SERVICEABILITY LIMITS AND ECONOMICAL STEEL BRIDGE DESIGN

INTERIM REPORT

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in cooperation with
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ABSTRACT

The objective of this MPC research effort is to conduct serviceability comparisons between state live-load deflection criteria and the American Association of State Highway and Transportation Officials (AASHTO) Load Resistance and Factor Design (LRFD) and AASHTO Load Factor Design (LFD) standards to provide information on the conservative nature of state serviceability criteria and loss of economical benefits for steel bridge design. Of special concern is loss of economy when using high performance steel. The relationship between the LRFD and LFD methods and the impact of moving towards LRFD was also examined.

A group of six states was selected for the serviceability comparisons. The states represent various levels of conservativeness across the country. The same states are used for the LFD and LRFD provision comparisons. A set of ten steel bridges that are in service (one with significant field test data) are used for the study. The bridges range in number of spans, length, width, girder spacing and geographical location.

The selected state deflection limit practices are applied to the set of bridges and compared to the AASHTO specifications for both the LFD and LRFD methods. The variations in state practice loading, analysis and deflection limits are studied and discussed. Finally, for the set of bridges, the design impact of the state practices is examined.

Results show that the current AASHTO LFD and LRFD deflection criteria typically do not control in design and, therefore, do not have a negative impact on economy of conventional or high performance steel bridges.

The relationship of AASHTO LRFD criteria to AASHTO LFD criteria is applied to the set of bridges to determine the possible impact on deflections as states move towards using the LRFD provisions. Variables studied are the differences in loading and analysis, where analysis differences are attributed to stiffness distribution, multi-presence of vehicles, and dynamic effects. The results show that LRFD deflections are slightly larger overall for single span bridges, but are reduced for multi-span bridges. However, the differences are relatively small in both cases. Thus, states using LRFD should not notice a significant change in the deflection criteria results from the LFD procedures.

The deflection criteria of the six states for both LFD and LRFD are applied to the ten study bridges. Many of these states have adopted more conservative criteria than AASHTO. The results show that when states use more conservative criteria, bridge design may be controlled by deflections. This means these states must put more steel (more cost) into the structure to meet their standards. Some of the results are quite astonishing when a state must greatly increase the bridge stiffness (significant cost increase) to meet state criteria. The problem is, besides the additional cost, these 10 bridges are in service and performing well. There are no apparent deficiencies in either user comfort or deformation-induced damage. The conclusion that can be drawn is that these conservative states are expending unnecessary materials and costs.

This MPC project is part of an overall research effort to produce rational deflection criteria (or a form of serviceability criteria) to limit user discomfort and deformation-induced structural damage in steel girder bridges. Current AASHTO criteria do not effectively meet that purpose. Additionally, many states have chosen to use more conservative deflection criteria than AASHTO. This results in more costly bridges and inconsistent design procedures. It also impedes the use of high performing materials such as HPS when deflections and not strength (safety) controls the design. Implementation of realistic and appropriate deflection limits over the nation's bridge inventory will result in more efficient and less costly bridges. Conventional steel and high performance steel bridge design will be more consistent and cost effective

across the country. The authors are continuing the serviceability work with two additional and related research projects from HDR/FHWA and AISI/IDDOT.

This interim report for the MPC portion of the overall research effort includes the results of the MPC research contract work plan. The MPC program will be used for final dissemination of the overall research effort with a final report to be submitted in summer 2009. The final report will include the work of the MPC project in addition to the work and results from HDR/FHWA and AISI/IDDOT research projects.

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1. INTRODUCTION

1.1 Background

Currently, there are two AASHTO specifications commonly in use for bridge design. These standards are the Load Factor Design (LFD) method of the *2002 AASHTO Standard Specifications for Highway Bridges* and the Load and Resistance Factor Design (LRFD) method of the *2003 AASHTO LRFD Bridge Design Specifications*. The LRFD method is a recent specification intended to replace the LFD method as states adopt the newer specification procedures. States are currently moving toward the LRFD method. AASHTO specifications recommend in the LFD specification and present for optional adoption in the LRFD specification limiting live-load deflections to a maximum allowable live-load deflection. The specifications recommend limiting the allowable live-load deflections in bridges to the span length divided by 800 (L/800) for most bridges and a more restrictive L/1000 for bridges in urban areas with pedestrian traffic. Most states have applied the optional deflection limits in the LRFD specification. The limits were originally established to control vibrations in an attempt to limit human discomfort to bridge vibration. State engineers have also come to believe that the deflection limits may control deflection induced structural damage such as deck cracking.

The engineering community is currently attempting to address several issues with bridge deflection limits:

1. Current deflection limits are intended to limit user discomfort and limit deformation-induced structural damage. However, past practice and research has shown that limiting deflections may not be adequate for either user comfort or damage.

2. The recent LRFD provisions include optional deflection limit criteria similar to the LFD provisions. As states move toward adoption of LRFD, most states have decided to apply the optional limits or even more conservative limits. However, the loading and analysis procedures have changed from LFD to LRFD and the impact of the newer LRFD criteria are unknown on the role of deflection limits for design economy.

3. The steel industry has developed a high performance steel (HPS) for steel bridges that has improved the quality of the steel material and led to cost savings through weight savings. However, if deflections control in the design, which may happen with the higher strength HPS, these benefits are not realized.

4. States' application of deflection criteria varies significantly across the country, and many states have adopted more restrictive deflection criteria than AASHTO, which inherently impacts economy of steel bridges. This is especially true when using HPS, but with the more restrictive deflection limits, conventional steel bridges would also be more costly if deflections control the design.

Michael Barker at the University of Wyoming and Karl Barth at West Virginia University have been addressing these four issues through a series of research projects (Barker and Barth 2007a and 2007b, Christopher 2001, Barth et al. 2004, Anderson 2005). The objective of the overall research effort is to develop rational serviceability/deflection criteria for steel girder bridges that states adopt that will meet the intended purpose of user comfort and limited deformation-induced structural damage. The effort started as problems arose when high performance steel was introduced to the bridge market (No. 3 in the list above). HPS, in this case HPS70W (yield strength of 70 ksi), is a superior steel with higher yield strength, improved weldability, greater levels of toughness and improved weathering resistance that can lead to more economical bridges than conventional 50W (yield strength 50 ksi) designs. HPS can produce significant weight and cost savings if strength controls the design, especially when used in hybrid designs with 50W steel (Barker and Schrage 2000). However, using HPS results in lighter sections and less steel material, and therefore, live-load deflections increase over conventional 50W bridges. If deflections control the design, which may occur with the lighter sections, there is no benefit in using HPS and there is no weight or cost savings. Since the lighter section may result in deflection controlled designs, the initial thoughts were to investigate the deflection limits themselves (No. 1 in the list above) knowing from past research that current criteria are not the most effective way of limiting the vibration of a bridge that causes discomfort, nor do they provide positive effects on durability or maintenance of steel bridges.

In addition, states are currently in the process of switching from the AASHTO LFD design provisions to the AASHTO LRFD design provisions. Limiting deflections is optional in LRFD. The specification developers understood that limiting deflections did not correlate well to limiting user discomfort or structural damage. However, with the states' reliance on the belief that limiting deflections does limit user discomfort and deformation-induced damage, most states apply deflection limits. Although the optional allowable deflections in LRFD are the same as those in the LFD provisions, the loads and analysis differs, and the design impact of switching to LRFD is unknown (No. 2 in the list above).

When conducting initial studies on the impact of deflection limits for HPS bridges, Barker and Barth found that there not only may be a problem with HPS bridges meeting current AASHTO LFD and LRFD deflection criteria, but also many states have adopted much more conservative or restrictive deflection criteria that exacerbates the problem. Thus, there not only is an issue with HPS bridges considering current AASHTO requirements, there is also an issue with all steel bridges when states apply more conservative deflection criteria than AASHTO dictates (No. 4 in the list above).

1.2 Research Effort

Addressing the four issues has been broken up into three main research thrusts (and three research contracts). Issues 2 and 4 are primarily addressed in this Mountain Plains Consortium (MPC) project. The MPC contract was leveraged to expand the scope (to meet the overall research objectives and address all four issues) with other research entities. Issues 1 and 3 are currently being addressed in two contracts: one with an HDR/FHWA research project and the other with an American Iron & Steel Institute (AISI) and Idaho DOT project. However, certainly all the research projects are inter-related in the overall effort to produce rational serviceability/deflection criteria for steel girder bridges. In addition, there needs to be an overall comprehensive report that disseminates the results from the total research effort. The reporting requirements for the HDR/FHWA and AISI/IDDOT contracts are not conducive to the dissemination requirements. Barker and Barth believe the MPC report should be this comprehensive report representing the total research effort. Thus, this is only an interim report containing the findings from the work plan of the MPC contract with additional sections on current work and planned work on the HDR/FHWA and AISI/IDDOT projects. A final report will be completed and submitted to MPC in the summer of 2009.

1.2.1 MPC Research Project

The objective of the MPC research effort is to conduct serviceability comparisons between state live-load deflection criteria and the AASHTO LRFD and AASHTO LFD standards to provide information on the conservative nature of state serviceability criteria and loss of economical benefits for steel bridge design. The relationship between the LRFD and LFD methods and the impact of moving toward LRFD is also examined.

A group of six states was selected for the serviceability comparisons. The states represent various levels of conservativeness across the country. The same states are used for the LFD and LRFD provision comparisons. The states were selected from an LRFD deflection criteria survey as part of the HDR/FHWA project described below. The LFD criteria for the states selected are from previous work by Barth et al. (2002). A set of 10 high performance steel bridges that are in service (one with significant field test data) are used for the study. The bridges range in number of spans, length, width, girder spacing and geographical location.

The selected state deflection limit practices are applied to the set of bridges and compared to the AASHTO specifications for both the LFD and LRFD methods. The variations in state practice loading, analysis and deflection limits are studied and discussed. Finally, for the set of bridges, the design impact of the state practices is examined.

The relationship of LRFD criteria to LFD criteria is applied to the set of bridges. Variables studied are the differences in loading and analysis, where analysis differences are attributed to stiffness distribution, multi-presence of vehicles, and dynamic effects.

Dissemination of the research is discussed in Section 1.2.2 below. Initial results from the MPC project have already been presented (Barker and Barth 2007b) in the Keynote Session of the World Steel Bridge Symposium in New Orleans, LA, in 2007, accompanied with a proceedings paper. Barker and Barth actively converse with state DOTs on the impact deflection criteria, especially conservative state practices, have on economy of steel bridges.

1.2.2 HDR/FHWA Research Project

The HDR/FHWA project has four tasks. These will be presented here and discussed in terms of how they fit with and advance the MPC work to meet the overall research objectives.

Task 1 – Survey of Current State Practice for LRFD Deflection Design

A survey of state transportation departments was conducted by Barth to determine state deflection criteria for the LFD provisions as a part of NCHRP 20-07/133 (Barth et al. 2002). This survey was conducted at time when there was considerable change in state transportation department bridge design practice as more DOTs moved toward the implementation of LRFD. Also, the use of HPS 70W in bridge designs was beginning to be more widespread, and relatively little understanding of implications of the application of historic deflection limits is available. This survey was used for the state practice deflection criteria for the selected bridges and the LFD method.

As part of the HDR/FHWA project, a new survey was conducted to determine state practices for the LRFD method. Prior to this survey, it was not known how states applied deflection criteria. This survey, completed in 2007, is the basis for selecting the states used in the MPC project.

Task 2 – Assess Implications of State Practice on Bridge Design

The HDR/FHWA project is currently using the results of the MPC project and expanding the study to examine several issues. First, the MPC project studied bridges as-built with their actual design capacities at the LRFD Strength I limit state (LFD Strength) and the LRFD Service II limit state (LFD Overload). This as-built condition assessment meets the objectives of the MPC project, but additional study can result in better understanding of the deflection criteria and states' practices. For instance, if the bridges are artificially modified so the deflections are based on the optimum strength limit (modified by the rating factor), the deflections represent the maximum possible for an optimized bridge. Likewise, and conversely, if the bridges are optimized for deflection (deflection at the allowable limit), the load capacities can be determined. This work, coupled with state practices, will yield additional information on the impact state criteria have on bridge economy and design.

In addition, current thought is that deflection criteria limit deformation-induced structural damage. Bridge engineers are particularly interested in deck cracking over the piers. The HDR/FHWA project is studying the mechanistic strains expected in the deck over the piers at various expected load levels. To prevent premature deck cracking, these concrete deck strains should remain low. The question is does limiting live-load deflections adequately control these deck strains? Initial results show there is not a good correlation between deflection limits and deck strains at either the as-built or strength or deflection optimized bridges. The HDR/FHWA project will study and make recommendations on this behavior.

Task 3 – Develop Guidelines for Recommended Practice

The results of the MPC and the HDR/FHWA projects will be used to develop more rational specifications that assure a more unified application of serviceability limits in current practice. The results will be compared to past research and current codes that limit bridge deflections. The outcome and benefit of this work will be improved serviceability specifications, improved consistency of design across the states and more economical use of high performance materials in bridges such as HPS.

Task 4 - Dissemination of Information through Technology Transfer

The final report will be prepared and submitted to MPC during the last stage of the research program. This report will summarize the results of the research and design recommendations. It will be widely distributed to bridge engineers and researchers.

The results and recommendations of the research will be presented to the AASHTO T-14 committee for consideration for possible adoption into the AASHTO LRFD Specifications. This will be done after completion of the research program. In addition, the results and recommendations will be presented to the TRB A2C02 Steel Bridge Committee, and they will be presented at conferences such as the ASCE/SEI Structures Congresses and the TRB Annual Meeting. Technical papers will be written for professional journals such as the ASCE/SEI "Journal of Structural Engineering" and the "Journal of Bridge Engineering." These papers will also be co-authored by the co-PIs and all graduate students working on this project.

