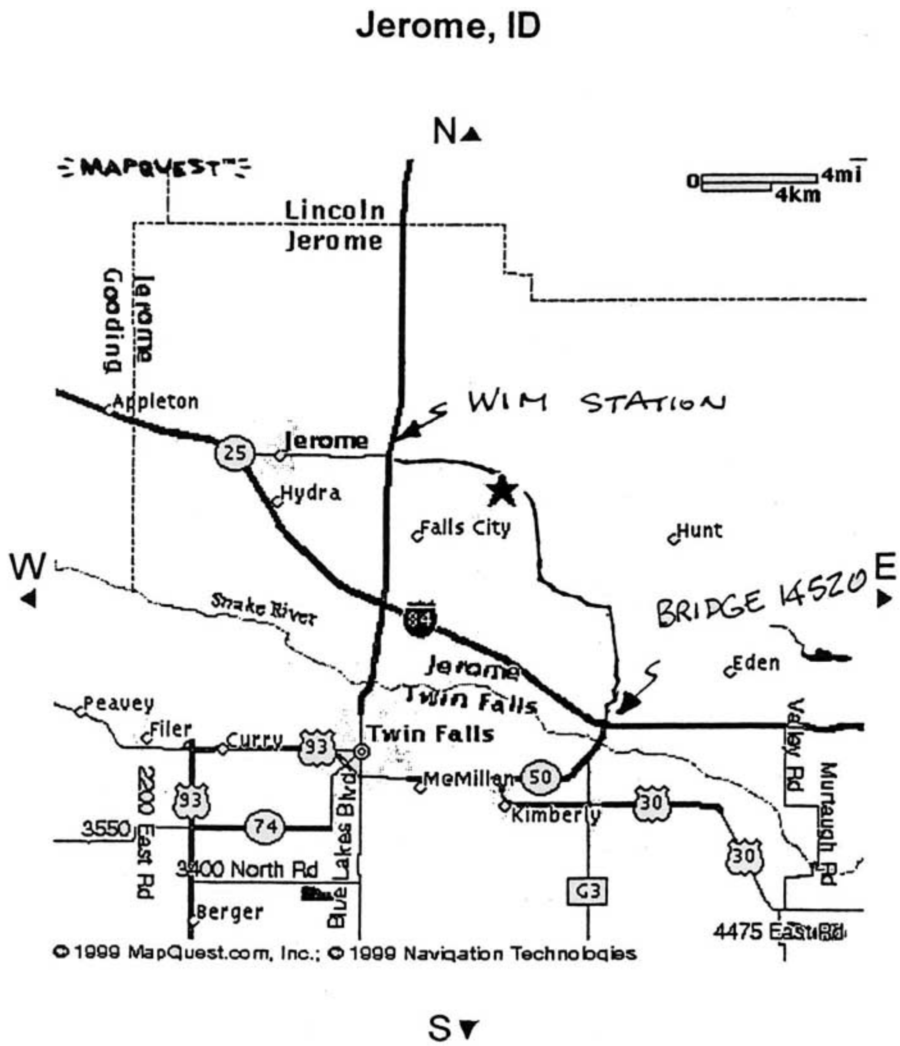


Fig.B-1.1.2 Location of Bridge No. 14520 and WIM Station in Idaho

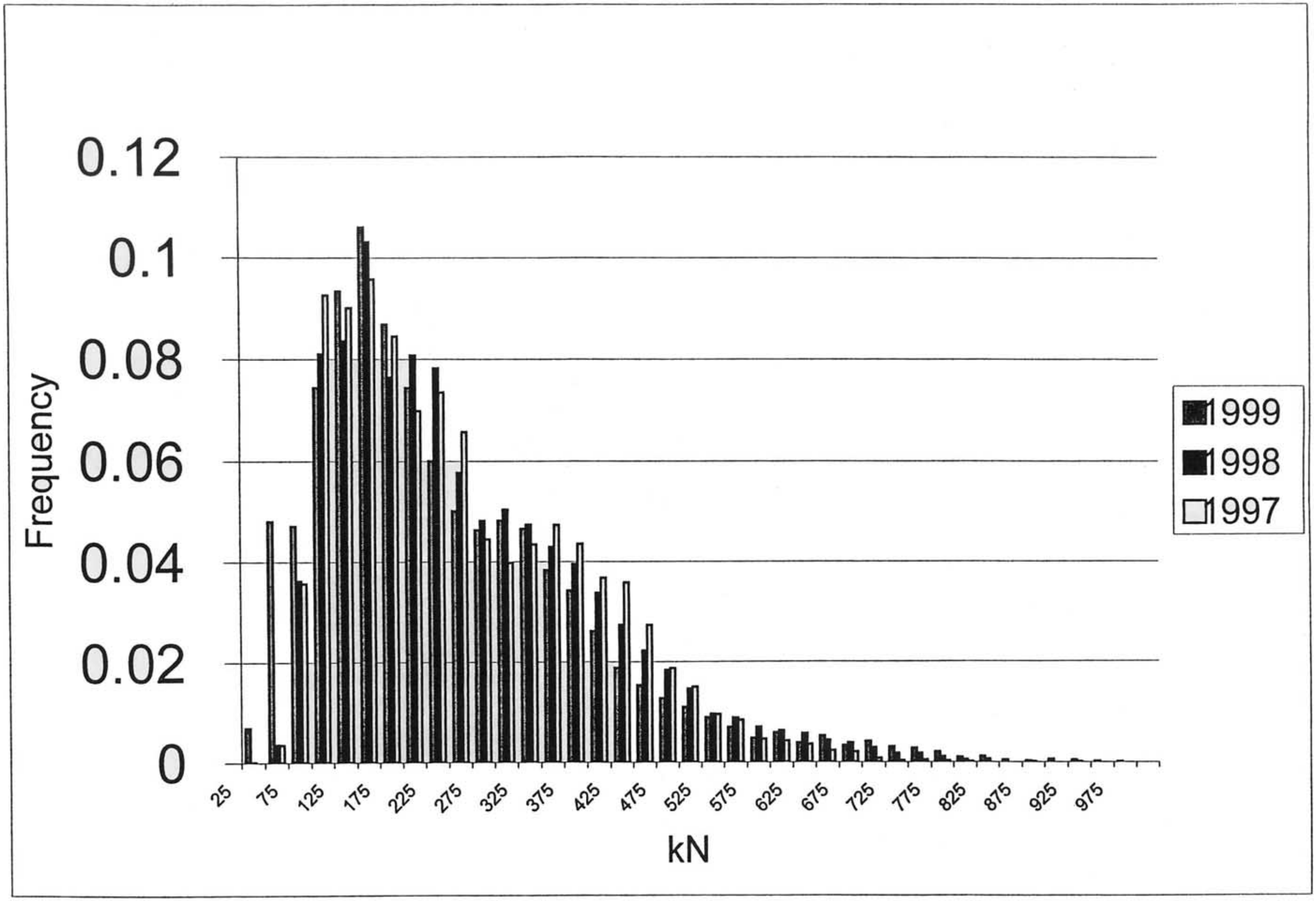


Fig.B-1.1.3 Truck Weight Histograms for the Pilot Routes (1 kN = 0.225 kips)

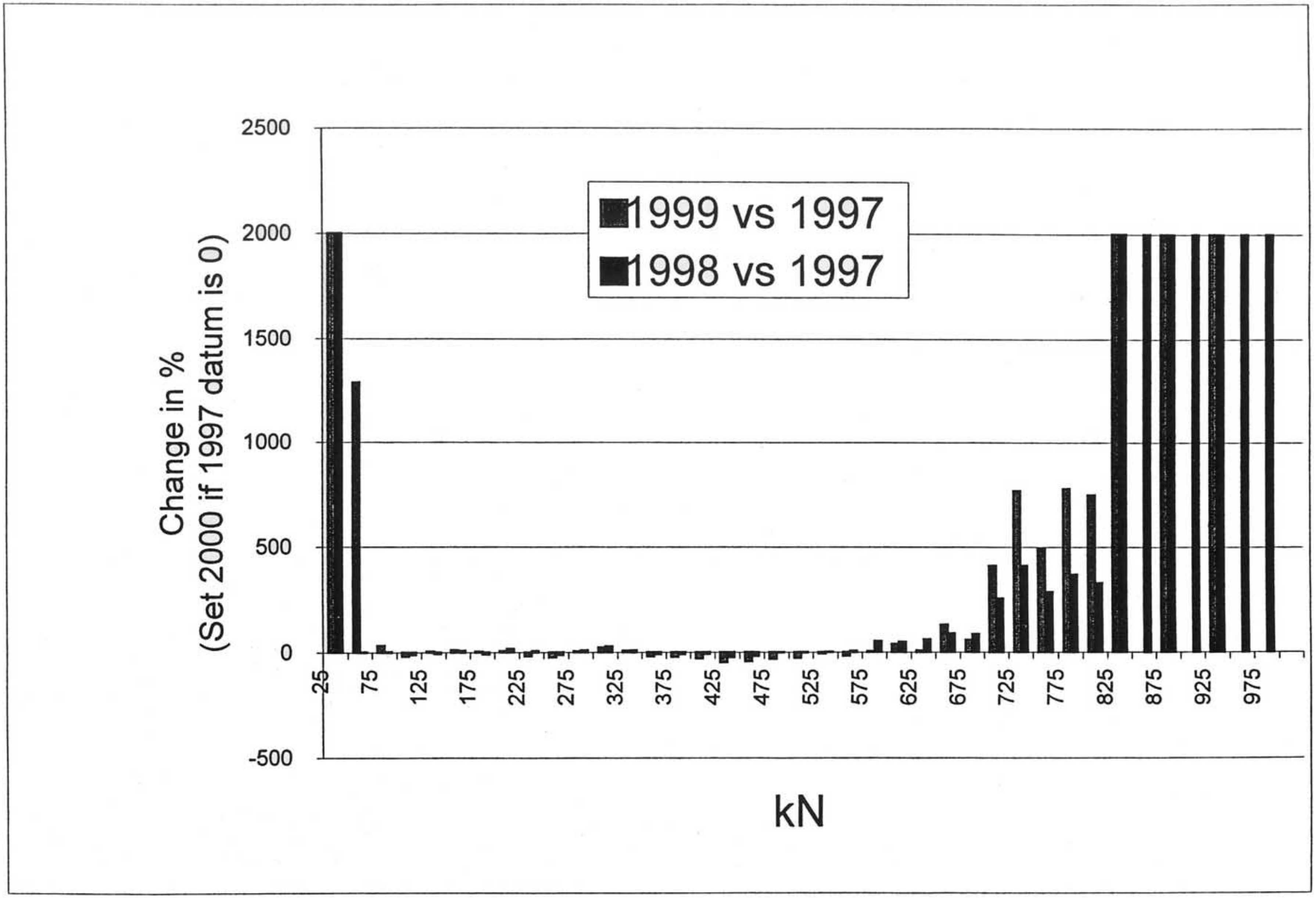


Fig.B-1.1.4 TWH Change after Pilot Project in Idaho (1 kN = 0.225 kips)