1.2.3 Idaho DOT/AISI Research Project

The overall work effort is being coordinated with the AISI Bridge Task Force and the AASHTO T14 committee. The Idaho DOT is part of AASHTO T14 and has a vested interest in deflection criteria and performance of a particular bridge in Coeur d'Alene, ID. The bridge is a replacement bridge over an interstate highway. Due to approach elevation and clearance requirements, the bridge design is not typical in its superstructure. To meet the clearance requirements, the superstructure is shallow with a high span-to-depth ratio and haunches near the center pier. Thus, the design is "out-of-the-box" in terms of typical bridges and deflections, and in-service performance becomes more important to predict and understand.

There is not a contract in hand as of yet, but the plan is to test this bridge in the Fall 2008. Barth and Barker have significant experience field testing bridges for serviceability and design performance. The recommended procedures and the results of the MPC and HDR/FHWA projects will be applied and tested on the Idaho bridge. The outcome should be confirmation of the recommendations and a demonstration of rational procedures to other states, especially those that implement conservative deflection criteria.

1.3 Summary and Interim Report Format

The objective of the overall research work is to produce rational deflection criteria (or a form of serviceability criteria) to limit user discomfort and deformation-induced structural damage in steel girder bridges. Current AASHTO criteria do not effectively meet that purpose. Additionally, many states have chosen to use more conservative deflection criteria than AASHTO. This results in more costly bridges and inconsistent design procedures. It also impedes the use of high performing materials such as HPS when deflections and not strength (safety) controls the design. Implementation of realistic and appropriate deflection limits over the nation's bridge inventory will result in more efficient and less costly bridges. Conventional steel and high performance steel bridge design will be more consistent and cost effective across the country.

This interim report for the MPC portion of the overall research effort includes the results of the MPC research contract work plan. The MPC program will be used for final dissemination of the overall research effort with a final report to be submitted in summer 2009. The final report will include the work of the MPC project in addition to the work and results of the HDR/FHWA and AISI/IdDOT research.

This interim report is organized as follows: Section 2 is the background particular to the MPC project, AASHTO LFD and LRFD deflection criteria, selected state practices for LFD and LRFD and a discussion on the state surveys, and the set of bridges selected for the study. Sections 3 and 4 are the LFD and LRFD AASHTO and state practice comparisons, respectively. Section 5 is a direct comparison between the AASHTO LFD and LRFD deflection criteria and the expected impact of the LRFD procedures as states continue the move towards LRFD. Section 6 is the summary and conclusions of the MPC project results and the expectations of the future work and final report from the HDR/FHWA and AISI/IdDOT projects. Although much information is presented in this interim report, the appendices supporting the report are not included. However, they will be included in the final report.

2. LIVE-LOAD DEFLECTIONS, RESEARCH DATA SETS AND DESIGN CRITERIA

2.1 Introduction

AASHTO Load Factor Design (LFD) specifications require that live-load deflections be controlled by limiting span-to-depth ratio and by limiting the maximum allowable live-load deflection. These limits are intended to control excessive bridge deflections and prevent possible bridge deterioration (Fountain and Thunman 1987). Deflection limits are a recommendation in the specification for LFD (AASHTO 2002). The same restrictions are optional deflection limits imposed by the Load Resistance and Factor Design (LRFD) Specification (AASHTO 2003); however, applied loads and analysis procedures are different than for LFD. Many states use more conservative live-load deflection limits and deflection calculation methods than are prescribed in AASHTO. These more conservative state procedures can reduce, or completely mitigate, the benefits of using high performance materials when deflections control the design. Some states' procedures are conservative enough that even conventional steel bridges are limited by deflection criteria, which results in a more costly bridge. Herein lays the objective of this research. If more appropriate deflection (serviceability) criteria can be developed that limits user discomfort and prevents deformation-induced damage, and states can accept these proposed provisions, high performance steel and even conventional steel bridge design will be more consistent and cost effective across the country.

This chapter describes the AASHTO deflection limit criteria, the background for the requirements and past research on deflections in bridges. It continues with the state survey summaries for the LFD and LRFD procedures applied across the country. The full surveys are not shown here in the interim report, but will be shown in the final report. Six state criteria are selected and described for the analysis of the set of bridges selected for this research. Ten bridges are presented that represent a range of variables and locations. These bridges are described in terms of their geometries and characteristics. Finally, analysis and modeling methods are presented along with LFD and LRFD design requirements for safety (strength), service (expected overloads), and deflection. One of the bridges will be used to demonstrate the design requirements.

Deflection limits were originally established to control vibrations in an attempt to limit user discomfort. Other methods for limiting bridge vibrations have shown that deflection limits may not be the most effective way of limiting objectionable bridge vibrations. (Barth, Bergman and Roeder 2004) The AASHTO limits are essentially user-defined empirical limits with no mechanical basis. This does not explain the more restrictive limits placed on deflection by state transportation departments.

Current AASHTO limits have not been found to contribute to any undesirable structural effects (Barth, Bergman and Roeder 20042). Limits were originally designed to prevent excessive bridge vibrations, but studies have shown that vibrations are best characterized by vertical accelerations and not live-load deflection. Bridges that do meet AASHTO limits and the more restrictive state limits have produced objectionable bridge vibrations (Christopher 2001). This suggests that there may be better methods of controlling bridge vibrations than limiting deflection.

2.2 AASHTO Deflection Criteria

Origin of AASHTO live-load deflection criteria can be traced to the 1905 American Railway Engineering Association (AREA) specifications. AREA specifications limit the span-to-depth ratio, which indirectly limit maximum live-load deflections.

Direct live-load deflection limits were established in the 1930's when the Bureau of Public Roads conducted a study that attempted to limit objectionable bridge vibrations. (Barth, Bergman and Roeder 2002) The bridges included in the Bureau of Public Roads study were constructed with wood plank decks and superstructures consisting of pony trusses, simple beams, or pin connected through-trusses. These study bridges were non-composite, and rarely contained continuous spans. ASTM A7 steel with 33,000 psi yield strength was the accepted steel for bridge design and construction. AASHTO limitations first appeared in the 1941 edition in part due to the results of the Bureau of Public Roads vibration study (Fountain and Thunman 1987).

As far back as 1950, the American Society of Civil Engineers began investigating the source of the live-load deflection requirements. In 1958 the ASCE committee reported that no clear basis for the limits could be found. (Barth, Bergman and Roeder 2002) The AASHTO steel flexibility limits do not exist in European codes, or in the AASHTO specifications for reinforced or prestressed concrete.

AASHTO Standard Specifications for Highway Bridges (2002) states in Article 10.6.2 that members having simple or continuous spans preferably should be designed so that deflection due to service live loads plus impact shall not exceed 1/800 of the span. Deflections in bridges in urban areas used in part by pedestrians preferably shall not exceed 1/1000 of the span. The recommended allowable deflections on pedestrian bridges are smaller due to the increased sensitivity to bridge accelerations felt by pedestrians as opposed to those of a person in a vehicle. For checking live-load deflection, the service live load preferably shall not exceed HS 20 loading.

The deflection criteria stated in the AASHTO LRFD (2003) code are optional. Design Engineers are responsible for determining if the deflection criteria should be met. Applied live loads for LRFD deflection criteria differ from those used in LFD design.

AASHTO (LFD and LRFD) analysis for live-load deflection distributes the live load to the bridge girders equally. The deflection distribution factor is determined by the number of whole 12 ft. traffic lanes, an intensity reduction factor and the total number of bridge girders. The intensity reduction factor decreases deflection based on the fact that the more lanes there are, the less likely every lane has a side-by-side full design truck. Intensity reduction factors are therefore based on the number of traffic lanes.

2.3 Live-Load Deflection Studies

In a 1987 study, Fountain and Thunman (1987) examined live-load deflection criteria and proposed changes for the existing criteria for steel bridges with concrete decks. It stated that AASHTO live-load deflection limits provide no positive effects on strength, durability, safety or maintenance of steel bridges. This study also determined that transverse cracking of concrete bridge decks is the most common form of bridge deterioration. These cracks can be caused by any of the following: plastic shrinkage, drying shrinkage of concrete combined with deck restraint, long-term flexure of continuous spans under service loads and traffic-induced repetitive vibrations.

In modern bridge construction, a majority of the steel bridges built use composite design. Fountain and Thunman questioned the beneficial influence of AASHTO deflection criteria, because flexural stresses in the deck of composite bridges are small. They also suggested that increased stiffness could cause an increase in deck deterioration attributed to the effects of volume change on the tensile stresses due to a deck/beam interaction increase.

Another study (Goodpasture and Goodwin 1971) focused on the relationship between deck deterioration and live-load deflection. One phase of the study examined the effect of stiffness on transverse cracking in continuous steel bridges. No correlation between girder flexibility and transverse cracking could be established. Nevels and Hixon (1973) completed a similar study of field measurements on I-girders bridges to try to determine causes of bridge deck deterioration. This study found no relationship between flexibility and deck deterioration.

A Wright and Walker study (1971) looked at the rationality of live-load deflection limits as well as the effects of flexibility on bridge serviceability. The study reviewed human response to vibration and the effects of deflection and vibration on the concrete deck deterioration. Their conclusions advise that live-load bridge deflections do not have a significant influence on structural performance of steel bridges. Deflection limits alone are not an efficient method of controlling excessive bridge vibrations or assuring human comfort.

2.4 Effect of Bridge Deflection on Superstructure Bridge Vibration

The existing AASHTO live-load deflection limits were initiated to provide vibration control. According to a 1930 Bureau of Public Roads study, bridges that exhibited intolerable accelerations had calculated deflections of greater than L/800 (Barth, Bergman and Roeder 2002). The measure of intolerable vibrations is determined by human perception. Two parameters influence human perception to vibration and they are acceleration and frequency.

There are two classifications of human reactions to vibrations: physiological and psychological. Psychological discomfort results from unexpected motions, and physiological discomfort results from a low frequency and high amplitude of vibration. The main source of these discomforts can be attributed to vertical bridge accelerations.

Many factors other than live-load deflection influence dynamic behavior of steel brides. Vehicle properties, bridge geometric and material properties, and vehicle/structural interaction all contribute to bridge accelerations. A 1975 analytical study (Amaraks 1975) investigated finite element models of non-composite simple and multi-span bridges. This study tried to determine the effects that span length, stiffness, surface roughness, axle spacing and number of axles had on bridge vibrations. This was accomplished by varying the parameters for the finite element model to determine which variables had the most effect. Surface roughness caused the most significant effect on accelerations of the bridges. Modeling a rough roadway surface was found to increase accelerations as much as five times those with a smooth surface. The study also determined that the shorter the bridge length, the greater the maximum accelerations. As the stiffness of the bridges were reduced, accelerations increased, but this increase was significantly less than the surface roughness increase. It was also observed that vehicle speed has a large impact on the maximum bridge accelerations.

A later study (Dewolf and Kou 1997) looked at deck considerations such as thickness and surface roughness. This study also included the effect of vehicle speed, vehicle weight, and girder flexibility. A sample composite, continuous, four-span steel girder bridge was used for this study. Results of this study show that bridge accelerations increased as much as 75% when the road surface was changed from smooth to rough. Girder stiffness was found to have minor influences on the overall bridge amplification. Increase in test vehicle speed also had a large influence on the dynamic behavior of the sample bridge.

These studies show that the presence of excess bridge vibrations is due more to the period of the bridge, surface roughness, and vehicle speed than stiffness of the girders. In contrast to this, live-load deflections were developed to limit the amount of vibration experienced by the bridge users. Results show that deflection limits do not have a great dynamic effect and, therefore, are not the most effective way of limiting bridge vibrations.

The Ontario Highway Bridge Design Code (1983) does not limit deflection as a function of span length as AASHTO does. They use deflections requirements to limit bridge vibrations based on the natural frequency of the bridge. Natural frequency depends on the bridge mass and stiffness. The code allows less deflection for bridges that have pedestrian use similar to AASHTO. While determining natural frequency of bridges is more complicated than the AASHTO method, it provides a much more efficient way of limiting bridge vibrations (Barth, Christopher, Roeder and Wu 2003).

2.5 Effect of Deflection Criteria on HPS Bridges

High performance steel (HPS), yield = 70 ksi, provides increased yield strength, enhanced weldability and improved toughness compared to conventional grade 50 ksi steel. Due to its many advantages, HPS can be utilized to produce lighter, more economical structures. Implementation of HPS for bridge applications has caused a concern with existing AASHTO live-load deflection criteria. For conventional grade 50 ksi steel bridges, this limit rarely governs the girder geometry using standard AASHTO criteria. Use of HPS can reduce the stiffness of the cross sections, which produces higher deflection values. This higher deflection can cause the deflection limits to control the bridge design. Some states use more conservative live-load deflection criteria than AASHTO, which creates an even greater chance that deflection limits will control the bridge design.

HPS is more expensive than conventional steel and, therefore, it is most cost effective to use HPS in only high stress areas of bridge girders (Barker and Schrage 2000). For this reason, HPS is most commonly utilized in the girder flanges. The web and low stress areas can use conventional 50 ksi steel. This creates a hybrid girder design which utilizes HPS in the most cost effective manner.

2.6 State Practices for Deflection Limits

Two surveys of professional practice were completed to determine how each state applies live-load deflection criteria for steel bridges (Barth, Bergman and Roeder 2002 for LFD and HDR/FHWA project for LRFD). The surveys were completed by questionnaire and telephone to bridge engineers from all 50 states. The purpose was to determine information on the application of live-load deflection criteria within each state. Questions were asked to determine what actual live-load deflection limits are applied to steel bridges. It was also important to determine the live-loads that are used to compute the deflections. Load factors and lane load distribution factors were also recorded for each state. From this study, six state criteria were selected for further analysis in this work; South Dakota, New York, Arizona, New Jersey, Rhode Island and Tennessee were chosen to be compared to the AASHTO criteria. The deflection criteria of these states provide a variety of loadings, limits and distribution factors. In general, except for Tennessee, these states have significantly more conservative live-load deflection criteria than the AASHTO standard.

2.6.1 LFD State Practices

The AASHTO LFD Standard Specifications for Highway Bridges (2002) limits the maximum live-load deflection for non-pedestrian steel bridges to the span length divided by 800, L/800. The LFD survey established a wide variation in the deflection limits utilized by each state. The most restrictive deflection limit has an allowable deflection of one-half the AASHTO limit (L/1600). AASHTO standards also indicate that deflection due to live-load plus impact is to be controlled by the deflection limit. AASHTO uses an HS20 truck loading plus impact to determine bridge deflections (Figure 2.1).

However states have chosen to use larger live-loads. Larger design trucks, lane loads and truck plus lane loads all with or without impact are used by states producing larger live-load deflections. For example, many states use an HS25 truck and/or lane loading (shown in Figure 2.1). An HS25 truck is similar to an HS20 truck except axel/wheel loadings are multiplied by 1.25 (25/20) to adjust for the heavier design truck. The difference in applied live-loads provides a wide variation in calculated deflection.

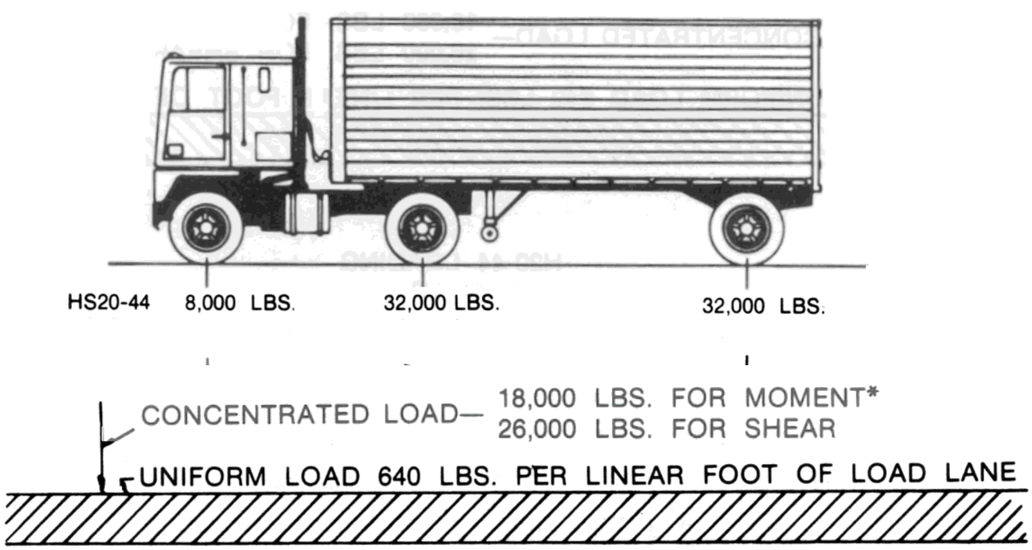


Figure 2.1 HS20 Truck & Lane Loading (AASHTO 17th Ed.)

Load factors are used in strength and overload design to ensure safety and performance of a structure. Load factors are not normally used to calculate deflection, but some states apply load factors to provide a more conservative calculated deflection. Moment lane load distribution factors can also greatly increase the loads used to calculate deflections. Many states use moment lane load distribution factors for calculating deflections, while others use the AASHTO equal distribution over all girders. The effect of lane distribution factors can largely increase live-load deflections depending on the spacing of bridge girders.

Information on 47 state criteria was collected from the survey. The state survey showed there was a wide range of variation in deflection limits and loading employed by each state. For steel bridges without pedestrian access, deflection limits imposed are as follows:

1 state uses a L/1600 limit
1 state uses a L/1100 limit
5 states use a L/1000 limit
40 states use a L/800 limit

For bridges with pedestrian access the following apply:

1 state uses a L/1600 limit
2 states use a L/1200 limit
1 state uses a L/1100 limit
39 states use a L/1000 limit
3 states use a L/800 limit

AASHTO LFD requirements state that deflections calculated due to an HS20 truck live load plus impact be subject to the deflection limits. The survey showed a large variation in the size and type of load use to calculate deflection:

1 state uses HS20 truck only
16 states use HS20 truck plus impact
1 state uses HS20 lane load plus impact
1 state uses HS20 truck load plus lane load without impact
7 states use the larger deflection caused by either the HS20 truck load plus impact or HS20 lane load plus impact.
17 states use HS20 truck load plus lane load plus impact
4 states use military or permit vehicles
8 states use HS25 trucks

Typically, deflections are calculated by using service loads. Load factors and distribution factors that increase the applied live-load are not normally used. However, load factors are used by several states to increase the applied live loads when calculating deflections. Lane distribution factors can significantly affect the magnitude of loads used to calculated deflection. The survey did not include a question about whether the state employed the multi-presence reduction factor. Therefore, the analyses contained herein assume that all states apply the reduction when using equal distribution of deflections. The variation of distribution factors used for calculating deflections are as follows:

26 states use moment lane distribution factors
3 states use AASHTO LRFD lane load distribution factors
13 states use equal distribution to all girders
1 state uses state specific lane distribution factor

State deflection criteria for Arizona, New Jersey, New York, Rhode Island, South Dakota and Tennessee were chosen for further analysis in this study. These states were selected based on their conservative (except in the case of Tennessee) deflection limits, live-loads, and distribution factors. Of these states, South Dakota employs the most conservative deflection limit of L/1200 for pedestrian steel bridges (Rhode Island is most conservative for non-pedestrian bridges at L/1100). This conservative limit is 83% of the deflection permitted by AASHTO for live-load deflection of pedestrian bridges, and Rhode Island is 73% of AASHTO for non-pedestrian bridges. Rhode Island is the most conservative in terms of the magnitude of the live-load. Rhode Island uses a factored live-load of a HS20 Truck + HS20 Lane + Impact. For some spans, this provides an analysis live-load of more than twice that of the AASHTO HS20 Truck + Impact. On the other hand, Tennessee is equal in all aspects to the AASHTO criteria (LRFD Tennessee criteria differ from AASHTO, and that is why it is included here). A summary of the defection limits, live-loads, and distribution factors for each state are shown in Table 2.1. The AASHTO criteria and limits are provided for comparison.

These states represent a sample of the more conservative deflection criteria. All other states criteria fall in between these more conservative standards and the AASHTO criteria. Due to the differences in limits, live loads and distribution factors, it is difficult to determine which state would be the most conservative for every condition. For this reason, these six states were chosen to analyze the 10 study bridges to determine possible relationships formed by their live-load deflection criteria.

Table 2.1 Selected State Criteria for AASHTO LFD Comparisons

				Limits	
State	**Factored**	**Loading**	**Distribution**	**w/Ped.**	**w/o Ped.**
AASHTO	No	Truck + I	Equal Distribution	L/1000	L/800
Arizona	No	Truck + I or Lane	Moment Distribution	L/1000	L/800
New Jersey	No	LRFD HL93 Loading + I	Moment Distribution	L/1000	L/1000
New York	No	Truck + I or Lane + I	Equal Distribution	L/1000	L/800
Rhode Island	Yes = 5/3	Truck + Lane + I	Equal Distribution	L/1100	L/1100
South Dakota	No	Truck + I or Lane + I	Moment Distribution	L/1200	L/1000
Tennesse	No	Truck + I	Equal Distribution	L/1000	L/800

2.6.2 LRFD State Practices

AASHTO LRFD Specifications (2003) contain optional deflection criteria that limit the maximum live-load deflection for non-pedestrian steel bridges to the span length divided by 800, L/800. The LRFD survey established a wide variation in the deflection limits utilized by each state. The most restrictive deflection limit has an allowable deflection L/1200. AASHTO standards also indicate that deflection due to live-load plus impact is to be controlled by the deflection limit. AASHTO uses an HL93 truck loading (truck by itself or lane + 25% of the truck) plus impact on the truck to determine bridge deflections. The truck and 640 lb/ft lane loading is the same as shown for AASHTO LFD in Figure 2.1. However, states have chosen to use larger live loads. Larger design trucks, lane loads and full truck plus lane loads all with or without impact are used by states producing larger live-load deflections. The difference in applied live-loads provides a wide variation in calculated deflection.

Load factors are used in Strength I and Service II design to ensure safety and performance of a structure. Load factors are not normally used to calculate deflection, but some states apply load factors to provide a more conservative calculated deflection. Moment lane load distribution factors can also greatly increase the loads used to calculate deflections. Many states use moment lane load distribution factors for calculating deflections, while others use the AASHTO equal distribution over all girders. The effect of lane distribution factors can largely increase live-load deflections depending on the spacing of bridge girders.

Of the 50 states surveyed as part of the HDR/FHWA project, 44 valid responses were obtained. Of the six states that did not participate in the survey, one state designs primarily with concrete and did not wish to participate in a steel bridge survey, one state is currently developing its own design criteria and did not wish to comment at this time, and four states didn't respond.

The AASHTO LRFD specifications were implemented in 1998 with the optional design deflection limit of L/800 for steel bridges without pedestrian access and L/1000 with pedestrian access. Of the 44 states reporting deflection limits for bridges without pedestrian access the following apply:

1 state uses a L/1100 limit
3 states use a L/1000 limit
35 states use a L/800 limit
5 states use other criteria

Of the states reporting deflection limits for bridges with pedestrian access the following apply:

1 state uses a L/1200 limit
1 state uses a L/1100 limit
30 states use a L/1000 limit
8 states use a L/800 limit
4 states use other criteria

The survey results indicate that designers apply a wide variety of different combinations of loads and factors when determining live-load deflection. Of the 44 valid responses the following apply:

1 state uses Truck only (no factor)
6 states use Truck+Impact (no factor)
1 state uses Truck+Impact (factored)
14 states use larger of Truck+Impact or Lane + 25% Truck+Impact (no factor)
3 states use larger of Truck+Impact or Lane + 25% Truck+Impact (factored)
4 states use Lane+Truck+Impact (factored)
12 states use Lane+Truck+Impact (no factor)
2 states use larger of Truck+Impact or Lane+25% Truck (no factor)
1 state has no standard practice

The lane application of the loads is another source of variability between states. The LRFD specification equally distributes the loads to all lanes with a multiple lane reduction factor. Of the 44 responses the following apply:

31 states use equal distribution to all girders and use the multi-lane reduction factors
3 states use equal distribution to all girders but not the multi-lane reduction factors
5 states use moment lane distribution factors
3 states use LFD moment lane distribution factors (i.e., S/5.5)
2 states have no standard practice

State deflection criteria for Arizona, New Jersey, New York, Rhode Island, South Dakota and Tennessee were chosen for further analysis in this study. These states were selected based on their conservative (or unconservative in the case of Tennessee) deflection limits, live-loads, and distribution factors. Of these states, South Dakota employs the most conservative deflection limit of L/1200 for pedestrian steel bridges (Rhode Island is most conservative for non-pedestrian bridges at L/1100). This conservative limit is 83%

of the deflection permitted by AASHTO for live-load deflection of pedestrian bridges and Rhode Island is 73% of AASHTO for non-pedestrian bridges. Rhode Island is the most conservative in terms of the magnitude of the live-load. Rhode Island uses a factored live-load of a Lane + Full Truck + Impact. For some spans, this provides an analysis live-load of more than twice that of the AASHTO loading. On the other hand, Tennessee is less conservative than the AASHTO criteria with a Truck only loading without impact while the remaining criteria is the same as AASHTO. A summary of the defection limits, live-loads, and distribution factors for each state are shown in Table 2.2. The AASHTO criteria and limits are provided for comparison.

These states represent a sample of the more conservative (and unconservative) deflection criteria. All other states criteria fall in between these more conservative standards and the Tennessee criteria. Due to the differences in limits, live-loads, and distribution factors, it is difficult to determine which state would be the most conservative for every condition. For this reason, these six states were chosen to analyze the ten study bridges to determine possible relationships formed by their live-load deflection criteria.

Table 2.2 Selected State Criteria for AASHTO LRFD Comparisons

				Limits	
State	Factored	Loading	Distribution	w/Ped.	w/o Ped.
AASHTO	No	Truck + I or Lane + 25%Truck + I	Equal w/ multiple lane reduction	L/800	L/1000
Arizona	Yes=1.75	Truck + I or Lane + 25%Truck + I	Equal w/ multiple lane reduction	L/800	L/800
New Jersey	Yes=1.75	Truck + I or Lane + 25%Truck + I	Multiple Lane Moment Distribution	L/1000	L/1000
New York	No	Truck + Impact	Multiple Lane Moment Distribution	L/800	L/1000
Rhode Island	Yes=1.75	Lane + Truck + Impact	Equal w/ multiple lane reduction	L/1100	L/1100
South Dakota	No	Truck + I or Lane + 25%Truck + I	Equal w/out lane reduction	L/1200	L/1000
Tennesse	No	Truck Only	Equal w/ multiple lane reduction	L/1000	L/800

2.7 Selected Study Bridges for Analysis

Current AASHTO standards for live-load deflection, as well as the more conservative state deflections discussed, can have a negative impact on the economic use of HPS and conventional steel bridges. For AASHTO designs that use conventional grade 50 steel, deflection limits rarely govern the section geometry. In the case of HPS, reduced cross-sectional properties of bridge girders provide a much greater possibility of live-load deflections controlling bridge designs. This is a potential problem as the benefits of using HPS can be reduced or even negated. If conservative state practices are employed, the problem is exacerbated, and even conventional steel bridges may be controlled by deflections. This study uses 10 study bridges that were chosen based on varying characteristics and applies the sample state and AASHTO live-load deflection criteria previously discussed.

Bridges were chosen from Missouri, Massachusetts, Wyoming, Utah, Illinois, West Virginia and Pennsylvania for this study, including four one-span, three two-span, two three-span and one four-span bridges. Four of the bridges use HPS in combination with conventional steel in a hybrid design. The bridges range in span length from 54 ft. to 200 ft. This variable provides a range of span lengths, which allows different live-loadings to control the deflection calculation. Truck loadings control design on shorter spans, while lane loadings can be significantly larger on longer spans. The design lanes range from two to six design lanes. The girder spacing ranges from approximately 5 ft. to a little over 9 ft. Design lanes and girder spacing are variables that determine the amount of live-load that gets applied to each girder line. Three of the bridges carry sidewalks. All of the bridges are composite in the positive moment regions. For the continuous spans, two are non-composite over the piers, and the remaining are composite in the negative moment region. The general characteristics of each of the study bridges are given below. The names used in figures and tables in later chapters are in parentheses.