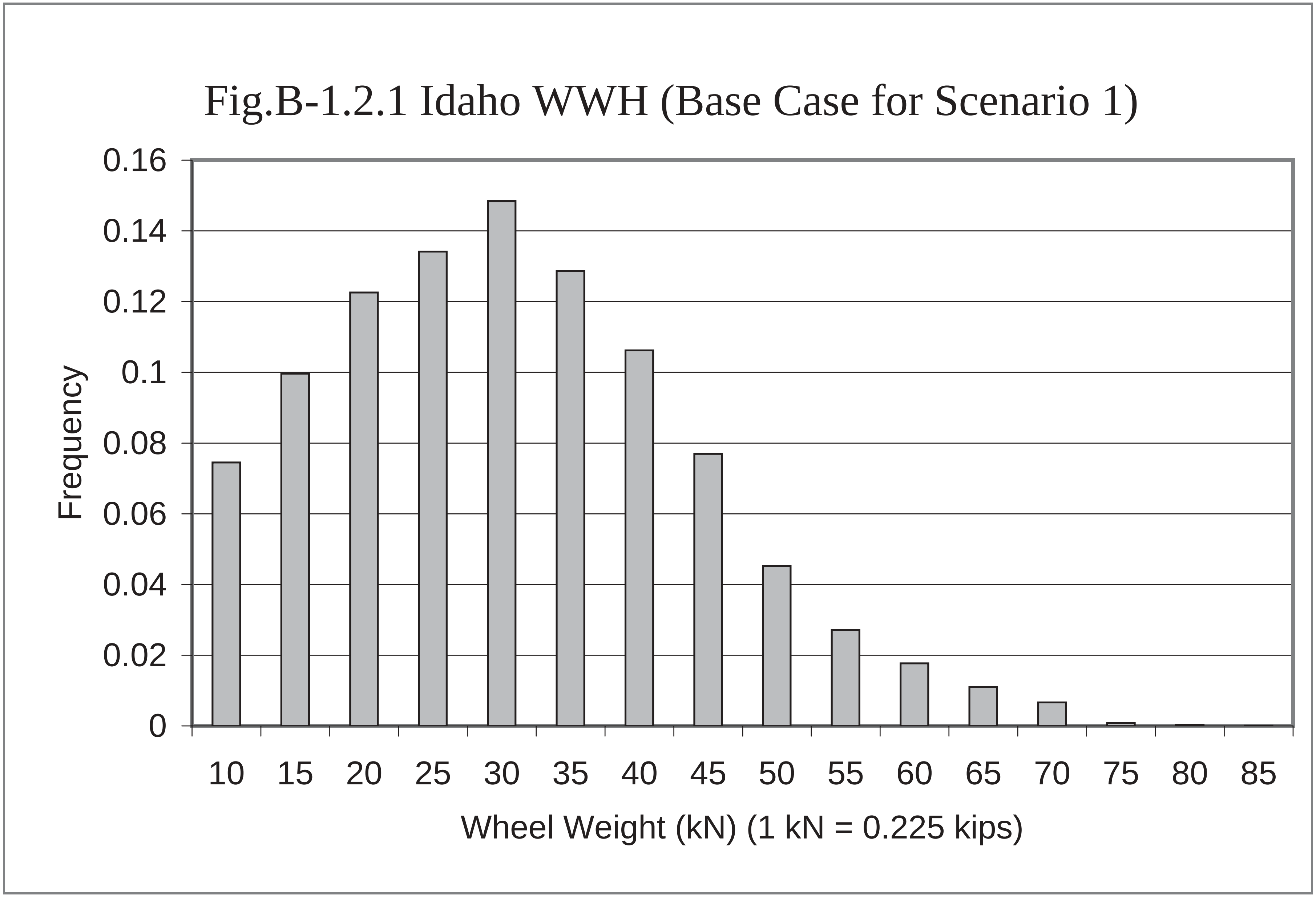
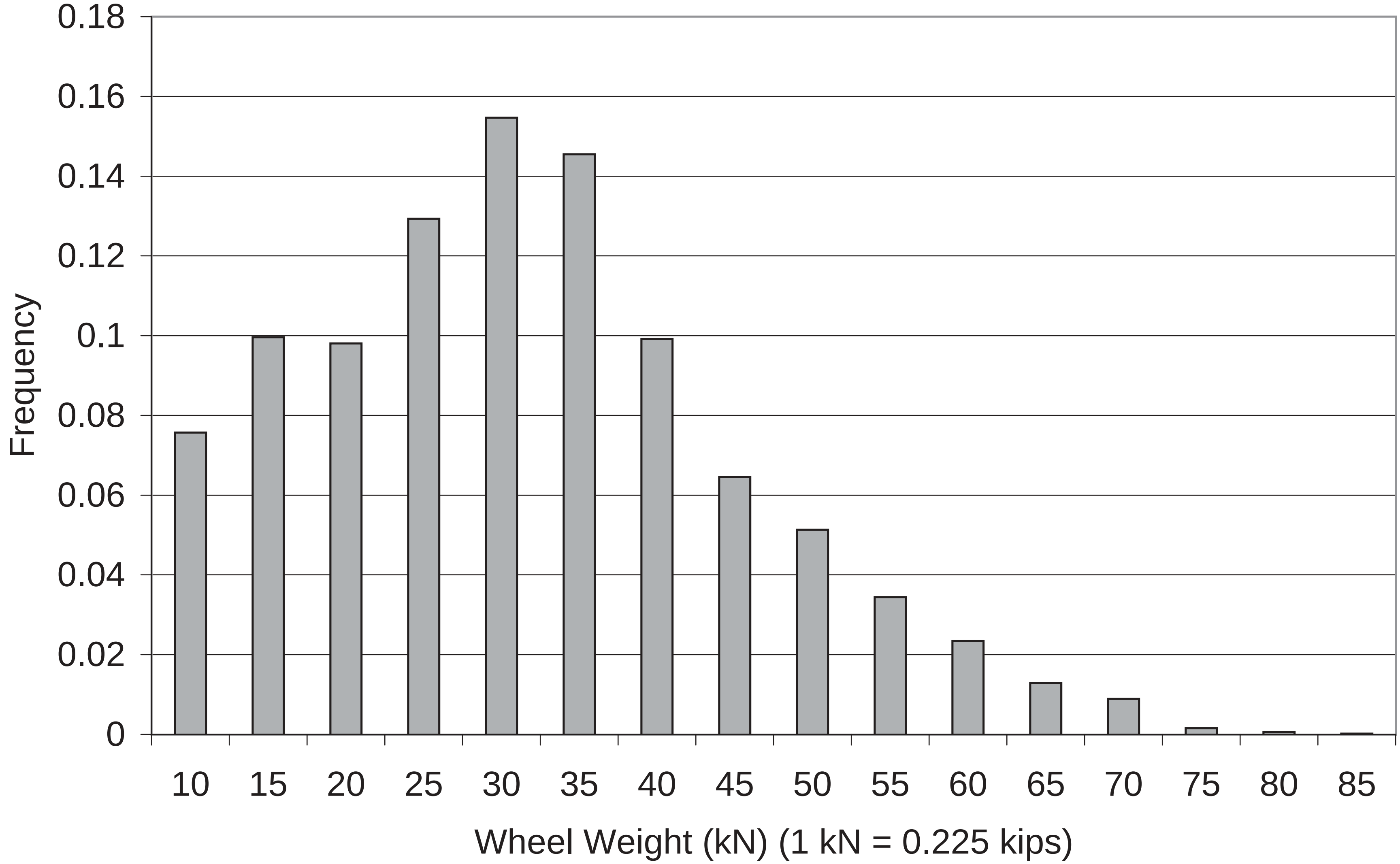


Fig.B-1.2.1 Idaho WWH (Base Case for Scenario 1)
Frequency
0.16
0.14
0.12
0.1
0.08
0.06
0.04
0.02
0
10 15 20 25 30 35 40 45 50 55 60 65 70 75 80 85
Wheel Weight (kN) (1 kN = 0.225 kips)

Fig.B-1.2.2 Idaho WWH (Alternative Scenario for Scenario 1)