Missouri Bridge A6101 (MO A6101)

The Missouri bridge in this study is a two-span bridge located on Route 224 overt Route 13 in Lafayette County, Missouri. A6101 is the first bridge in Missouri to utilize high performance steel. The bridge is composed of HPS grade 70W steel and conventional 50W steel. The bridge meets LFD and LRFD AASHTO criteria for live-load deflection.

Missouri bridge A6101 has the following characteristics:

- 2-span 138 ft. – 138 ft.
- L/D = 27.3
- Composite bridge in the positive moment regions, non-composite in negative
- 5-girder bridge – girders spaced at 8.96 ft.
- 3 design lanes
- No sidewalks
- HPS and conventional steel. HPS hybrid bridge is 17% lighter and costs 11% less than conventional grade 50 steel bridge (Davis 2003).

Utah Bridge over Asay Creek (UT Asay Creek)

The Utah bridge in this study is a single-span bridge located over the Asay Creek in Garfield County, Utah. The bridge is composed of conventional grade 50W steel and meets LFD and LRFD AASHTO criteria for live-load deflection.

Utah bridge over Asay Creek has the following characteristics:

- 1-span 96.4 ft.
- L/D = 23.7
- Composite bridge
- 6-girder bridge – girders spaced at 7.90 ft.
- 3 design lanes
- No sidewalks
- Conventional steel bridge

Massachusetts Bridge C-08-031 (MA Chelmsford)

The Massachusetts bridge in this study is a two-span bridge is located on State Route 4 over State Route 3 in Middlesex County, Massachusetts. HPS grade 70W steel is used in some flange sections of this bridge. The bridge meets LFD and LRFD AASHTO criteria for live-load deflection.

Massachusetts bridge C-08-031 has the following characteristics:

- 2-span 161 ft. – 161 ft.
- L/D = 37.7
- Composite bridge in the positive and negative moment regions
- 6-girder bridge – girders spaced at 7.80 ft.
- 3 design lanes
- No sidewalks
- HPS and conventional steel hybrid bridge

Wyoming Bridge over Little Laramie River (WY Little Laramie River)

The Wyoming bridge in this study is a single-span bridge located over Little Laramie on County Road No. 416 in Albany County, Wyoming. The bridge is composed of conventional 50W steel and meets LFD and LRFD AASHTO criteria for live-load deflection.

Wyoming bridge over Little Laramie River has the following characteristics:

- 1-span 96.4 ft.
- L/D = 23.1
- Composite bridge
- 4-girder bridge – girders spaced at 7.67 ft.
- 2 design lanes
- No sidewalks
- Conventional steel bridge

West Virginia Culloden Overpass 10462 (WV Overpass 10462)

The West Virginia bridge in this study is a three-span bridge is located over the CSX Railroad in Putam and Cabell County, West Virginia. This bridge is composed of HPS grade 70W steel and grade 100W steel along with conventional 50W steel. The bridge meets LFD and LRFD AASHTO criteria for live-load deflection for pedestrian bridges.

West Virginia Culloden Overpass 10462 has the following characteristics:

- 3-span 54 ft. – 80 ft. – 54 ft.
- L/D = 23.4 and 34.6
- Composite bridge in the positive and negative moment regions
- 7-girder bridge – girders spaced at 8.00 ft.
- 4 design lanes
- Carries sidewalks
- HPS and conventional steel hybrid bridge. HPS100W (Fy = 100 ksi used)

Illinois Bridge 079-4402 (IL FAS Route 860)

The Illinois bridge in this study is a four-span bridge located over the Old Mississippi River channel in Randolph County, Illinois. This bridge is composed of conventional 50W steel and meets LFD and LRFD AASHTO live-load deflection criteria.

Illinois Bridge 079-4402 has the following characteristics:

- 4-span 81 ft. – 129.5 ft. – 129.5 ft. - 81 ft.
- L/D = 19.4 and 31.4
- Composite bridge in the positive moment regions, non-composite in negative
- 5-girder bridge – girders spaced at 5.17 ft.
- 2 design lanes
- No sidewalks
- Conventional steel bridge

Pennsylvania SR 0329 Sec 04B (PA Northampton County)

The Pennsylvania bridge in this study is a single-span bridge located over Quarry Hall Road in Northampton County, Pennsylvania. The bridge is composed of conventional 50W steel and meets LFD and LRFD AASHTO live-load deflection criteria.

Pennsylvania SR 0329 Sec 04B has the following characteristics:
- 1-span 123 ft.
- L/D = 20.8
- Composite bridge
- 5-girder bridge – girders spaced at 9.00 ft.
- 3 design lanes
- No sidewalks
- Conventional steel bridge

Idaho Bridge A010 (ID A010)

The Idaho bridge in this study is a two-span bridge located on 9th Street over Interstate 90 in Kootenai County, Idaho. The bridge is composed of conventional 50W steel and meets LFD and LRFD AASHTO live-load deflection criteria for pedestrian bridges.

Idaho Bridge A010 has the following characteristics:
- 2-span 70.25 ft. – 70.25 ft.
- L/D = 34.1
- Composite bridge in the positive and negative moment regions
- 7-girder bridge – girders spaced at 6.83 ft.
- 3 design lanes
- Carries sidewalks
- Conventional steel bridge

Massachusetts Bridge Billerica (MA Billerica)

The Massachusetts bridge in this study is a single-span bridge located on Route 3 over Concord River in Middlesex County, Massachusetts. The bridge is composed of HPS grade 70W steel and conventional 50W steel. The bridge does not meet AASHTO LFD and LRFD criteria for live-load deflection.

Massachusetts Bridge Billerica has the following characteristics:
- 1-span 197 ft.
- L/D = 23.3
- Composite bridge
- 9-girder bridge – girders spaced at 9.40 ft.
- 6 design lanes
- No sidewalks
- HPS and conventional steel hybrid bridge

Wyoming Bridge over Laramie River (WY Laramie River)

The Wyoming bridge in this study is a three-span bridge located on Garfield Street over the Laramie River in Albany County, Wyoming. The bridge is composed of conventional 50W steel. The bridge meets AASHTO LFD and LRFD criteria for live-load deflection for pedestrian bridges.

Bridge over Laramie River has the following characteristics:
- 3-span 67.2 ft. – 89 ft. – 67.2 ft.
- L/D = 21.2 and 28.1
- Composite bridge in the positive and negative moment regions
- 6-girder bridge – girders spaced at 9.04 ft.
- 2 design lanes

- Carries sidewalks
- Conventional steel bridge

2.8 Structural Modeling and Analysis

2.8.1 Introduction

This section presents the basis for modeling and the structural analysis method for the LFD and LRFD specifications. The loading criteria for LFD and LRFD will also be explained in some detail. A complete description of the Missouri Bridge A6101 properties will be presented in this section, and Missouri Bridge A6101 will be used as an example to demonstrate the modeling and analysis methods used in the study. The ten study bridges are modeled and analyzed using the same methods in this research.

2.8.2 Modeling

Appropriate modeling is important for both AASHTO LFD and LRFD methods because the force effects are dependent on the model. One important consideration when defining a model is the overall behavior of the bridge. The way the bridge is assumed to behave defines the modeling for the longitudinal analysis. Another aspect to consider when modeling a bridge is the construction method. The construction method defines the stages of construction and the sequence of loading, and, therefore, need to be accounted for in the modeling.

Although composite bridges composed of steel and concrete are generalized as having composite sections, the interpretation of what composite means for structural analysis modeling is different. The definition of a composite section is one with proper shear connection between the concrete deck and the steel girder. This ensures that the concrete and steel both contribute to the flexural capacity when subjected to positive or negative bending. However, stiffness properties (composite vs. non-composite) used for structural analysis modeling may differ from those used for stress calculations. Therefore, the structural analysis modeling may use composite section properties while the design check provisions may use non-composite section provisions.

This research assumes that the construction for all the study bridges is unshored, meaning that the bridge supports itself during construction. In unshored construction, there are three stages of loading. The first stage consists of the girder bearing the self weight of the steel and the weight of the wet concrete deck. After the concrete has hardened and gained strength, the second stage begins. The remaining permanent loads applied at this stage are referred to as long-term loading, and they are applied to a long-term section. The long-term section properties account for time dependant creep of concrete. The final stage is where the concrete has cured and reached full capacity. Live loads, or short term loading, are then applied to the short-term section.

For both LRFD and LFD specifications, the assumed structural behavior over the piers is imperative in developing a model. The bridge model used for an LFD analysis differs from that used for an LRFD analysis. As is common in practice, the LFD method for design assumes that the concrete is cracked over the piers for both structural analysis and stress calculations, and the concrete in the negative moment regions is therefore ignored in the model. However, this does not apply to the LRFD analysis. Article 4.5.2.2 from the AASHTO LRFD Design Specifications says stiffness characteristics can be based on full participation of the concrete deck for structural analysis. The theory behind this is that minor cracking of concrete seems to have little effect on the behavior of the structure under elastic conditions. Therefore, the concrete can safely be modeled as uncracked in the negative moment regions for the purpose of analysis. However, the concrete is assumed to crack for stress calculations. To meet structural analysis needs for

LFD and LRDF, both a model ignoring the concrete deck over the pier and a model including the concrete deck over the pier were developed. For simplicity, the model including the concrete contribution is referred to as "prismatic" in this report, where as the model ignoring the concrete contribution is referred to as "non-prismatic." This terminology should not be confused with the section stiffness properties along the length (i.e. constant EI), which commonly vary along the length of the span.

For each analysis (LFD or LRFD), models for each sequence of loading were developed using either non-prismatic (LFD) or prismatic (LRFD) modeling. Section moments of inertia account for the differences between the steel girder, short-term and long-term stiffness in the structural analysis. For composite sections over the piers (shear connectors over the pier), the nonprismatic (LFD) models account for the reinforcing steel in the negative moment region—resulting in an increase in stiffness.

2.8.3 Loading and Analysis

The following section explains the method of loading as well as the analysis for the loading. The LFD and LFRD criteria for dead and live loads are both explained. The deflection criteria loading for LRD and LFRD are also included in this section. The structural analysis methods for the LFD and LRFD loadings are presented in this section as well.

2.8.3.1 LFD Loading

Section 3 of the AASHTO LFD specification includes the load requirements. The loads considered in the analysis consist of the live loads and dead loads according to the specification.

The dead load for the structure consists of the self weight of the girders and deck, sidewalk, curbs, parapets and railings, stay-in-place forms, and a wearing surface. The weight of these items can be estimated using the geometry along with the unit weight of the material provided in Article 3.3.6. Some of these dead loads are applied to the steel girder properties, while the remaining dead loads are applied to the long-term properties model.

The live load applied in the analysis is the standard HS20-44 highway live load. This loading includes a tractor truck with a semi-trailer or the corresponding lane load. The HS20 truck is defined using three point loads. The first axle has a magnitude of 8 kips, and the second and third axles both have magnitudes of 32 kips (Figure 2.1). The spacing between the first and second axle is fixed at a distance of 14 ft., and the spacing between the second and third axle can vary from 14 ft. to 30 ft. The HS 20-44 design lane is defined by a uniform load of 640 lbs per linear foot plus a moving concentrated load of 18 kips. For maximum negative moment in the design of continuous span bridges, an additional concentrated load of 18 kips is placed on an adjacent span in the series such that it causes the maximum effect. The live-load effect for design is the extreme effect caused by either the truck or the design lane loading with dynamic impact allowance. The LFD criterion for deflection, however, considers only the effects of the HS20 truck with dynamic impact allowance.

2.8.3.2 LRFD Loading

The LRFD loading is found in Section 3 of the LRFD AASHTO Specifications. Like the LFD analysis, the only loads considered for this LRFD analysis are the dead and live loads. The dead loads are calculated using the same procedures as those used in the LFD analysis.

The live load applied in the analysis is the standard HL93 LRFD loading. This loading includes the design truck plus the design lane, the design tandem plus the design lane, or 90 percent of a double truck plus 90 percent of the design lane. The design truck is defined the same as the HS 20-44 truck used in the

LFD analysis. The design lane is defined as a 640 pound per foot distributed load. A design tandem is defined as two concentrated loads with magnitudes of 25 kips spaced at 4 ft. The double truck is two design trucks spaced at a minimum of 50 ft. between the rear axle of the first truck and the front axle of the second located in adjacent spans to maximize pier moments. The LRFD defection criteria is defined as the maximum effect from either the truck with dynamic impact allowance or 25% of the truck with impact allowance plus the design lane.

2.8.3.3 CONSYS™ 2000 Analysis Software

In this study the CONSYS™ structural analysis program (LEAP 2005) is used to analyze the 10 study bridges using a single girder model. CONSYS™ is a product of LEAP Software for the analysis of static and transient loads on simple-span and multi-span bridges. It is capable of analyzing both LRFD and LFD loadings in both U.S. customary units and metric units. A complete library of loads for both methods is provided within the program. The loads can be modified to suit different study cases. Distribution factors, load factors, and impact factors can be included in the model and are user defined. Analysis can be performed for individual loads, combinations of loads and envelopes of loads. CONSYS™ computes moments, shears and deflections along the length of the girder. Influence lines for moments, shears and deflections are also generated.

Euler-Bernoulli two-dimensional beam elements are used in creating a CONSYS™ model. Each node within a CONSYS™ beam, or span, has two degrees of freedom: rotation and lateral translation with respect to the axis of the beam. A beam element, or segment, is modeled by inputting the modulus of elasticity and the moment of inertia. The span can be divided into segments of different lengths as necessary. Each segment of the beam can vary in length, modulus of elasticity and moment of inertia. The program defaults to 10 checkpoints evenly distributed along each span, but can be modified to any number of checkpoints at any location. Support conditions can be modeled as fixed, rollers or free ends.

For analysis, CONSYS™ utilizes a continuous beam model. Moments, shears and deflections of the indeterminate structure are computed using the standard stiffness method. The output consists of a table with shears, moments and deflections along the length of each span and their corresponding plots.

2.8.4 Missouri Bridge A6101 Modeling and Analysis Example

The following provides a detailed description of Missouri Bridge A6101. This section also provides the input used for CONSYS™ and the results obtained to be applied in the following chapters.

2.8.4.1 Missouri Bridge A6101

Missouri Bridge A6101 is a two span composite (positive moment region) steel bridge with symmetrical spans equal to 137.8 ft. in length. The plate girders are a hybrid design composed of HPS70W (yield strength of 70ksi) and 50W (yield strength of 50 ksi). The negative moment region over the pier is non-composite. The bridge is composed of five girders spaced at 8.96 ft. and has an overhang of 3.12 ft. The total bridge width is 42.06 ft. with a travel way width of 39.37 ft. The deck is 8.5 inches thick and has a haunch of 2.5 inches from the top of the web. The general layout of the bridge is shown in Figure 2.2, and the geometry is presented in Table 2.3 and Figure 2.3.

Table 2.3 Missouri Bridge A6101 Girder Geometry

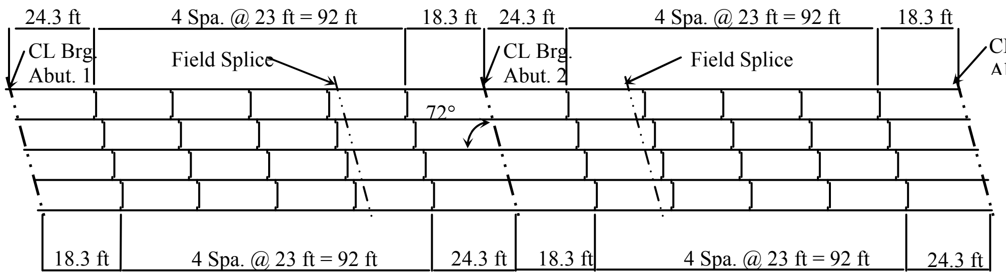
SPAN 1									
Length [ft]	Top Flange			Web			Bottom Flange		
	t_f [in]	b_f [in]	F_y [ksi]	t_f [in]	D [in]	F_y [ksi]	t_f [in]	b_f [in]	F_y [ksi]
0.00 - 95.80	0.787	12.598	50	0.472	59.055	50	0.787	16.535	70
95.80 - 137.5	1.260	20.472	70	0.511	59.055	50	1.260	20.472	70

SPAN 2									
Length [ft]	Top Flange			Web			Bottom Flange		
	t_f [in]	b_f [in]	F_y [ksi]	t_f [in]	D [in]	F_y [ksi]	t_f [in]	b_f [in]	F_y [ksi]
0.00 - 42.00	1.260	20.472	70	0.511	59.055	50	0.787	16.535	70
42.00 - 137.5	0.787	12.598	50	0.472	59.055	50	1.260	20.472	70

From this information, the section properties from Figure 2.3 are calculated to construct the models. The positive moment and negative moment regions have properties for the three stages, as shown in Table 2.4, that apply to the sequence of loads: steel girder, short-term composite and long-term composite. A model representing each stage shows the corresponding loads for both LFD and LRFD.

Table 2.4 Missouri Bridge A6101 Girder Cross Section Properties

			Steel, s	**Short-term, n**	**Long-term, 3n**
Positive Span 1					
Moment of Inertia:	I	[in^4]	28,466	78,678	58,793
Section Modulus:	S_{top}	[in^3]	886	12,221	6,556
	S_{bottom}	[in^3]	999	1,452	1,333
Negative					
Moment of Inertia:	I	[in^4]	56,396	119,292	89,938
Section Modulus:	S_{top}	[in^3]	1,832	11,555	4,534
	S_{bottom}	[in^3]	1,832	2,327	2,154



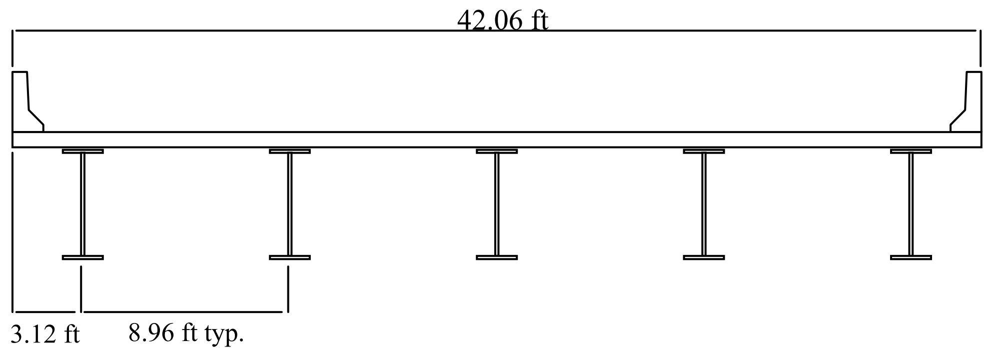
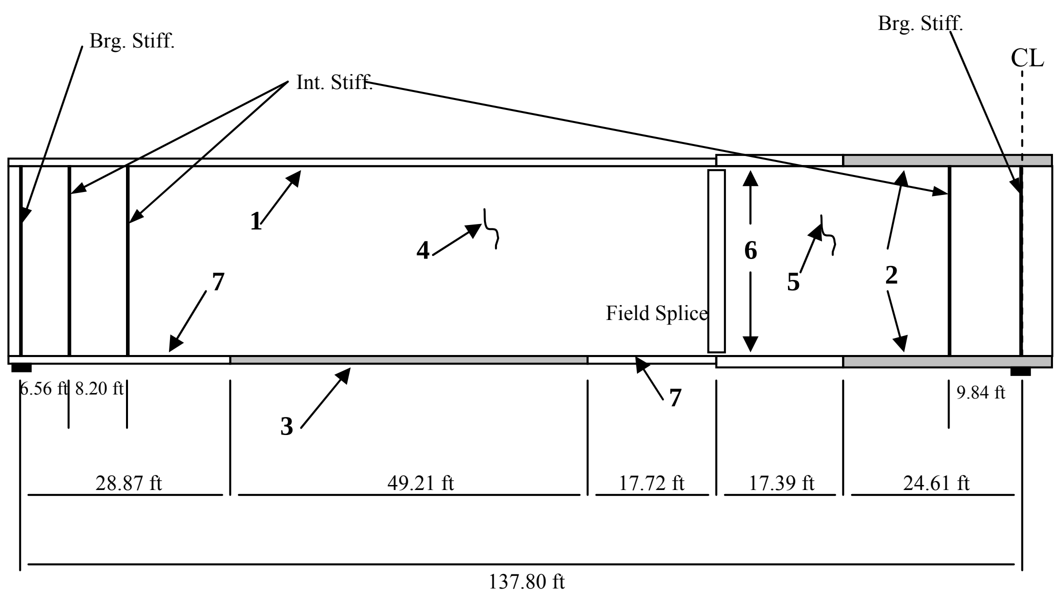


Figure 2.2 Missouri Bridge A6101 General Layout and Cross Section



1. 12.598 x 0.787 in (50 ksi)
2. 20.472 x 1.26 in (HPS70W)
3. 16.535 x 0.787 in (HPS70W)
4. 59.055 x 0.4724 in (50 ksi)
5. 59.055 x 0.5512 in (50 ksi)
6. 20.472 x 1.26 in (50ksi)
7. 16.535 x 0.787 in (50 ksi)

Figure 2.3 Missouri Bridge A6101 Girder Elevation

2.8.4.2 LFD Example

The LFD structural analysis model is a nonprismatic, noncomposite over the pier model. The construction stages were represented using three different CONSYS™ models. Each model varies according to the load stage cross section properties along the length and their corresponding loads.

The first load stage consists of the steel girder bearing the steel and deck self-weight. The moment of inertia of the steel girder is input along the length of the span and the self-weight was applied as a uniformly distributed load. The magnitude of the load varies for different segments depending on the cross section properties. The resulting moment diagram is shown in Figure 2.4.

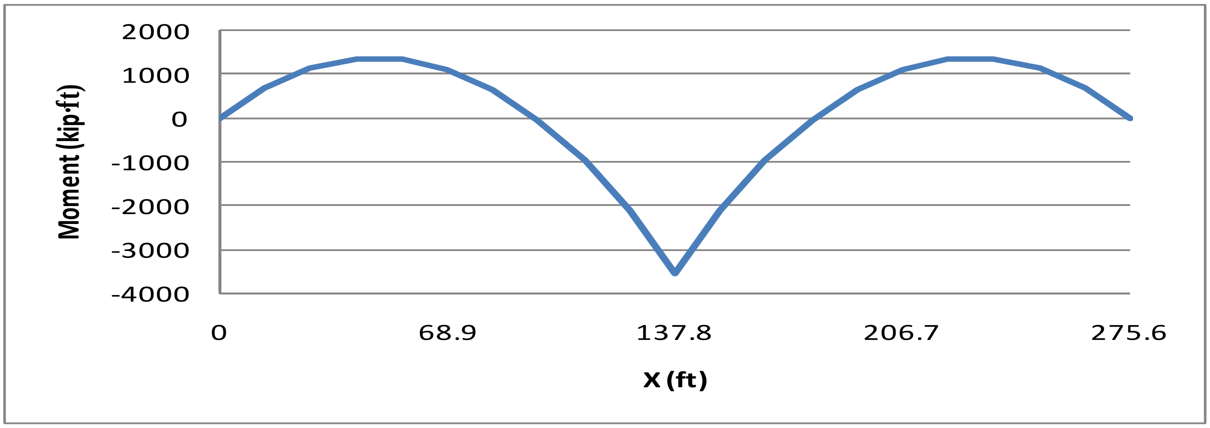


Figure 2.4 LFD Moment Diagram for Self-Weight Dead Load on Steel Girder Model

The second load stage, or long-term model, consists of composite moment of inertia calculated with the equivalent concrete contribution (considering creep) and the steel girder. In the positive moment regions, the long-term moment of inertia (composite) is applied and in the negative moment regions the steel girder moment of inertia (non-compostite) is applied. The remaining dead loads, such as wearing surfaces and curbs, are applied as a uniformly distributed load along the length of the girder. The resulting moment diagram is provided in Figure 2.5.

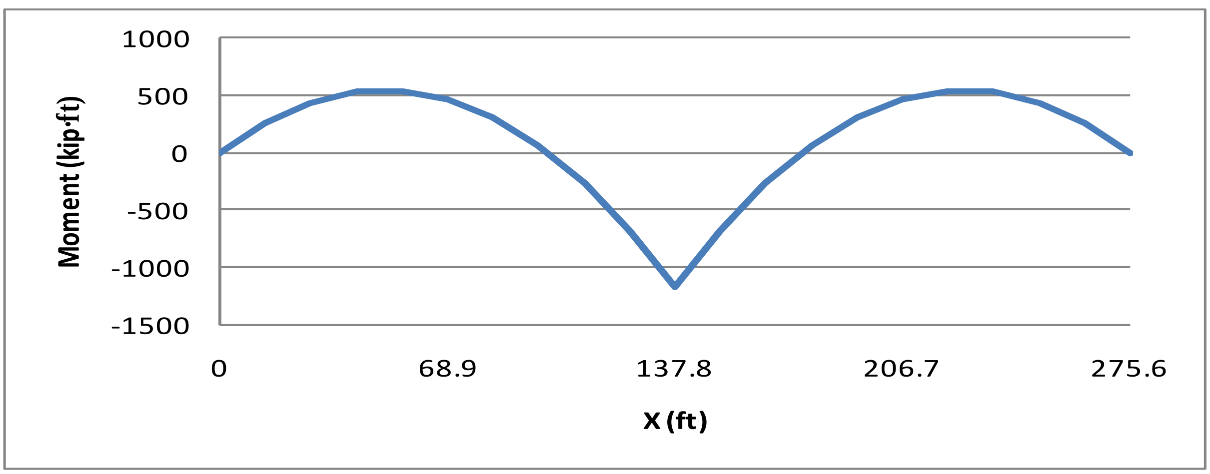


Figure 2.5 LFD Moment Diagram for Long-Term Dead Load on Long-Term Model

The last load stage, or short-term model, consists of composite moment of inertia calculated with the full concrete properties and the steel girder. In the positive moment regions, the short-term moment of inertia (composite) is applied, and in the negative moment regions the steel girder moment of inertia (non-composite) is applied. The HS20-44 live loads are applied as wheel line transient loads and a moment envelope is developed from the loading. The wheel line moment envelope obtained is shown in Figure 2.6.