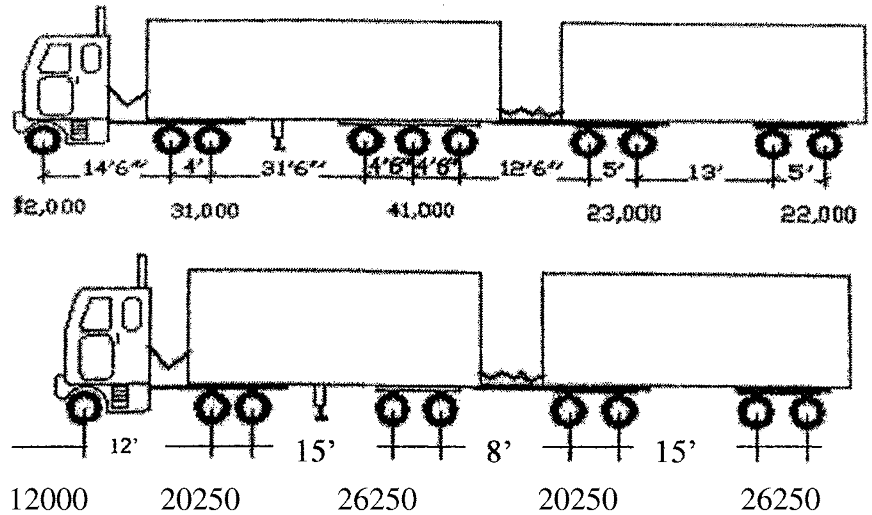


Fig. B-1.3.1 Truck Models Developed by Idaho Transportation Department

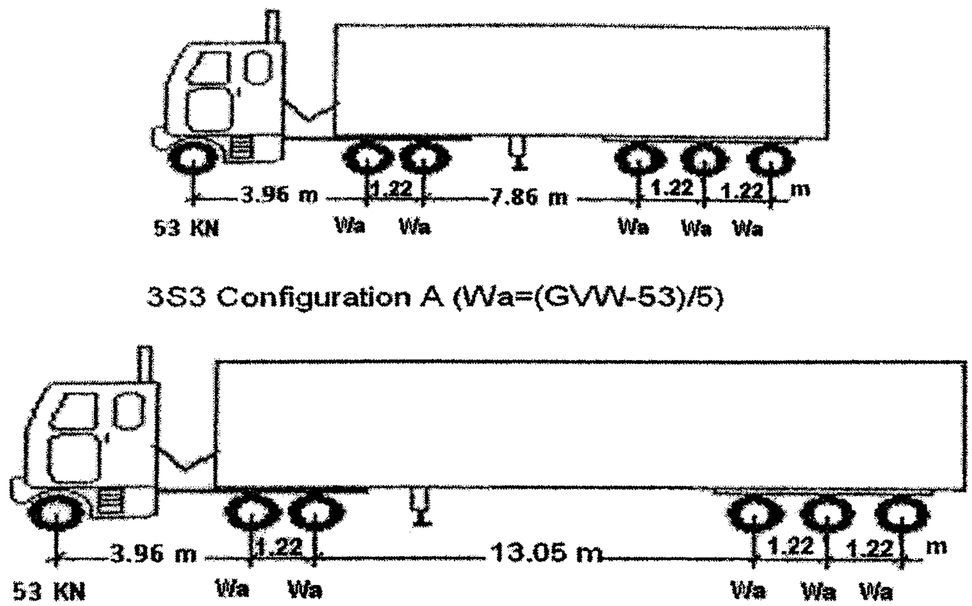


Fig. B-2.1.1 3S3 Truck Model under Alternative Scenario for the Michigan Example (1 kn = 0.225 kips, 1 m = 3.25 ft)

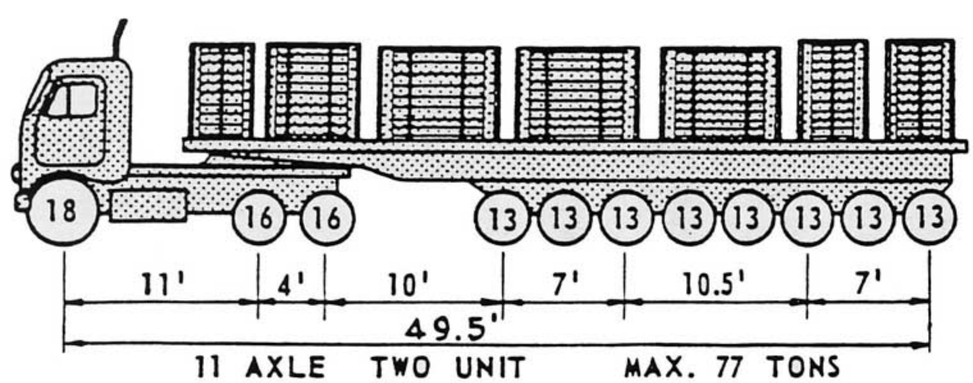


Fig. B-2.3.1 Michigan DOT Current Rating Vehicle (Axle Weights in kips)

Table B-1.1.1 Remaining Fatigue Life (Year) of Bridge 14520 for Idaho Example

	Scenario 1 (Stress Range Sr)		Scenario 2 (Stress Range Sr)		Scenario 3: 5 axle shifted to 105k* (Stress Range Sr)		Scenario 3: 5 axle shifted to 129 (Stress Range Sr)	
	Mena Life	Safe Life	Mena Life	Safe Life	Mena Life	Safe Life	Mena Life	Safe Life
Base Case	206 (1.92 ksi 13.3 MPa)	126	201 (1.98 ksi 13.7 Mpa)	120	201 (1.98 ksi 13.7 Mpa)	120	201 (1.98 ksi 13.7 Mpa)	120
Altenative Scenario	201 (1.99 ksi 13.7 MPa)	121	196 (2.06 ksi 14.2 MPa)	115	183 (2.22 ksi 15.3 Mpa)	104	174 (2.36 ksi 16.3 Mpa)	95

* See the 105k and 129k truck models in Fig.B-1.3.1

Table B-1.1.2 Sensitivity of Shifting Window Parameters for Scenario 2

Shifting Window Parameters	Expected Cost Impact
$a_1 = a_1 = 5\%$, $b_1 = b_2 = 10\%$	\$0.000
$a_1 = a_1 = 10\%$, $b_1 = b_2 = 20\%$ (reference case)	\$0.000
$a_1 = a_1 = 15\%$, $b_1 = b_2 = 30\%$	\$0.000

Table B-1.1.3 Sensitivity of Impact Factor for Scenario 2

Impact Factor I	Expected Cost Impact
I = 1.10	\$0.000
I = 1.15 (reference case)	\$0.000
I = 1.30	\$0.000

Table B-1.2.1 RC Deck Cost Impact for Idaho Example (Scenario 1)

Bridge ID	Year Built	Deck Area (sq ft)	Number of Bridges Represented	Deck Area Represented by the Bridge (sq ft)	Increased Pf	Expected Unit Cost ($/sq ft)	Expected Cost for the Group ($)
12560	1980	8566	8	31702	6.728E-03	0.12783	4052
13730	1976	4994	10	84385	5.448E-03	0.10351	8735
14520	1966	25925	8	75552	5.219E-03	0.09917	7492
15220	1956	2739	6*	20688	7.366E-03	0.13995	2895
16641	1995	15552	4	57121	1.252E-02	0.23782	13585
17565	1949	1587	3	5379	3.429E-03	0.06516	350
						Total	37110

* Another bridge (ID=17570) was used to check sensitivity of sampling, and the same result was obtained as that for bridge 15220.

Table B-1.2.2 RC Deck Cost Impact for Idaho Example (Scenario 2)

Bridge ID	Year Built	Deck Area (sq ft)	Number of Bridges Represented	Deck Area Represented by the Bridge (sq ft)	Increased Pf	Expected Unit Cost ($/sq ft)	Expected Cost for the Group ($)
12560	1980	8566	8	31702	1.780E-03	0.03382	1072
13730	1976	4994	10	84385	1.453E-03	0.02760	2329
14520	1966	25925	8	75552	2.936E-03	0.05578	4214
15220	1956	2739	6*	20688	2.967E-03	0.05637	1166
16641	1995	15552	4	57121	1.685E-03	0.03202	1829
17565	1949	1587	3	5379	0.000549168	0.01043	56
						Total	10667

* Another bridge (ID=17570) was used to check sensitivity of sampling, and the same result was obtained as that for bridge 15220

Table B-1.2.3 RC Deck Cost Impact for Idaho Example (Scenario 3)

Lower bound case: 5 axle truck shifting to 105 k truck in Fig.B-1.3.1							
Bridge ID	Year Built	Deck Area (sq ft)	Number of Bridges Represen te d	Deck Area Represen ted by the Bridge (sq ft)	Increased Pf	Expected Unit Cost ($/sq ft)	Expected Cost for the Group ($)
12560	1980	8566	8	31702	0.000E+00	0.00000	0
13730	1976	4994	10	84385	0.000E+00	0.00000	0
14520	1966	25925	8	75552	0.000E+00	0.00000	0
15220	1956	2739	6*	20688	0.000E+00	0.00000	0
16641	1995	15552	4	57121	0.000E+00	0.00000	0
17565	1949	1587	3	5379	0.000E+00	0.00000	0
						Total	0

Lower bound case: 5 axle truck shifting to 129 k truck in Fig.B-1.3.1							
Bridge ID	Year Built	Deck Area (sq ft)	Number of Bridges Represen te d	Deck Area Represen ted by the Bridge (sq ft)	Increased Pf	Expected Unit Cost ($/sq ft)	Expected Cost for the Group ($)
12560	1980	8566	8	31702	0.000E+00	0.00000	0
13730	1976	4994	10	84385	0.000E+00	0.00000	0
14520	1966	25925	8	75552	0.000E+00	0.00000	0
15220	1956	2739	6*	20688	0.000E+00	0.00000	0
16641	1995	15552	4	57121	0.000E+00	0.00000	0
17565	1949	1587	3	5379	0.000E+00	0.00000	0
						Total	0

* Another bridge (ID=17570) was used to check sensitivity of sampling, and the same result was obtained as that for bridge 15220.