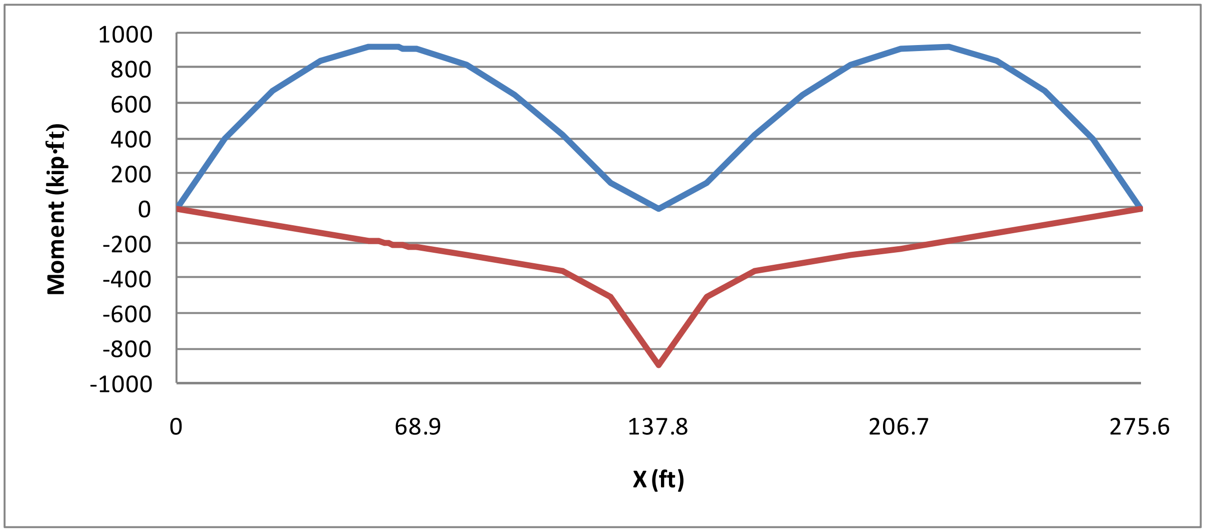


Figure 2.6 LFD Live-load Moment Envelope on Short Term Model

For deflection, different deflection live loading of the truck wheel line is applied to the short-term model resulting in the deflection envelope of Figure 2.7.

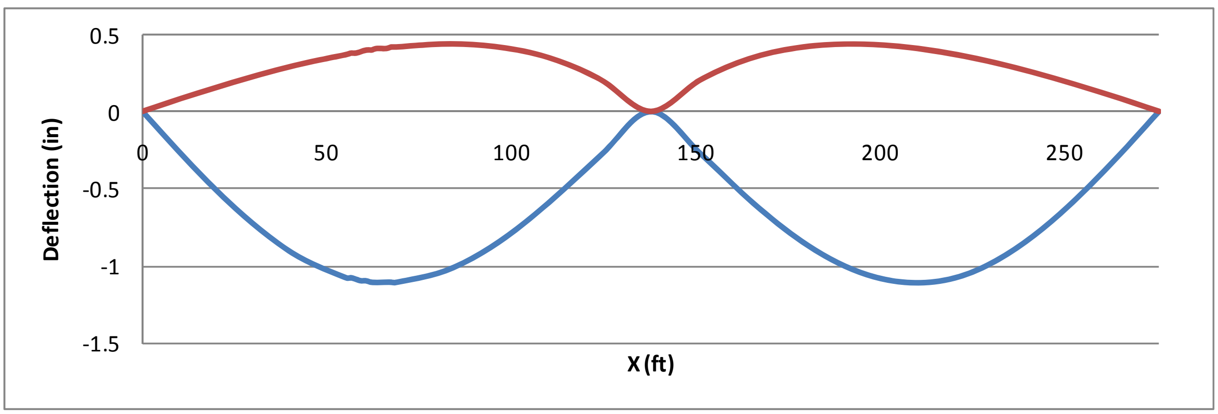


Figure 2.7 LFD Deflection Envelope for Deflection Loading on Short-Term Model

The CONSYS™ moments and deflections are used later in the LFD design example in Section 2.9. The maximum positive and negative moments and maximum deflection for Missouri Bridge A6101 are summarized in the Table 2.5.

Table 2.5 Missouri Bridge A6101 Summary of LFD Results

M_{DL1} (k-ft)		M_{DL2} (k-ft)		M_{WL} (k-ft)		Δ_{WL} (in)
Positive	Negative	Positive	Negative	Positive	Negative	Maximum
1345	3558	527	1166	920	901	1.11

where
M_{DL1} = the moment from self-weight on a single girder (steel girder model),

M_{DL2} = the moment caused by long-term dead load other than the self-weight on a single girder (long-term model),

M_{WL} = the moment caused by the live load from one wheel line (short-term model), and

Δ_{WL} = the deflection caused by the deflection live load from one wheel line (short-term model).

The live-load moments and deflections are modified by impact factors and distribution factors to determine the amount of live-load moment and live-load deflection that is applied to a single girder in the LFD provisions of Section 2.9.

2.8.4.3 LRFD Example

The LRFD structural analysis model is a prismatic composite over the pier model. The construction stages are represented using three different CONSYS™ models. Each model varies according to the load stage cross section properties along the length and their corresponding loads.

The first load stage consists of the steel girder bearing the steel and deck self-weight. The moment of inertia of the steel girder is input along the length of the span, and the self-weight is applied as a uniformly distributed load. The magnitude of the load varies for different segments depending on the cross section properties. The resulting moment diagram is shown in Figure 2.8.

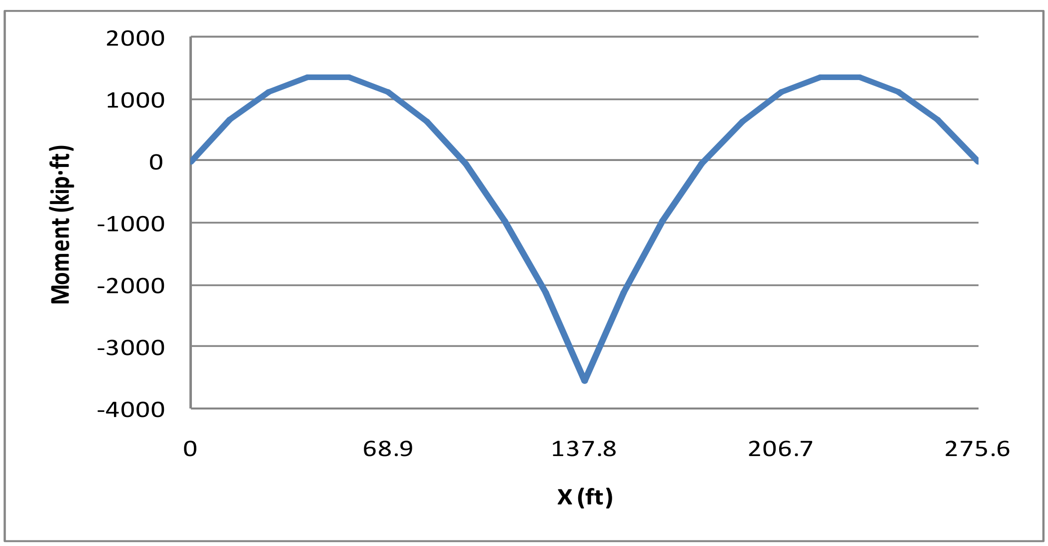


Figure 2.8 LRFD Moment Diagram for Self-Weight Dead Load on Steel Girder Model

The second load stage, or long-term model, consists of composite moment of inertia calculated with the equivalent concrete contribution (considering creep) and the steel girder. In the positive and negative moment regions, the long-term moment of inertia (composite) was applied. For LRFD there are two load cases applied, resulting in two moment diagrams. The first load case consists of the remaining dead loads, such as curbs but not wearing surfaces. These remaining dead loads are applied as a uniformly distributed load along the length of the girder. The resulting moment diagram is provided in Figure 2.9.

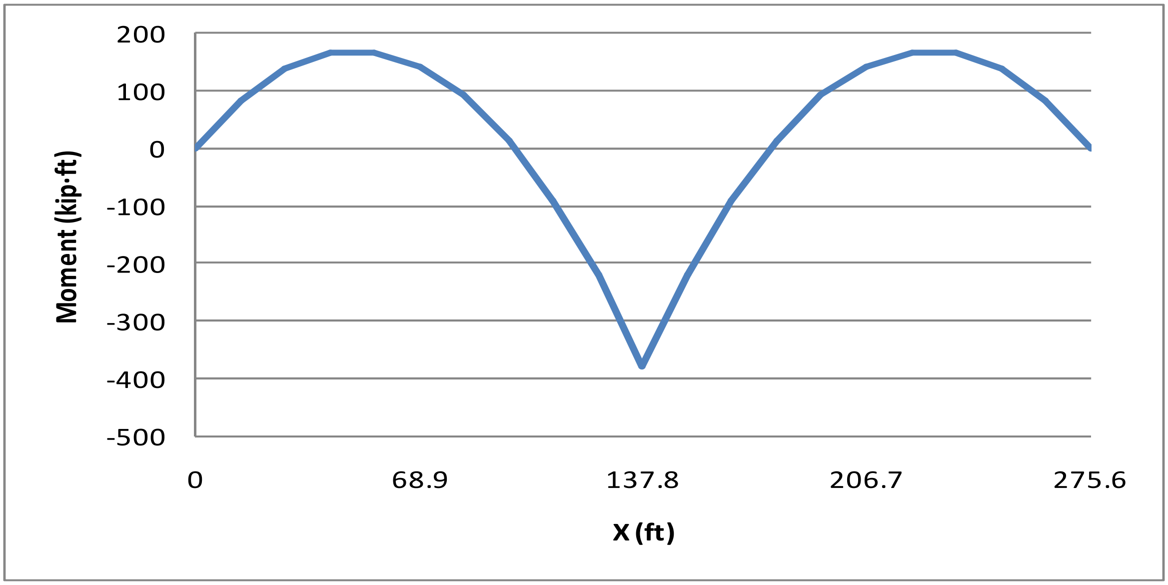


Figure 2.9 LRFD Moment Diagram for Long-Term Dead Load on Long-Term Model

The second load case consists of the wearing surface. The load of the wearing surface is applied as a uniformly distributed load along the length of the girder. The resulting moment diagram is provided in Figure 2.10.

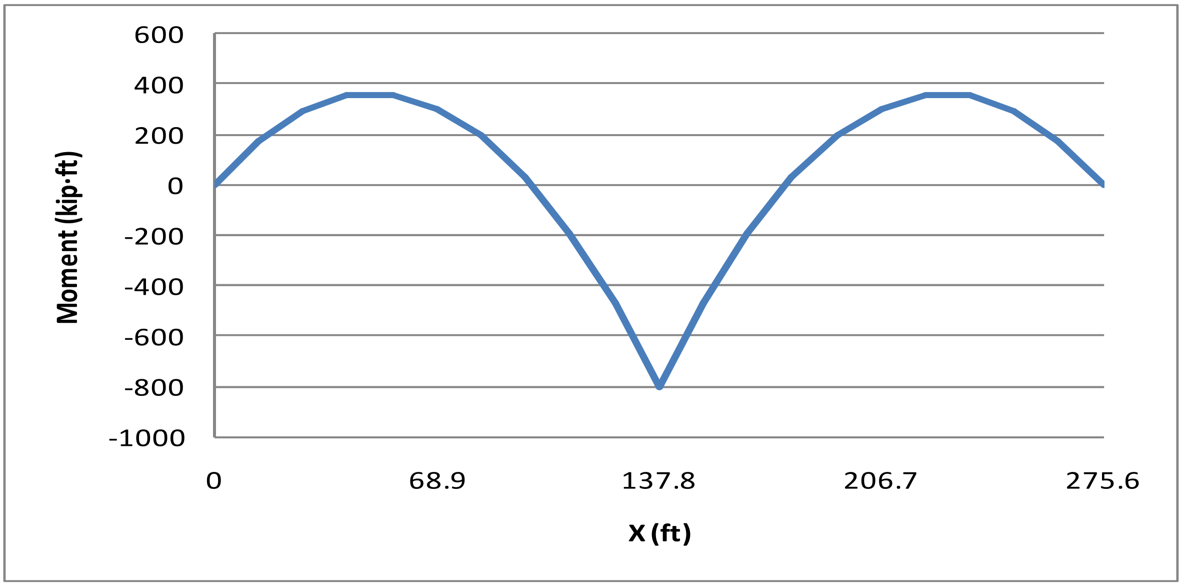


Figure 2.10 LRFD Moment Diagram for Long-Term Wearing Surface Load on Long-Term Model

The last load stage, or short-term model, consists of composite moment of inertia calculated with the full concrete properties and the steel girder. In the positive and negative moment regions, the short-term moment of inertia (composite) is applied. A load envelope is utilized to give the results of the HL93 live loading. The moment envelope from the live loading with the impact factor applied to the truck and tandem obtained from CONSYS is shown in Figure 2.11.

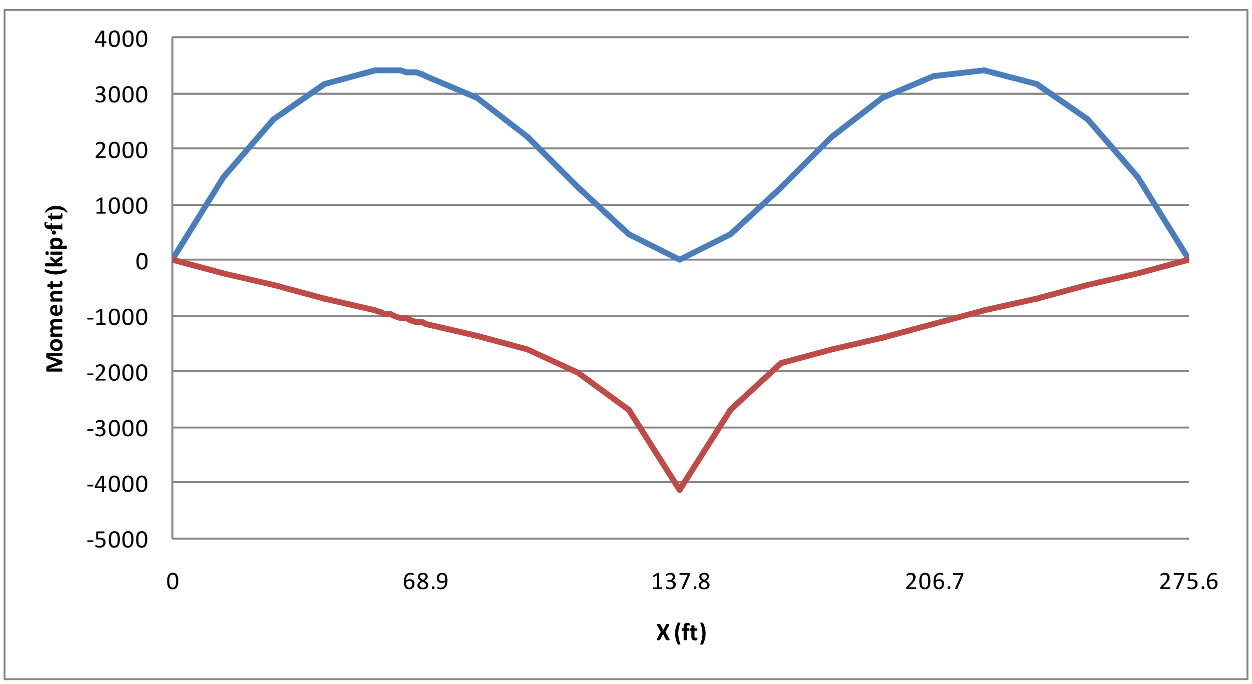


Figure 2.11 LRFD Live-load Moment Envelope with Impact Factor Applied to Truck and Tandem on Short-Term Model

For deflection, a loading envelope gave the results of the worst of two live loads applied to the short-term model resulting in the deflection envelope presented in Figure 2.12.

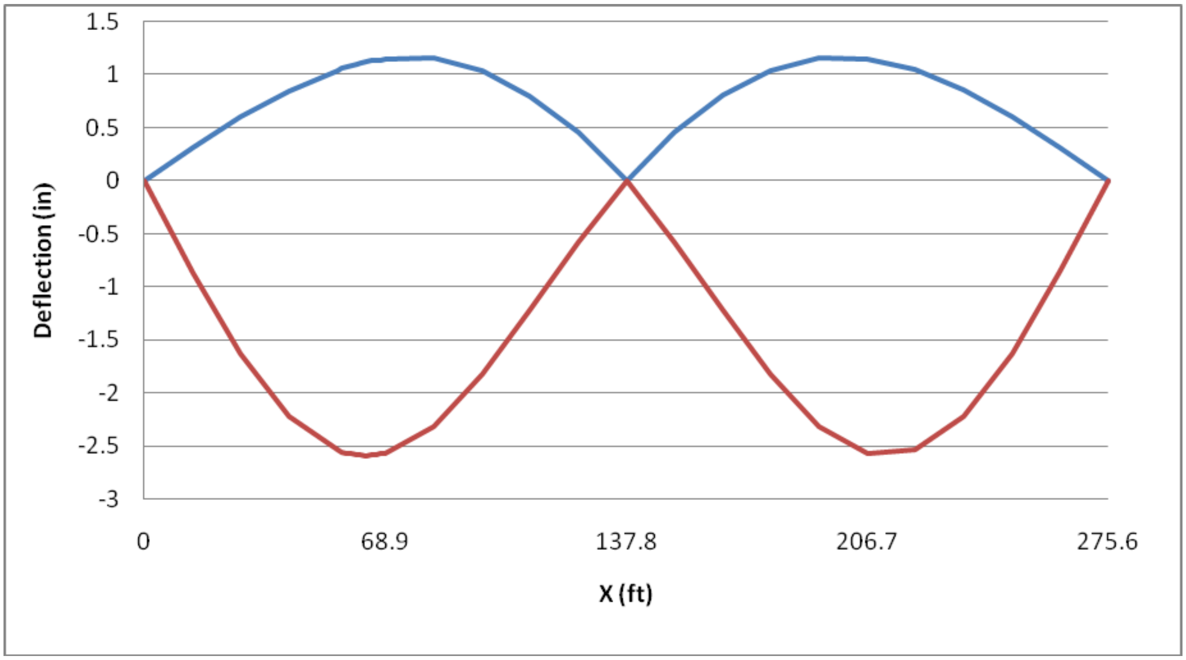


Figure 2.12 LRFD Deflection Envelope for Deflection Loading on Short-Term Model

The CONSYS™ moments and deflections are later used in the LRFD design example in Section 2.10. The maximum positive and negative moments and maximum deflection for Missouri Bridge A6101 are summarized in Table 2.6.

Table 2.6 Missouri Bridge A6101 Summary of LRFD Results

M_{DC1} (k-ft)		M_{DC2} (k-ft)		M_{DW} (k-ft)		M_{WT+I} (k-ft)		Δ_{WT+IM} (in)
Positve	Negative	Positive	Negative	Positive	Negative	Positive	Negative	Maximum
1345	3558	168	378	352	805	3429	4122	2.59

where

M_{DC1} = the moment from self-weight on a single girder (steel girder model),

M_{DC2} = the moment caused by long-term dead load other than the self-weight and wearing surface on a single girder (long-term model),

M_{DW} = the moment caused by long-term dead load from wearing surface on a single girder (long-term model) and

M_{WT+I} = the moment caused by the live load from live loading with impact factor applied to truck and tandem (short-term model).

Δ_{WT+IM} = The deflection caused by the deflection live load with impact factor applied to truck (short-term model)

The live load moments and deflections are modified by distribution factors to determine the amount of live-load moment and live-load deflection that is applied to a single girder in the LRFD provisions of Section 2.10.

2.8.5 Summary

Section 2.8 details the modeling, loading and structural analysis methods used to determine the behavior of the study bridges. The method of modeling the bridges for both LFD and LRFD analysis is covered in some detail. The differences between the LFD and LRFD models, such as prismatic versus non-prismatic, are explained. The system of loading the bridges was covered for both LFD and LRFD specifications along with a description and detailing of the differences that exist between the specifications. The computer structural analysis program, CONSYS, is described, including the program's methods and capabilities. A design example was provided for both LFD and LRFD analysis using the Missouri Bridge A6101. An elevation and plan view are provided, and the results are shown in Figures 2.4-2.12 and in Tables 2.5 and 2.6. These procedures are used on the remaining bridges in sections 3 and 4.

2.9 LFD Design Criteria

The following section presents the AASHTO LFD design criteria for strength, overload and deflection limits. Missouri Bridge A6101 will be used to demonstrate the provisions. The procedures here will be applied to the ten study bridges in section 3.

The LFD example of Missouri Bridge A6101 uses a non-prismatic model. The moments, as well as deflections, are obtained using CONSYS™. Missouri Bridge A6101 is a noncomposite bridge in the negative moment region; therefore, the steel reinforcement has no effect on the cross section properties over the piers. The 2002 AASHTO Standard Specifications for Highway Bridges are referenced.

2.9.1 LFD Load Combinations

Using the general combination equation given in Article 3.22.1 along with Table 3.22.1A, the Group(I) equation for design is that the design capacity must exceed the effects from

$$Group(I) = 1.3\left[D + \frac{5}{3}(L+I)\right] \tag{2-1}$$

where
D = dead load on the girder,
L = live load on the girder (modified live-load moments from Section 2.8) and
I = live-load impact allowance.

An additional serviceability requirement for overload, found in Article 10.5.7, must also be met. This requirement states that the stresses caused by $D + \frac{5}{3}(L+I)$ must be less than or equal to 0.95•R•F_{yf} for a composite section where R is the hybrid reduction factor and F_{yf} is the yield stress of the steel girder flange. The impact factor, I, and the distribution factor applied to the live load, L, are described in the following section.

2.9.2 Impact and Distribution Factors

To account for the dynamic load effect caused by the suspension system of a vehicle and the dynamic properties of the bridge, an impact allowance is calculated and is expressed as a fraction of the live load. The following formula is used for impact:

$$I = \frac{50}{L+125} \leq 0.30 \tag{2-2}$$

For Missouri Bridge A6101

$$I = \frac{50}{137.8+125} = 0.190$$

where L = length in feet of the portion of the span that is loaded to produce the maximum stress in the member.

For continuous spans, L is equal to the length of the span under consideration for the positive moment, and L is equal to the average of the two adjacent loaded spans for the negative moment. The impact allowance has an upper bound limit of 30%. For the purpose of this study, impact was calculated for positive and negative moment regions, and the largest value is applied to the entire bridge.

The moment distribution factor is used to apply the appropriate amount of live load to a single girder from the wheel line (half truck or half lane) loading. CONSYS™ analyzes force effects for a wheel line loading and the distribution factor determines the amount on a single girder. For bridges with two or more lanes and girder spacing less than 14 ft., the distribution factor for an interior girder is calculated as follows:

$$DF = \frac{S}{5.5} \tag{2-3}$$

For Missouri Bridge A6101

$$DF = \frac{8.96\,ft}{5.5} = 1.629$$

where S = girder spacing.

The distribution factor calculated from this equation applies to a wheel load, which is half of the HS 20-44 traffic load. All live-load values used in the following LFD analysis calculations are wheel load, not truck loads. However, the truck load (two times the wheel load) is used in the results section for comparison purposes.

AASHTO assumes all girders deflect equally when calculating live-load deflections. The deflection distribution factor distributes the deflection to the system of girders assuming whole 12 ft. lanes and is given by the following equation:

$$DF_{\Delta} = 2 \cdot i \cdot \frac{\#\, of\ lanes}{\#\, of\ girders} \tag{2-4}$$

For Missouri Bridge A6101

$$DF_{\Delta} = 2 \cdot 0.9 \cdot \frac{3}{5} = 1.08$$

where i = the load intensity reduction factor as provided in AASHTO LFD Article 3.12.1.

The i factor is used to reduce the total deflection when there are more than two lanes, assuming not all the lanes will be fully loaded. The factor of 2 is used because LFD (and CONSYS) determines deflections based on a wheel line; the 2 factor modifies the deflection to a lane deflection.

2.9.3 Negative Moment Region

The negative moment regions must satisfy the flexural strength limit state, as well as an overload requirement to control permanent deformations. Both are shown in the following calculations. For Missouri Bridge A6101 the negative moments at the interior pier used in this analysis are: M_{DL1} = 3358 k-ft, M_{DL2} = 1166 k-ft, and M_{WL} = 901 k-ft (see Section 2.8). The live-load moment is factored by the lane distribution factor and impact as follows:

$$M_{L(1+I)} = M_{WL} \cdot DF \cdot (1 + I) \qquad (2\text{-}5)$$

For Missouri Bridge A6101

$$M_{L(i+I)} = 902k \cdot ft \cdot 1.629 \cdot (1 + 0.19) = 1747k \cdot ft$$

where

M_{DL1} = the moment from self-weight on a single girder,

M_{DL2} = the moment caused by any dead load other than the self-weight on a single girder,

M_{WL} = the moment caused by the live load from one wheel line and

$M_{L(i+I)}$ = the live load plus impact moment on a single girder.

2.9.3.1 Strength Limit State Design Check

Group I Limit State Design Check (Equation 2-1) is a flexural design strength check that must be satisfied for safety. The maximum bending strength of the girder is dependent on whether the steel section is compact or noncompact according to Articles 10.48.1 and 10.48.2. Since Missouri Bridge A6101 meets the requirement for a noncompact section in the negative moment region and the section is noncomposite, the maximum strength of the section is

$$M_u = R \cdot M_y \qquad (2\text{-}6)$$

For Missouri Bridge A6101

$$M_u = 0.981 \cdot 10685k \cdot ft = 10482k \cdot ft$$

where

R = the hybrid reduction factor;

M_y = the yield moment.

or, if working in stresses,

$$f_u = R \cdot F_{yf} \tag{2-7}$$

For Missouri Bridge A6101

$$f_u = 0.981 \cdot 70 \cdot ksi = 68.7ksi$$

The compression flange is the critical flange, and the maximum flange stress due to the Group I loading (Equation 2-1) in this flange is given as

$$f_f = 1.3 \cdot \left(\frac{M_{DL1}}{S_{s\_bottom}} + \frac{M_{DL2}}{S_{bottom}} + \left(\frac{5}{3}\right) \cdot \frac{M_{L(1+I)}}{S_{bottom}} \right) \tag{2-8}$$

For Missouri Bridge A6101

$$f_f = 1.3 \cdot \left(\frac{3558k \cdot ft}{1832in^3} + \frac{1166k \cdot ft}{1832in^3} + \left(\frac{5}{3}\right) \cdot \frac{1747k \cdot ft}{1832in^3} \right) \cdot \frac{12in}{1ft} = 65.0ksi$$

where

S_{s_bottom} = the section modulus at the bottom using the steel girder only and

S_{bottom} = the section modulus at the bottom of the composite section of the girder and reinforcement.

To satisfy the strength limit state, the maximum factored design stress must be less than the maximum strength, or

$$f_u \geq f_f \tag{2-9}$$

For Missouri Bridge A6101

$$f_u = 68.7\ ksi > f_f = 65.0\ ksi$$

The design stress is less than the maximum strength, and this limit state is met.

2.9.3.2 Overload Design Check

To meet the overload design check given in Article 10.57.2, the flange stress caused by the load combination *D+5(L+I)*/3 must be less than $0.95RF_{yf}$= 65.3 ksi for Missouri Bridge A6101. The flange stress in the flange can be determined by

$$f_f = \frac{M_{DL1}}{S_{s\_bottom}} + \frac{M_{DL2}}{S_{bottom}} + \left(\frac{5}{3}\right) \cdot \frac{M_{L(1+I)}}{S_{bottom}} \tag{2-10}$$

For Missouri Bridge A6101

$$f_f = \left(\frac{3558k \cdot ft}{1832in^3} + \frac{1166k \cdot ft}{1832in^3} + \left(\frac{5}{3}\right) \cdot \frac{1747k \cdot ft}{1832in^3}\right) \cdot \frac{12in}{1ft} = 50.0ksi$$

The strength and overload design checks for the girder is satisfied.