Table B-1.3.1 Calculation Details for Overstress Deficiency in Existing Bridges For Idaho Example (to be continued)

		EXISTING RATING FACTORS:				RATING FACTORS UNDER ALTERNATIVE SCENARIO:							
						Scenario 1		Scenario 2		Scenario 3-105 kips		Scenario 3-129 kips	
Bridge ID		INVNTRY		OPRTNG		INVNTRY	OPRTNG	INVNTRY	OPRTNG	INVNTRY	OPRTNG	INVNTRY	OPRTNG
	Rating Vehicle	Type 3S2	Type 3-3	Type 3S2	Type 3-3	129 K	129K	129 K	129k	129 K	129 K	129 K	129 k
12015		1.332	1.332	2.147	2.031	1.233	1.706	1.348	1.866	1.325	1.834	1.147	1.588
12020		1.378	1.378	2.229	2.108	1.275	1.771	1.394	1.936	1.370	1.903	1.186	1.648
12560		1.051	1.141	1.751	1.902	0.621	1.034	0.675	1.125	0.651	1.084	0.532	**0.886**
12565		1.051	1.141	1.751	1.902	0.621	1.034	0.675	1.125	0.651	1.084	0.532	**0.886**
12570		1.241	1.523	2.062	2.577	1.118	1.876	1.216	2.041	1.226	2.056	1.064	1.784
12580		1.324	1.667	2.208	2.760	1.262	2.059	1.372	2.237	1.381	2.252	1.198	1.954
12585		0.890	0.867	1.484	1.445	NA	1.166	NA	1.258	NA	1.265	NA	1.098
12590		0.890	0.867	1.484	1.445	NA	1.166	NA	1.258	NA	1.265	NA	1.098
12595		1.500	1.886	2.500	3.143	1.444	2.407	1.559	2.599	1.567	2.612	1.360	2.267
12600		1.376	1.721	2.293	2.868	1.164	1.939	1.257	2.093	1.264	2.105	1.097	1.827
12605		1.345	1.614	2.238	2.686	1.150	1.913	1.241	2.065	1.249	2.077	1.084	1.803
12620		1.096	1.348	1.570	1.931	0.949	1.360	1.023	1.465	1.030	1.474	0.894	1.280
12625		0.647	0.747	1.399	1.616	NA	1.232	NA	1.327	NA	1.336	NA	1.160
12630		1.422	1.784	2.351	2.939	1.377	2.305	1.484	2.484	1.493	2.499	1.297	2.170
12635		1.011	1.257	2.086	2.147	0.928	1.549	1.011	1.687	0.986	1.646	0.853	1.423
12645		1.379	1.555	1.697	1.914	1.324	1.621	1.442	1.765	1.407	1.722	1.217	1.489
12650		1.368	1.639	1.739	2.084	1.201	1.527	1.289	1.639	1.296	1.647	1.124	1.429
12654		1.656	1.969	2.057	2.446	1.457	1.809	1.563	1.941	1.571	1.951	1.363	1.693
12660		0.796	1.000	1.316	1.645	NA	1.322	NA	1.423	NA	1.370	NA	1.183
12665		1.320	1.232	1.716	1.601	1.015	1.319	1.087	1.412	1.064	1.383	0.921	1.196
12970		0.794	1.000	2.065	2.581	NA	2.260	NA	2.287	NA	2.392	NA	2.212
12975		0.807	1.014	1.341	1.676	NA	1.206	NA	1.255	NA	1.238	NA	1.125
13040		0.894	0.954	1.490	1.591	NA	1.262	NA	1.264	NA	1.319	NA	1.218
13045		0.917	1.128	1.517	1.897	NA	1.552	NA	1.554	NA	1.622	NA	1.498
13150		1.303	1.478	2.171	2.462	1.227	2.044	1.314	2.190	1.253	2.089	1.081	1.802
13155		1.084	1.016	1.738	1.630	0.837	1.343	0.881	1.414	0.881	1.413	0.764	1.226
13160		2.900	3.846	4.800	6.385	3.259	5.394	3.488	5.773	3.303	5.468	2.848	4.713
13165		1.179	1.208	1.529	1.565	0.947	1.230	0.997	1.295	0.995	1.292	0.863	1.120
13170		1.667	1.578	2.083	1.972	1.369	1.710	1.441	1.801	1.439	1.798	1.247	1.559
13175		1.721	1.615	2.179	2.043	1.355	1.786	1.427	1.881	1.424	1.877	1.235	1.628
13180		1.478	1.617	1.930	2.110	1.290	1.683	1.358	1.773	1.355	1.769	1.175	1.534
13185		0.976	1.058	1.533	1.662	0.833	1.309	0.883	1.387	0.884	1.389	0.767	1.205
13190		0.992	0.969	1.674	1.634	0.774	1.306	0.821	1.384	0.821	1.385	0.712	1.201
13195		1.004	1.004	1.659	1.659	0.948	1.579	0.997	1.661	0.995	1.657	0.863	1.437
13635		1.381	1.726	2.304	2.881	1.106	1.845	1.255	2.093	1.240	2.068	0.985	1.642
13705		1.154	1.235	1.923	2.040	0.979	1.493	1.057	1.612	1.067	1.626	0.926	1.411
13710		1.280	1.280	1.875	1.841	1.025	1.433	1.118	1.562	1.099	1.536	0.952	1.330
13715		1.409	1.320	2.156	2.020	1.113	1.548	1.220	1.697	1.195	1.661	1.034	1.438
13720		0.948	0.905	1.419	1.355	NA	1.127	NA	1.225	NA	1.234	NA	1.072
13725		0.948	0.905	1.419	1.355	NA	1.127	NA	1.225	NA	1.234	NA	1.072

Table B-1.3.1 Calculation Details for Overstress Deficiency in Existing Bridges For Idaho Example (continued)

	EXISTING RATING FACTORS:				RATING FACTORS UNDER ALTERNATIVE SCENARIO:							
					Scenario 1		Scenario 2		Scenario 3-105 kips		Scenario 3-129 kips	
Bridge	INVNTRY		OPRTNG		INVNTRY	OPRTNG	INVNTRY	OPRTNG	INVNTRY	OPRTNG	INVNTRY	OPRTNG
ID	Type 3S2	Type 3-3	Type 3S2	Type 3-3	129 K	129K	129 K	129k	129 K	129 k	129 K	129 k
13730	0.948	0.905	1.419	1.355	NA	1.127	NA	1.225	NA	1.234	0.716	1.072
13740	1.407	1.370	1.788	1.742	1.014	1.289	1.102	1.401	1.111	1.411	0.964	1.225
13745	1.000	1.250	1.673	2.091	0.987	1.642	1.074	1.785	1.080	1.796	0.938	1.559
13750	1.232	1.386	2.033	2.295	1.053	1.745	1.160	1.923	1.137	1.885	0.984	1.631
13985	1.253	1.291	2.062	2.139	1.009	1.752	1.042	1.810	1.050	1.823	0.960	1.666
13990	2.122	2.636	3.572	4.466	1.909	3.207	1.922	3.229	2.015	3.385	1.864	3.131
13995	1.390	1.727	2.323	2.883	1.207	2.024	1.215	2.036	1.273	2.134	1.177	1.974
14000	1.069	1.126	1.780	1.875	0.968	1.592	0.999	1.645	0.994	1.635	0.905	1.489
14005	1.213	1.500	2.032	2.523	1.057	1.772	1.063	1.783	1.115	1.868	1.031	1.728
14010	1.056	1.284	1.754	2.130	0.928	1.546	0.934	1.556	0.979	1.631	0.905	1.508
14015	0.875	1.089	1.452	1.815	NA	1.467	NA	1.486	NA	1.557	NA	1.439
14020	1.458	1.443	1.832	1.814	1.207	1.516	1.246	1.566	1.255	1.577	1.147	1.442
14025	2.857	3.478	4.755	5.944	2.862	4.803	2.896	4.858	3.043	5.106	2.817	4.727
14030	1.317	1.497	1.988	2.260	1.183	1.786	1.197	1.807	1.258	1.899	1.165	1.758
14515	1.059	1.333	1.765	2.222	1.045	1.741	1.083	1.805	1.105	1.842	1.015	1.690
14520	1.405	1.362	2.341	2.271	1.075	1.791	1.115	1.857	1.137	1.895	1.043	1.739
14525	1.327	1.327	2.010	1.936	1.212	1.550	1.255	1.605	1.273	1.629	1.166	1.492
15220	1.286	1.362	2.117	2.180	1.172	1.876	1.181	1.890	1.239	1.983	1.146	1.834
15226	2.678	2.657	3.278	3.252	2.089	2.557	2.273	2.783	1.967	2.408	1.544	1.890
16631	1.354	1.566	2.256	2.605	1.177	1.955	1.259	2.091	1.190	1.977	1.013	1.683
16635	1.370	1.370	2.276	2.278	1.250	2.083	1.339	2.231	1.272	2.119	1.081	1.801
16641	1.455	1.419	2.425	2.365	0.938	1.564	1.023	1.706	0.993	1.656	0.840	1.400
16645	1.014	0.983	1.690	1.638	0.585	**0.974**	0.628	1.045	0.595	0.991	0.506	**0.843**
17450	1.318	1.657	2.204	2.755	1.192	1.975	1.277	2.115	1.212	2.007	1.030	1.706
17455	0.891	1.004	1.485	1.673	NA	1.299	NA	1.391	NA	1.320	NA	1.122
17460	0.763	0.951	1.257	1.572	NA	1.180	NA	1.264	NA	1.198	NA	1.019
17560	0.920	1.150	1.525	1.907	NA	1.507	NA	1.627	NA	1.636	NA	1.420
17565	0.753	0.753	1.249	1.249	NA	1.248	NA	1.347	NA	1.355	NA	1.176
17570	1.155	1.102	1.924	1.837	0.935	1.559	0.997	1.663	1.001	1.669	0.869	1.448
17576	1.453	1.409	1.823	1.769	1.110	1.393	1.186	1.489	1.191	1.495	1.034	1.297
17595	0.987	0.987	1.635	1.643	1.449	1.449	0.949	1.551	0.952	1.557	0.826	1.351
17600	1.124	1.124	1.874	1.874	1.053	1.620	1.122	1.727	1.126	1.734	0.977	1.504
17605	1.362	1.350	2.074	2.058	1.087	1.693	1.172	1.826	1.133	1.764	0.979	1.524

Note: [grey box] = Additional deficient bridges due to overstress under the Alternativce Scenario

Table B-1.3.2 Agency Specific New Bridge Cost - Idaho Transportation Department

	Bridge ID	Year Built	Total Cost in Y1998 Dollars
1	12831	1995	$67.42
2	14294	1995	$52.10
3	16631	1995	$63.85
4	18031	1995	$92.77
5	20011	1995	$73.06
6	21101	1995	$55.74
7	21526	1995	$66.32
8	26086	1995	$48.16
9	10201	1996	$98.87
10	12096	1996	$54.87
11	13621	1996	$38.13
12	13856	1996	$110.11
13	14297	1996	$56.13
14	15769	1996	$94.09
15	19706	1996	$44.10
16	21126	1996	$55.59
17	21661	1996	$62.38
18	22256	1996	$64.80
19	31686	1996	$137.93
20	10141	1997	$92.39
21	10396	1997	$91.67
22	13608	1997	$72.95
23	18446	1997	$94.36
24	22151	1997	$84.16
25	25341	1997	$66.07
26	26261	1997	$43.85
Average			$72.38

Table B-1.4.1 New Bridge Cost Impact for Idaho Example for Scenarios 1 and 2 (from 1993 to 1998 inclusive)

					Scenario 1			Scenario 2		
Bridge ID	Yr built	Max Span Length (m)	Area (sq m	Type	DLCF	Cost Increase Coefficie nt	Cost Impact	DLCF	Cost Increase Coefficie nt	Cost Impact
12654	1994	9	130	ss Concret	1.191	0.0482	$4,723	1.069	0.0173	$1,699
15226	1993	32	3168	Concrete (	1.191	0.0109	$25,966	1.069	0.0039	$9,343
16631	1995	6	192	reteslab S	1.191	0.0560	$8,103	1.069	0.0201	$2,915
16641	1995	28	1424	el Continu	1.477	0.1983	$212,781	1.325	0.1353	$145,192
17576	1994	30	576	ss Concret	1.285	0.0156	$6,749	1.153	0.0084	$3,631
						Total	$258,322		Total	$162,780

Average Annual Cost Impact	$258,322/ 6=	$43,054	$162,780/6=	$27,130
Total Cost for 20 Years		$861,080		$542,600

Table B-1.4.2 New Bridge Cost Impact for Idaho Example for Scenario 3

					Scenario 3 - shifting to 105k*			Scenario 3 - shifting to 129k*		
Bridge ID	Yr built	Max Span Length (m)	\rea (sq m	Type	DCLF	Cost Increase Coefficie nt	Cost Impact	DCLF	Cost Increase Coefficie nt	Cost Impact
12654	1994	9	130.0	;s Concret	1.060	0.0152	$1,485	1.248	0.0627	$6,139
15226	1993	32	3168.0	Concrete (	1.060	0.0034	$8,166	1.248	0.0141	$33,752
16631	1995	6	192.0	rete slab S	1.060	0.0176	$2,548	1.248	0.0728	$10,532
16641	1995	28	1424.0	el Continu	1.315	0.1312	$140,748	1.548	0.2279	$244,516
17576	1994	30	576.0	;s Concret	1.144	0.0079	$3,409	1.347	0.0189	$8,210
							$156,357			$303,148

Average Annual Cost Impact	$156,357/ 6=	$26,059	$303,148/6=	$50,525
Total Cost for 20 Years		$521,180		$1,010,500

* See new truck models in Fig. B-1.3.1

Table B-1.5.1 Summary of Expected Cost Impact for Idaho Example

Scenario 1 – Practiced Situation of 129K Permissible for the Pilot Routes

	Estimated Expected Costs
Cost Category 1: Steel fatigue	$0
Cost Category 2: RC deck fatigue	$37,100
Cost Category 3: Deficient existing bridges	$6,000 ($1,032,200 if replacement)
Cost Category 4: New deficient bridges	$861,100
Total	$904,200 in Y1998 dollars ($1,930,400 if replacement for Category 3)

Scenario 2 – Hypothetical Situation of 129K Permissible for the Entire State

	Estimated Expected Costs
Cost Category 1: Steel fatigue	$0
Cost Category 2: RC deck fatigue	$10,700
Cost Category 3: Deficient existing bridges	$0
Cost Category 4: New deficient bridges	$542,600
Total	$552,300 in Y1998 dollars

Scenario 3 - Hypothetical Situation of 129K Legalized for the Entire State

	Estimated Expected Costs
Cost Category 1: Steel fatigue	$0
Cost Category 2: RC deck fatigue	$0
Cost Category 3: Deficient existing bridges	$0 to $18,000 ($0 to $2,083,100 if replacement)
Cost Category 4: New deficient bridges	$521,200 to $1,010,500
Total	$521,200 to $1,028,500 Y1998 dollars ($521,200 to $3,093,600 if replacement for Category 3)

Table B-1.6.1 Percentage of Trucks Exceeding Inventory Rating - Scenario 3

Bridge ID	Functional Class	5 axle trucks shifting to 105k	5 axle trucks shifting to 129k	Bridge ID	Functional Class	5 axle trucks shifting to 105k	5 axle trucks shifting to 129k
12015	2	0.00%	0.54%	13730	2	21.09%	19.59%
12020	2	0.00%	0.18%	13740	2	0.45%	15.02%
12560	14	13.07%	11.30%	13745	2	0.45%	15.02%
12565	14	13.07%	11.30%	13750	2	0.00%	10.54%
12570	2	0.00%	2.62%	13985	7	0.33%	11.55%
12580	2	0.00%	0.18%	13990	7	0.00%	0.00%
12585	2	21.77%	19.59%	13995	7	0.00%	0.14%
12590	2	21.77%	19.59%	14000	7	1.03%	14.06%
12595	2	0.00%	0.00%	14005	7	0.00%	4.85%
12600	2	0.00%	1.70%	14010	7	1.03%	14.06%
12605	2	0.00%	2.62%	14015	7	2.22%	14.68%
12620	2	1.42%	18.32%	14020	7	0.00%	0.43%
12625	2	21.77%	19.59%	14025	7	0.00%	0.00%
12630	2	0.00%	0.00%	14030	7	0.00%	0.14%
12635	2	3.06%	19.01%	14515	7	0.00%	4.85%
12645	2	0.00%	0.00%	14520	7	0.00%	4.85%
12650	2	0.00%	0.54%	14525	7	0.00%	0.14%
12654	2	0.00%	0.00%	15220	7	0.00%	0.43%
12660	2	21.09%	19.59%	15226	17	0.00%	0.00%
12665	2	1.42%	18.32%	16631	6	0.00%	5.00%
12970	7	0.00%	0.43%	16635	6	0.00%	2.12%
12975	7	15.85%	15.14%	16641	6	5.58%	15.86%
13040	7	15.35%	15.14%	16645	6	17.95%	15.86%
13045	7	1.03%	14.06%	17450	6	0.00%	5.00%
13150	2	0.00%	2.62%	17455	6	17.95%	15.86%
13155	2	18.92%	19.59%	17460	6	17.95%	15.86%
13160	2	0.00%	0.00%	17560	2	3.06%	19.01%
13165	2	3.06%	19.01%	17565	2	21.09%	19.59%
13170	2	0.00%	0.00%	17570	2	3.06%	19.01%
13175	2	0.00%	0.00%	17576	2	0.00%	6.16%
13180	2	0.00%	0.18%	17595	2	6.77%	19.59%
13185	2	18.92%	19.59%	17600	2	0.00%	10.54%
13190	2	21.09%	19.59%	17605	2	0.00%	10.54%
13195	2	3.06%	19.01%				
13635	14	0.00%	5.67%				
13705	2	1.42%	18.32%				
13710	2	0.45%	15.02%				
13715	2	0.00%	6.16%				
13720	2	21.09%	19.59%				
13725	2	21.09%	19.59%				

Table B-2.1.1 Sample Bridges for Michigan Example
Cost Impact Analysis: Steel Bridge Fatigue

Order	BridgeID	Represent'd Group *by Year Built*	*by Max. Span Length*	*by Jurisdiction*	*by Beam Type*	*by Span Type*	Number of Bridges in the Group	How Many Bridges Represent the Group?	Focused Details Exist? / Remarks
1	975	1961-70	50-60m	Local	PlateGirder	Continuous	1	1	Yes
2	994	1961-70	40-50m	Local	PlateGirder	Continuous	2	1	Yes
3	12344	<1941	<10m	Local	RolledBeam	Continuous	18	1	No
4	12240	<1941	10-20m	Local	RolledBeam	Continuous	7	1	No
5	11988	1941-50	<10m	Local	RolledBeam	Continuous	5	1	No
6	5578	1941-50	10-20m	Local	RolledBeam	Continuous	5	1	No
7	12282	1951-60	<10m	Local	RolledBeam	Continuous	1	1	No
8	9244	1951-60	10-20m	Local	RolledBeam	Continuous	12	1	No
9	12032	1951-60	20-30m	Local	RolledBeam	Continuous	3	1	No
10	6998	1961-70	<10m	Local	RolledBeam	Continuous	3	1	To Be Replaced
11	1136	1961-70	10-20m	Local	RolledBeam	Continuous	6	1	No
12	12321	1961-70	20-30m	Local	RolledBeam	Continuous	5	1	Yes
13	12089	1971-80	<10m	Local	RolledBeam	Continuous	2	1	No
14	12077	1971-80	10-20m	Local	RolledBeam	Continuous	2	1	No
15	9273	1971-80	20-30m	Local	RolledBeam	Continuous	5	1	Yes
16	5198	1971-80	30-40m	Local	RolledBeam	Continuous	1	1	No
17	12207	1951-69	>40m	Local	PlateGirder	Simple	1	1	No
18	2749	1961-70	>40m	Local	PlateGirder	Simple	1	1	No
19	1678	1971-80	>40m	Local	PlateGirder	Simple	1	1	No
20	5193	1971-80	40-50m	Local	RolledBeam	Simple	1	1	No
21	2756	1971-80	30-40m	Local	RolledBeam	Simple	1	1	
22	6556	1971-80	20-30m	Local	RolledBeam	Simple	20	1	No Plans Available
23	9298	1971-80	10-20m	Local	RolledBeam	Simple	53	1	No
24	771	1971-80	<10m	Local	RolledBeam	Simple	21	1	No
25	5565	1961-70	20-30m	Local	RolledBeam	Simple	17	1	Yes
26	10624	1961-70	<10m	Local	RolledBeam	Simple	87(A)	2	No
27	6333	1961-70	<10m	Local	RolledBeam	Simple	87(B)	2	No
28	10663	1961-70	10-20m	Local	RolledBeam	Simple	123(A)	2	No
29	5437	1961-70	10-20m	Local	RolledBeam	Simple	123(B)	2	No
30	6878	1951-60	<10m	Local	RolledBeam	Simple	36	1	No
31	5918	1951-60	10-20m	Local	RolledBeam	Simple	94(A)	2	No
32	9294	1951-60	10-20m	Local	RolledBeam	Simple	94(B)	2	No
33	1048	1951-60	20-30m	Local	RolledBeam	Simple	20	1	Yes
34	10614	1941-50	<10m	Local	RolledBeam	Simple	71(A)	2	No
35	4141	1941-50	<10m	Local	RolledBeam	Simple	71(B)	2	To Be Repalced
36	3873	1941-50	10-20m	Local	RolledBeam	Simple	104(A)	2	No
37	7256	1941-50	10-20m	Local	RolledBeam	Simple	104(B)	2	No
38	4632	1941-50	20-30m	Local	RolledBeam	Simple	9	1	No
39	5126	1931-40	20-30m	Local	RolledBeam	Simple	7	1	No
40	2764	1931-40	10-20m	Local	RolledBeam	Simple	176(A)	2	No
41	6993	1931-40	10-20m	Local	RolledBeam	Simple	176(B)	2	No Plans Available
42	12662	1931-40	<10m	Local	RolledBeam	Simple	143(A)	2	No
43	5038	1931-40	<10m	Local	RolledBeam	Simple	143(B)	2	No Plans Available
44	4132	<1930	<10m	Local	RolledBeam	Simple	306(A)	2	No
45	3923	<1930	<10m	Local	RolledBeam	Simple	306(B)	2	No
46	5624	<1930	10-20m	Local	RolledBeam	Simple	180(A)	2	No Plans Available
47	1395	<1930	10-20m	Local	RolledBeam	Simple	180(B)	2	No

48	12222	<1930	20-30m	Local	RolledBeam	Simple	6	1	No
49	5327	1971-80	40-50m	State	PlateGirder	Continuous	12	1	No
50	11467	1961-70	60-70m	State	PlateGirder	Continuous	1	1	No Plans Available
51	8336	1971-80	70-80m	State	PlateGirder	Continuous	2	1	No
52	11840	1971-80	50-60m	State	PlateGirder	Continuous	2	1	No
53	11716	1961-70	>90m	State	PlateGirder	Continuous	1	1	Yes
54	11505	1961-70	50-60m	State	PlateGirder	Continuous	1	1	Yes
55	11850	1961-70	40-50m	State	PlateGirder	Continuous	5	1	No
56	7129	<1940	10-20m	State	RolledBeam	Continuous	3	1	No
57	9115	<1940	>40m	State	RolledBeam	Continuous	1	1	No
58	11406	1941-50	10-20m	State	RolledBeam	Continuous	4	1	No
59	11444	1951-60	10-20m	State	RolledBeam	Continuous	19	1	No
60	11431	1951-60	20-30m	State	RolledBeam	Continuous	10	1	No
61	2646	1961-70	20-30m	State	RolledBeam	Continuous	31	1	No
62	635	1961-70	30-40m	State	RolledBeam	Continuous	7	1	No
63	11732	1961-70	10-20m	State	RolledBeam	Continuous	41	1	Yes
64	2093	1971-80	10-20m	State	RolledBeam	Continuous	1	1	No
65	11575	1971-80	20-30m	State	RolledBeam	Continuous	13	1	No
66	9986	1941-50	40-50m	State	PlateGirder	Simple	1	1	No
67	7173	1951-60	40-50m	State	PlateGirder	Simple	2	1	No
68	1241	1961-70	40-50m	State	PlateGirder	Simple	24	1	No
69	6106	1971-80	50-60m	State	PlateGirder	Simple	13	1	No
70	11944	1971-80	40-50m	State	PlateGirder	Simple	86(A)	2	Yes
71	6708	1971-80	40-50m	State	PlateGirder	Simple	86(B)	2	No
72	3449	<1940	10-20m	State	RolledBeam	Simple	161(A)	2	No
73	2095	<1940	10-20m	State	RolledBeam	Simple	161(B)	2	No
74	3127	<1940	20-30m	State	RolledBeam	Simple	5	1	No Plans Available
75	384	<1940	<10m	State	RolledBeam	Simple	15	1	No
76	9478	1941-50	20-30m	State	RolledBeam	Simple	7	1	No
77	3462	1941-50	<10m	State	RolledBeam	Simple	9	1	No Plans Available
78	2949	1941-50	10-20m	State	RolledBeam	Simple	87(A)	2	No
79	94	1941-50	10-20m	State	RolledBeam	Simple	87(B)	2	No
80	882	1951-60	<10m	State	RolledBeam	Simple	2	1	No Plans Available
81	11246	1951-60	20-30m	State	RolledBeam	Simple	128(A)	2	No
82	2507	1951-60	20-30m	State	RolledBeam	Simple	128(B)	2	No
83	11192	1951-60	10-20m	State	RolledBeam	Simple	243(A)	2	No
84	4440	1951-60	10-20m	State	RolledBeam	Simple	243(B)	2	No
85	3770	1961-70	20-30m	State	RolledBeam	Simple	500(A)	2	No Plans Available
86	1209	1961-70	20-30m	State	RolledBeam	Simple	500(B)	2	Yes
87	7833	1961-70	10-20m	State	RolledBeam	Simple	339(A)	2	Yes
88	9158	1961-70	10-20m	State	RolledBeam	Simple	339(B)	2	Yes
89	9753	1961-70	30-40m	State	RolledBeam	Simple	8	1	Yes
90	4242	1961-70	<10m	State	RolledBeam	Simple	6	1	No
91	8431	1971-80	30-40m	State	RolledBeam	Simple	6	1	Yes
92	9143	1971-80	20-30m	State	RolledBeam	Simple	16	1	Yes
93	2583	1971-80	10-20m	State	RolledBeam	Simple	8	1	No Plans Available

A and B: There is another bridge representing the same group
1 m = 3.28 ft

Table B-2.1.2 Steel Fatigue Cost Impact for Michigan Example
(a1=a2=10%, b1=b2=20%, I=15%, traffuc growth factor u from NBI)

Bridge ID	Number of Bridges in Group	Functional Class	Remaining Mean Life for BC (Yr)	Remaining Safe Life for BC (Yr)	Remaining Mean Life for AS (Yr)	Remaining Safe Life for AS (Yr)	Increased Pf	Expected Cost for Group
975	1	16	191	50	182	45	9.00E-04	$1
994	2	7	122	69	120	67	7.64E-08	$0
1048	20	16	1250	697	1220	671	0.00E+00	$0
1209	500(B)	11	232	67	221	61	2.44E-04	$115
5565	17	7	875	597	865	587	0.00E+00	$0
7833	339(A)	12	140	6	125	2	1.63E-02	$4,272
8431	6	1	185	115	181	111	4.75E-14	$0
9143	16	11	642	277	621	262	4.02E-12	$0
9158	339(B)	19	290	211	282	204	0.00E+00	$0
9273	5	7	109	65	108	63	7.56E-09	$0
9753	8	7	107	79	106	78	0.00E+00	$0
11505	1	14	22	-11	20	-13	2.87E-02	$92
11716	1	11	106	1	97	-2	1.69E-02	$23
11732	41	11	120	3	110	0	1.45E-02	$1,037
11944	86(A)	17	223	158	219	154	0.00E+00	$0
12321	5	14	49	1	46	0	2.71E-02	$284
	Total							
	755						Y1998 cost	$5,824
							Y2000 cost	$6,179

A and B: There is another bridge which jointly represents the same group of bridges

Table B-2.1.3 Steel Fatigue Cost Impact for Michigan Example
Sensitivity Analysis (a1=a2=5%, b1=b2=10%, I=15%, traffic growth factor u from NBI)

Bridge ID	Number of Bridges in Group	Functional Class	Remaining Mean Life for BC (Yr)	Remaining Safe Life for BC (Yr)	Remaining Mean Life for AS (Yr)	Remaining Safe Life for AS (Yr)	Increased Pf	Expected Cost for Group
975	1	16	191	50	187	48	3.27E-04	$0
994	2	7	122	69	121	68	2.74E-08	$0
1048	20	16	1250	697	1237	686	0.00E+00	$0
1209	500(B)	11	232	67	226	64	1.10E-04	$52
5565	17	7	875	597	871	593	0.00E+00	$0
7833	339(A)	12	140	6	134	5	6.36E-03	$1,671
8431	6	1	185	115	183	113	1.32E-14	$0
9143	16	11	642	277	631	269	1.27E-12	$0
9158	339(B)	19	290	211	286	208	0.00E+00	$0
9273	5	7	109	65	108	64	2.64E-09	$0
9753	8	7	107	79	107	78	0.00E+00	$0
11505	1	14	22	-11	21	-12	1.20E-02	$39
11716	1	11	106	1	101	-1	8.45E-03	$11
11732	41	11	120	3	115	2	7.26E-03	$517
11944	86(A)	17	223	158	221	156	0.00E+00	$0
12321	5	14	49	1	48	1	1.10E-02	$115
	Total							
	755						Y1998 cost	$2,406
							Y2000 cost	$2,553

A and B: There is another bridge which jointly represents the same group of bridges

Table B-2.1.4 Steel Fatigue Cost Impact for Michigan Example
Sensitivity Analysis (a1=a2=15%, b1=b2=30%, I=15%, traffic growth factor u from NBI)

Bridge ID	Number of Bridges in Group	Functional Class	Remaining Mean Life for BC (Yr)	Remaining Safe Life for BC (Yr)	Remaining Mean Life for AS (Yr)	Remaining Safe Life for AS (Yr)	Increased Pf	Expected Cost for Group
975	1	16	191	50	178	43	1.37E-03	$2
994	2	7	122	69	119	66	1.18E-07	$0
1048	20	16	1250	697	1208	660	0.00E+00	$0
1209	500(B)	11	232	67	218	59	3.70E-04	$174
5565	17	7	875	597	862	584	0.00E+00	$0
7833	339(A)	12	140	6	119	1	2.35E-02	$6,182
8431	6	1	185	115	179	109	1.13E-13	$0
9143	16	11	642	277	613	257	8.17E-12	$0
9158	339(B)	19	290	211	279	201	0.00E+00	$0
9273	5	7	109	65	107	63	1.19E-08	$0
9753	8	7	107	79	106	77	0.00E+00	$0
11505	1	14	22	-11	19	-13	3.97E-02	$128
11716	1	11	106	1	94	-3	2.36E-02	$32
11732	41	11	120	3	107	-1	2.04E-02	$1,452
11944	86(A)	17	223	158	218	153	0.00E+00	$0
12321	5	14	49	1	45	-1	3.81E-02	$400
	Total							
	755						Y1998 cost	$8,369
							Y2000 cost	$8,879

A and B: There is another bridge which jointly represents the same group of bridges

Table B-2.1.5 Steel Fatigue Cost Impact for Michigan Example
Sensitivity Analysis (a1=a2=10%, b1=b2=20%, I=10%, traffic growth factor u from NBI)

Bridge ID	Number of Bridges in Group	Functional Class	Remaining Mean Life for BC (Yr)	Remaining Safe Life for BC (Yr)	Remaining Mean Life for AS (Yr)	Remaining Safe Life for AS (Yr)	Increased Pf	Expected Cost for Group
975	1	16	206	58	196	53	4.38E-04	$1
994	2	7	126	73	124	71	1.49E-08	$0
1048	20	16	1298	741	1268	714	0.00E+00	$0
1209	500(B)	11	249	77	238	71	1.00E-04	$47
5565	17	7	898	620	889	610	0.00E+00	$0
7833	339(A)	12	162	12	145	7	1.25E-02	$3,292
8431	6	1	191	120	187	117	2.66E-15	$0
9143	16	11	677	303	655	287	2.28E-13	$0
9158	339(B)	19	296	218	289	211	0.00E+00	$0
9273	5	7	113	69	111	67	1.20E-09	$0
9753	8	7	109	81	108	80	0.00E+00	$0
11505	1	14	25	-9	23	-11	3.04E-02	$98
11716	1	11	122	5	112	2	1.38E-02	$19
11732	41	11	138	8	127	5	1.15E-02	$823
11944	86(A)	17	228	163	225	160	0.00E+00	$0
12321	5	14	54	4	51	3	2.43E-02	$255
	Total							
	755						Y1998 cost	$4,534
							Y2000 cost	$4,811

A and B: There is another bridge which jointly represents the same group of bridges

Table B-2.1.6 Steel Fatigue Cost Impact for Michigan Example
Sensitivity Analysis (a1=a2=10%, b1=b2=20%, I=30%, traffic growth factor u from NBI)

Bridge ID	Number of Bridges in Group	Functional Class	Remaining Mean Life for BC (Yr)	Remaining Safe Life for BC (Yr)	Remaining Mean Life for AS (Yr)	Remaining Safe Life for AS (Yr)	Increased Pf	Expected Cost for Group
975	1	16	152	29	143	25	4.14E-03	$6
994	2	7	109	57	107	55	3.64E-06	$0
1048	20	16	1117	582	1088	557	0.00E+00	$0
1209	500(B)	11	186	42	177	37	1.68E-03	$789
5565	17	7	810	534	801	524	0.00E+00	$0
7833	339(A)	12	92	-6	80	-8	2.74E-02	$7,207
8431	6	1	168	99	164	95	5.21E-11	$0
9143	16	11	549	213	528	200	2.85E-09	$0
9158	339(B)	19	271	194	264	186	0.00E+00	$0
9273	5	7	99	55	97	54	6.05E-07	$0
9753	8	7	100	72	99	71	7.66E-15	$0
11505	1	14	12	-16	10	-17	2.36E-02	$76
11716	1	11	69	-9	62	-11	2.48E-02	$34
11732	41	11	79	-8	72	-10	2.27E-02	$1,620
11944	86(A)	17	208	143	204	139	0.00E+00	$0
12321	5	14	36	-6	34	-8	3.01E-02	$316
	Total							
	755						Y1998 cost	$10,047
							Y2000 cost	$10,659

A and B: There is another bridge which jointly represents the same group of bridges

Table B-2.1.7 Steel Fatigue Cost Impact for Michigan Example
Sensitivity Analysis (a1=a2=10%, b1=b2=20%, I=15%, traffic growth factor u from NBI plus 3%)

Bridge ID	Number of Bridges in Group	Functional Class	Remaining Mean Life for BC (Yr)	Remaining Safe Life for BC (Yr)	Remaining Mean Life for AS (Yr)	Remaining Safe Life for AS (Yr)	Increased Pf	Expected Cost for Group
975	1	16	64	24	62	22	5.09E-03	$7
994	2	7	65	38	64	37	3.16E-05	$0
1048	20	16	170	120	167	118	0.00E+00	$0
1209	500(B)	11	80	37	78	35	4.42E-04	$208
5565	17	7	189	144	188	142	0.00E+00	$0
7833	339(A)	12	54	10	51	8	2.58E-02	$6,786
8431	6	1	93	62	92	61	1.75E-10	$0
9143	16	11	128	81	126	78	5.03E-11	$0
9158	339(B)	19	135	102	132	99	0.00E+00	$0
9273	5	7	66	41	65	40	1.67E-06	$0
9753	8	7	69	50	68	49	1.40E-10	$0
11505	1	14	11	-11	10	-12	1.87E-02	$60
11716	1	11	47	6	45	4	2.48E-02	$34
11732	41	11	51	8	48	6	2.20E-02	$1,568
11944	86(A)	17	114	84	113	83	0.00E+00	$0
12321	5	14	27	0	26	-1	3.44E-02	$361
	Total							
	755						Y1998 cost	$9,024
							Y2000 cost	$9,573

A and B: There is another bridge which jointly represents the same group of bridges

Table B-2.3.1 Cost Impact for Deficiency of Existing Bridges for Michigan Example (a1=a2=10%,b1=b2=20%)

	Bridge ID	Width(m)	Length(m)	RF_{AS} (3S3A)	RF_{AS} (3S3B)	Cost	Cost if 5% allowance is given
1	708	8.6	6.7	0.9995	0.9981	$53,541	$0
2	739	8.4	6.7	0.9835	0.9821	$52,295	$0
3	1381	7.9	6.7	0.9467	0.9454	$49,183	$49,183
4	4020	11.6	7	0.9850	0.9837	$75,451	$0
5	4154	6.4	6.7	0.9467	0.9454	$39,844	$39,844
6	5974	7.8	6.1	0.9467	0.9454	$44,211	$44,211
7	7301	26.8	14	0.9028	0.8624	$348,636	$348,636
8	7643	10.9	7	0.9896	0.9883	$70,898	$0
9	7657	10.5	7	0.9995	0.9981	$68,296	$0
10	8198	8.5	6.7	0.9467	0.9454	$52,918	$52,918
11	10132	6.4	7.6	0.9467	0.9454	$45,196	$45,196
12	10144	9.2	7.3	0.9467	0.9454	$62,405	$62,405
13	10165	12.5	7	0.9898	0.9889	$81,305	$0
14	10178	8.6	7.3	0.9467	0.9454	$58,335	$58,335
15	10218	5.4	6.1	0.9467	0.9454	$30,608	$30,608
						Total $1,133,122	Total $731,336

Table B-2.3.2 Cost Impact for Deficiency in Existing Bridges for Michigan Example Sensitivity Analysis (a1=a2=5%,b1=b2=15%)

	Bridge ID	Width(m)	Length(m)	RF_{AS}(3S3A)	RF_{AS}(3S3B)	Cost	Cost if 5% allowance is given
1	739	8.4	6.7	1.0013	0.9999	$52,295	$0
2	1381	7.9	6.7	0.9839	0.9826	$49,183	$0
3	4154	6.4	6.7	0.9839	0.9826	$39,844	$0
4	5974	7.8	6.1	0.9839	0.9826	$44,211	$0
5	7301	26.8	14	0.9172	0.8761	$348,636	$348,636
6	8198	8.5	6.7	0.9839	0.9826	$52,918	$0
7	10132	6.4	7.6	0.9839	0.9826	$45,196	$0
8	10144	9.2	7.3	0.9839	0.9826	$62,405	$0
9	10178	8.6	7.3	0.9839	0.9826	$58,335	$0
10	10218	5.4	6.1	0.9839	0.9826	$30,608	$0
						Total $783,632	Total $348,636

Table B-2.3.3 Cost Impact for Deficiency in Existing Bridges for Michigan Example Sensitivity Analysis (a1=a2=15%,b1=b2=30%)

	Bridge ID	Width(m)	Length(m)	RF_{AS} (3S3A)	RF_{AS} (3S3B)	Cost	Cost if 5% allowance is given
1	708	8.6	6.7	0.9950	0.9936	$53,541	$0
2	739	8.4	6.7	0.9765	0.9752	$52,295	$0
3	1381	7.9	6.7	0.9270	0.9257	$49,183	$49,183
4	3959	20.1	7	0.9986	0.9972	$130,738	$0
5	4020	11.6	7	0.9793	0.9780	$75,451	$0
6	4154	6.4	6.7	0.9270	0.9257	$39,844	$39,844
7	5068	7.9	6.1	0.9989	0.9976	$44,778	$0
8	5974	7.8	6.1	0.9270	0.9257	$44,211	$44,211
9	7301	26.8	14	0.8993	0.8591	$348,636	$348,636
10	7643	10.9	7	0.9709	0.9696	$70,898	$0
11	7657	10.5	7	0.9950	0.9936	$68,296	$0
12	8198	8.5	6.7	0.9270	0.9257	$52,918	$52,918
13	10098	10.2	6.4	0.9989	0.9975	$60,658	$0
14	10100	7.3	6.7	0.9965	0.9957	$45,447	$0
15	10132	6.4	7.6	0.9270	0.9257	$45,196	$45,196
16	10144	9.2	7.3	0.9270	0.9257	$62,405	$62,405
17	10150	5.4	7	0.9965	0.9957	$35,124	$0
18	10165	12.5	7	0.9641	0.9633	$81,305	$0
19	10178	8.6	7.3	0.9270	0.9257	$58,335	$58,335
20	10218	5.4	6.1	0.9270	0.9257	$30,608	$30,608
21	10222	4.5	6.7	0.9965	0.9957	$28,015	$0
22	10231	6.4	7.6	0.9965	0.9957	$45,196	$0
23	12112	18.7	7.3	0.9955	0.9941	$126,845	$0
						Total $1,649,925	Total $731,336

Table B-2.4.1 Cost Impact for New Bridges for Michigan Example (Using 1999 data for projection)

	Bridge ID	DLCF	Bridge Material	Max Span Length (m)	Max Span Length (ft)	Deck Area (sq m)	Deck Area (sq ft)	Cost Increase Coefficient	Cost Impact
1	3267	1.00	Prestressed Concrete Beam	14	46	361.2	3888	0.00	$0
2	9728	1.00	Prestressed Concrete Beam	42.6	140	1209.8	13023	0.00	$0
3	7799	1.00	Prestressed Concrete Beam	34.3	113	1030.9	11096	0.00	$0
4	8762	1.00	Prestressed Concrete Beam	20	66	608.0	6544	0.00	$0
5	12831	1.00	Prestressed Concrete Beam	42.1	138	2106.2	22671	0.00	$0
6	4382	1.00	Prestressed Concrete Beam	16.8	55	240.2	2586	0.00	$0
7	12812	1.00	Prestressed Concrete Beam	19.2	63	1468.3	15805	0.00	$0
8	12813	1.00	Prestressed Concrete Beam	19.2	63	1010.8	10880	0.00	$0
9	12814	1.00	Prestressed Concrete Beam	39	128	553.8	5961	0.00	$0
10	12816	1.00	Prestressed Concrete Beam	40.6	133	1092.0	11754	0.00	$0
11	12817	1.00	Prestressed Concrete Beam	40.6	133	1092.0	11754	0.00	$0
12	12818	1.00	Prestressed Concrete Beam	39	128	553.8	5961	0.00	$0
13	12825	1.00	Prestressed Concrete Beam	52.3	172	2055.4	22124	0.00	$0
14	5313	1.00	Prestressed Concrete Beam	18.6	61	293.8	3162	0.00	$0
15	162	1.00	Prestressed Concrete Beam	27	89	1031.4	11102	0.00	$0
16	140	1.00	Prestressed Concrete Beam	16.6	54	415.0	4467	0.00	$0
17	2609	1.00	Culvert	8.5	28	139.4	1500	0.00	$0
18	545	1.00	Prestressed Concrete Beam	20.3	67	671.6	7229	0.00	$0
19	4553	1.00	Prestressed Concrete Beam	22.2	73	1144.5	12320	0.00	$0
20	9966	1.00	Prestressed Concrete Beam	35.6	117	935.8	10073	0.00	$0
21	7840	1.00	Prestressed Concrete Beam	42.1	138	1810.1	19483	0.00	$0
22	7614	1.00	Prestressed Concrete Beam	19	62	785.4	8454	0.00	$0
23	2965	1.00	Prestressed Concrete Beam	34.3	113	967.3	10412	0.00	$0
24	10282	1.00	Prestressed Concrete Beam	27.1	89	422.8	4551	0.00	$0
25	65	1.00	Timber	8.2	27	62.3	671	0.00	$0
26	1968	1.00	Prestressed Concrete Beam	29.8	98	313.1	3370	0.00	$0
27	5874	1.00	Prestressed Concrete Beam	22.8	75	336.0	3617	0.00	$0
28	6268	1.00	Culvert	9.7	32	134.8	1451	0.00	$0
29	6312	1.00	Culvert	6.1	20	113.5	1221	0.00	$0
30	6428	1.00	Culvert	9.8	32	0.0	0	0.00	$0
31	8220	1.00	Culvert	12.1	40	158.8	1709	0.00	$0
32	10746	1.00	Prestressed Concrete Beam	26.3	86	319.0	3434	0.00	$0
								Total	$0

Table B-2.4.2 Cost Impact for New Bridges for Michigan Example (Using 1998 data for projection)

	Bridge ID	DLCF	Bridge Material	Max Span Length (m)	Max Span Length (ft)	Deck Area (sq m)	Deck Area (sq ft)	Cost Increase Coefficient	Cost Impact
1	2881	1.00	Prestressed Concrete Beam	27	88.6	383.4	4127	0.00	$0
2	5501	1.00	Prestressed Concrete Beam	19.8	65.0	281.2	3026	0.00	$0
3	2512	1.00	Prestressed Concrete Beam	29.2	95.8	2218.7	23882	0.00	$0
4	5505	1.00	Prestressed Concrete Beam	20.2	66.3	266.6	2870	0.00	$0
5	9776	1.00	Culvert	3	9.8	46.5	501	0.00	$0
6	12778	1.00	Prestressed Concrete Beam	35.6	116.8	535.7	5766	0.00	$0
7	12779	1.00	Prestressed Concrete Beam	35.6	116.8	535.7	5766	0.00	$0
8	799	1.00	Prestressed Concrete Beam	24	78.7	1674.0	18019	0.00	$0
9	12750	1.00	Prestressed Concrete Beam	27.4	89.9	411.8	4433	0.00	$0
10	12751	1.00	Prestressed Concrete Beam	27.4	89.9	383.2	4125	0.00	$0
11	12752	1.00	Prestressed Concrete Beam	36.1	118.4	3968.6	42718	0.00	$0
12	12753	1.00	Prestressed Concrete Beam	36.1	118.4	3595.7	38704	0.00	$0
13	6789	1.00	Prestressed Concrete Beam	20.9	68.6	453.1	4877	0.00	$0
14	1212	1.00	Prestressed Concrete Beam	18.6	61.0	956.3	10294	0.00	$0
15	7877	1.00	Prestressed Concrete Beam	23	75.5	1141.1	12283	0.00	$0
16	11129	1.00	Prestressed Concrete Beam	32.3	106.0	692.9	7458	0.00	$0
17	11333	1.00	Prestressed Concrete Beam	19.6	64.3	2843.3	30605	0.00	$0
								Total	$0

Table B-2.4.3 Sensitivity Analysis For TWH Prediction Window Parameters (Michigan Example)

	Window Parameters	Mean W* (kips)	Standard Deviation s* (kips)	AFdesign	
Base Case		90.99	17.34		
Alternative Scenario	a1=a2=15%, b1=b2=30%	97.35	15.57	1.002	
	a1=a2=10%, b1=b2=20%	96.29	15.62	0.996	(set equal to 1)
	a1=a2=5%, b1=b2=10%	93.37	16.93	1.006	

Abbreviations used without definitions in TRB publications:

AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
APTA	American Public Transportation Association
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
ATA	American Trucking Associations
CTAA	Community Transportation Association of America
CTBSSP	Commercial Truck and Bus Safety Synthesis Program
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FMCSA	Federal Motor Carrier Safety Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
IEEE	Institute of Electrical and Electronics Engineers
ITE	Institute of Transportation Engineers
NCHRP	National Cooperative Highway Research Program
NCTRP	National Cooperative Transit Research and Development Program
NHTSA	National Highway Traffic Safety Administration
NTSB	National Transportation Safety Board
SAE	Society of Automotive Engineers
TCRP	Transit Cooperative Research Program
TRB	Transportation Research Board
U.S.DOT	United States Department of Transportation