2.9.4 Positive Moment Region

Similar to the negative moment region, sample calculations are shown for the design checks of the flexural strength and overload limit states using the LFD design method. These regions act compositely with the concrete deck. For Missouri Bridge A6101, the positive moments used in the analysis are: M_{DL1} = 1345 k-ft., M_{DL2} = 527 k-ft., and M_{WL} = 920 k-ft. (see Section 2.8). As in the negative moment region, using Equation (2-5) the live-load moment is factored as follows:

$$M_{L(1+I)} = M_{WL} \cdot DF \cdot (1 + I)$$

For Missouri Bridge A6101

$$M_{L(1+I)} = 920kip \cdot ft \cdot 1.629 \cdot (1 + 0.19) = 1782k \cdot ft$$

2.9.4.1 Strength Limit State Design Check

Article 10.50.1 gives the criteria for composite girders in positive bending. Similar to the negative moment regions, the maximum bending strength depends on whether the section is compact or noncompact. However, because the bridge has continuous spans, and the negative moment pier sections are noncompact, the positive moment regions are automatically limited to the moment capacity at first yield. Using stresses to perform the analysis, the strength is limited to

$$f_u = R \cdot F_{yf} \tag{2-11}$$

For Missouri Bridge A6101

$$f_u = 0.957 \cdot 70ksi = 67.0$$

where

$$f_u \geq f_f \tag{2-12}$$

The maximum flange stress due to the Group I loading is given as

$$f_f = 1.3 \cdot \left(\frac{M_{DL1}}{S_{s\_bottom}} + \frac{M_{DL2}}{S_{3n\_bottom}} + \left(\frac{5}{3}\right) \cdot \frac{M_{L(1+I)}}{S_{n\_bottom}} \right) \tag{2-13}$$

For Missouri Bridge A6101

$$f_f = 1.3 \cdot \left(\frac{1345k \cdot ft}{999in^3} + \frac{527k \cdot ft}{1333in^3} + \left(\frac{5}{3}\right) \cdot \frac{1782k \cdot ft}{1452in^3} \right) \cdot \frac{12in}{1ft} = 59.1ksi$$

where

S_{n_bottom} = the section modulus at the bottom of the composite section for short-term loads and

S_{3n_bottom} = the section modulus at the bottom of the composite section for long-term loads.

The design stress is less than the maximum strength, and this limit state is met.

2.9.4.2 Overload Design Check

Like the negative moment regions, to meet the overload design, the flange stress caused by the load combination $D+5(L+I)/3$ must be less than $0.95RF_{yf}$= 63.7 ksi. The flange stresses in the flange of the composite section are

$$f_f = \frac{M_{DL1}}{S_{s\_bottom}} + \frac{M_{DL2}}{S_{3n\_bottom}} + \left(\frac{5}{3}\right) \cdot \frac{M_L(1+I)}{S_{n\_bottom}} \tag{2-14}$$

For Missouri Bridge A6101

$$f_f = \left(\frac{1345k \cdot ft}{999in^3} + \frac{527k \cdot ft}{1333in^3} + \left(\frac{5}{3}\right) \cdot \frac{1782k \cdot ft}{1452in^3} \right) \cdot \frac{12in}{1ft} = 45.4ksi$$

The strength overload design checks this for composite girder is satisfied.

2.9.5 Live-load Deflection Criteria

From the LFD CONSYS model, the deflection for a truck wheel line of the live load without impact for Missouri Bridge A6101 (see Section 2.8) is

$$\Delta_{WL} = 1.11\, in$$

To determine the maximum deflection of a girder, this deflection value is multiplied by the deflection distribution factor and the impact. The resulting maximum deflection is

$$\Delta_{LFD} = DF_{\Delta} \cdot \Delta_{WL} \cdot (1+I) \tag{2-15}$$

For Missouri Bridge A6101

$$\Delta_{LFD} = 1.08 \cdot 1.11 in \cdot (1.19) = 1.43\ in$$

The maximum allowable deflection for a bridge without pedestrian traffic given by AASHTO is

$$\Delta_{ALLOWABLE} = \frac{L}{800} \tag{2-16}$$

For Missouri Bridge A6101

$$\Delta_{ALLOWABLE} = \frac{1654 in}{800} = 2.07 in.$$

where L = span length

2.9.6 Rating Factors

When designing a bridge, dead load will always take up a certain amount of the strength. Once the dead loads are accounted for, the remaining strength can be used by live load. This means an actual live load of a higher magnitude could be applied to reach full capacity of the bridge. The strength rating factor can give perspective on how much a section is over-designed or under-designed for strength with respect to the standard LFD live loads. For optimal design, the rating factor should be equal to "1." The rating factor for the positive moment regions and negative moment regions are both calculated and the smaller of the two controls for the bridge. Similarly, a rating factor for overload is obtained to determine how much more live load can be applied to the bridge to reach the maximum overload limit state. The strength and overload rating factor calculations for Missouri Bridge A6101 are presented below for negative and positive sections.

2.9.6.1 Strength Rating Factor Sample Calculations

The strength rating factors are calculated using stresses because the section is non-compact in the negative moment region, which also limits the positive moment region to use stresses. The moments previously used are converted to kip-inches in the following equations:

$$RF_{Strength\_Negative} = \frac{3}{5} \cdot \frac{S_{bottom}}{M_L \cdot (1+I)} \cdot \left(\frac{R \cdot F_{yf}}{1.3} - \frac{M_{DL1}}{S_{s\_bottom}} - \frac{M_{DL2}}{S_{bottom}} \right) \tag{2-17}$$

For Missouri Bridge A6101

$$RF_{Strength\_Negative} = \frac{3}{5} \cdot \frac{1832\ in^3}{20959\ k \cdot in} \cdot \left(\frac{0.981 \cdot 70\ ksi}{1.3} - \frac{42{,}696\ k \cdot in}{1832\ in^3} - \frac{13{,}992\ k \cdot in}{1832\ in^3} \right)$$

$$RF_{Strength\_Negative} = 1.15$$

Similarly, the positive moment rating factor is given as

$$RF_{Strength\_Positive} = \frac{3}{5} \cdot \frac{S_{n\_bottom}}{M_L \cdot (1+I)} \cdot \left(\frac{R \cdot F_{yf}}{1.3} - \frac{M_{DL1}}{S_{s\_bottom}} - \frac{M_{DL2}}{S_{3n\_bottom}} \right) \qquad (2\text{-}18)$$

For Missouri Bridge A6101

$$RF_{Strength\_Positive} = \frac{3}{5} \cdot \frac{1452\ in^3}{21{,}390\ k \cdot in} \cdot \left(\frac{0.957 \cdot 70\ ksi}{1.3} - \frac{16{,}140\ k \cdot in}{999\ in^3} - \frac{6{,}324\ k \cdot in}{1333\ in^3} \right)$$

$$RF_{Strength\_Positive} = 1.25$$

The bottom section modulus is used because the concrete in the composite section takes most of the compressive stresses caused by the loads and raises the neutral axis. This makes the tensile stresses in the steel more critical.

2.9.6.2 Overload Rating Factor Sample Calculations

The overload rating factors are calculated using stresses for negative and positive sections because overload is a stress requirement:

$$RF_{Overload\_Negative} = \frac{3}{5} \cdot \frac{S_{bottom}}{M_L \cdot (1+I)} \cdot \left(0.95 R \cdot F_{yf} - \frac{M_{DL1}}{S_{s\_bottom}} - \frac{M_{DL2}}{S_{bottom}} \right) \qquad (2\text{-}19)$$

For Missouri Bridge A6101

$$RF_{Overload\_Negative} = \frac{3}{5} \cdot \frac{1832\ in^3}{20964\ k \cdot in} \cdot \left(0.95 \cdot 0.981 \cdot 70 ksi - \frac{42{,}696\ k \cdot in}{1832\ in^3} - \frac{13{,}992\ k \cdot in}{1832\ in^3} \right)$$

$$RF_{Overload\_Negative} = 1.80$$

Similarly, the positive moment rating factor is given as

$$RF_{Overload\_Positive} = \frac{3}{5} \cdot \frac{S_{n\_bottom}}{M_L \cdot (1+I)} \cdot \left(0.95 \cdot R \cdot F_{yf} - \frac{M_{DL1}}{S_{s\_bottom}} - \frac{M_{DL2}}{S_{3n\_bottom}} \right) \qquad (2\text{-}20)$$

For Missouri Bridge A6101

$$RF_{Overload\_Positive} = \frac{3}{5} \cdot \frac{1452\ in^3}{21{,}390\ k \cdot in} \cdot \left(0.95 \cdot 0.957 \cdot 70 ksi - \frac{16{,}140\ k \cdot in}{886\ in^3} - \frac{6{,}324\ k \cdot in}{1333\ in^3} \right)$$

$$RF_{Overload\_Positive} = 1.74$$

2.9.7 Summary of LFD Design Requirements

The previous section presents the strength, overload, and deflection requirements for the AASHTO LFD method. It also uses Missouri Bridge A6101 to demonstrate the procedure. These design requirements are applied to the 10 bridges in the study set in section 3. The state specific deflection criteria from each of the study states will be presented and demonstrated in section 3.

The strength and overload rating factors are presented, demonstrated and reported in section 3. However, this work only examines the behavior of the "as-built" bridges, and rating factor optimization will be investigated in future work.

2.10 LRFD Analysis Example

The following section presents the AASHTO LRFD design criteria for Strength I, Service II and deflection limits. Missouri Bridge A6101 is used to demonstrate the provisions. The procedures here will be applied to the ten study bridges in section 3.

This LRFD analysis example uses a prismatic model. The moments as well as deflections are obtained using CONSYS^{TM}. Missouri Bridge A6101 is a non-composite over the pier; therefore, the steel reinforcement has no effect on the cross section properties over the piers. AASHTO's 3rd Edition of the *LRFD Bridge Design Specifications* are referenced.

2.10.1 Load Combinations

Using combination Equation (3.4.1-1) as well as Table 3.1 and 3.2, the Strength I and Service II equations for design are that the design capacity must exceed the effects from

$$Strength\ I = 1.25 \cdot DC1 + 1.25 \cdot DC2 + 1.5 \cdot DW + 1.75 \cdot (LL + IM) \qquad (2\text{-}21)$$

$$Service\ II = DC1 + DC2 + DW + 1.30 \cdot (LL + IM) \qquad (2\text{-}22)$$

where

DC1 = the self-weight of the composite section on the girder,

DC2 = the weight of remaining permanent loads on the single girder,

DW = the weight of any future wearing surface on a girder and

$LL + IM$ = the live load applied according to the LRFD specifications.

2.10.2 Impact and Distribution Factors

The impact allowance for the LRFD design method is a constant 33% for the Strength I and Service II limit states per AASHTO LRFD Table 3.6.2.1-1. The factor applied to the live load is a constant 1.33. This impact factor is only applied to the design truck and tandem, and not the design lane, and for this reason it is applied in the loading directly.

The moment distribution factor is used to apply the appropriate amount of live load to a single girder from the standard HL93 loading. CONSYS^{TM} in LRFD analyzes force effects of the standard HL93 loading, and the distribution factor determines the amount on a single girder. For a steel girder bridge with a concrete deck, the distribution factor for two or more design lanes loaded is:

$$mg = 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \cdot \left(\frac{S}{L}\right)^{0.2} \cdot \left(\frac{K_g}{12L \cdot t_s^{\,3}}\right)^{0.1} \qquad (2\text{-}23)$$

For Missouri Bridge A6101

$$mg = 0.075 + \left(\frac{8.96\,ft}{9.5}\right)^{0.6} \cdot \left(\frac{8.96\,ft}{137.8\,ft}\right)^{0.2} \cdot \left(\frac{1{,}323{,}812\,in^4}{12 \cdot 137.8\,ft \cdot (8.66\,in)^3}\right)^{0.1} = 0.646$$

where

S = girder spacing,

L = span length,

t_S = slab thickness,

K_g = longitudinal stiffness parameter.

For positive moment regions, L is the length of the span under consideration, and for negative moment regions, L is the average length of the two adjacent spans. For this analysis, distribution factors are calculated for all positive and negative moment regions, and the largest value is applied to the entire bridge. Distribution factors for LRFD are roughly two times those for LFD, since LRFD uses a whole lane and LFD uses a wheel line (one-half lane).

AASHTO assumes all girders deflect equally when calculating live-load deflections. The deflection distribution factor distributes the deflection to the system of girders assuming 12 ft lanes and is given by the equation

$$DF_\Delta = m \cdot \frac{\#\,of\ lanes}{\#\,of\ girders} \qquad (2\text{-}24)$$

For Missouri Bridge A6101

$$DF_\Delta = 0.85 \cdot \frac{3}{5} = 0.51$$

where m = the multiple presence factor as provided in AASHTO LRFD Table 3.6.1.1.2-1

The m factor is used to reduce the total deflection when there are more than two lanes, assuming not all the lanes will be fully loaded.

2.10.3 Negative Moment Region

The negative moment regions must satisfy the flexural Strength I limit state as well as a Service II limit state to control permanent deformations. Both are shown in the following calculations. The moments used in this analysis are M_{DC1} = 3558 k-ft, M_{DC2} = 378 k-ft, M_{DW} = 805 k-ft and M_{WL+IM} = 4122 k-ft (see Section 2.8). The live-load moment is factored by the lane distribution factor for moment:

$$M_{LL+IM} = M_{WT+IM} \cdot mg$$

For Missouri Bridge A6101

$$M_{LL+IM} = 4122k \cdot ft \cdot 0.646 = 2663k \cdot ft$$

2.10.3.1 Strength I Limit State Design Check

Strength I limit state is a flexural design strength check that must be satisfied for safety. In this research, it is assumed that the bottom flange is braced adequately such that lateral tortional buckling and flange local buckling will not affect the yield strength of the compression flange. The maximum bending strength of the girder is dependent on whether the steel section is compact or non-compact according to Article 6.10.2.3 and Appendix A. Since Missouri Bridge A6101 meets the requirements for a non-compact section in the negative moment region, the maximum strength of the section is

$$F_{nc} = \phi_f \cdot R_b \cdot R_h \cdot F_{yf} \tag{2-25}$$

For Missouri Bridge A6101

$$F_{nc} = 1 \cdot 1 \cdot 0.981 \cdot 70ksi = 68.7ksi$$

where

ϕ_f = the flexural resistance factor, which is one,

R_b = the web load-shedding factor,

R_h = the hybrid reduction factor and

F_{yf}= the yield strength of the critical flange.

The compression flange is the critical flange, and the maximum flange stress due to strength I loading in the flange is given as

$$f_{bu} = \frac{1.25M_{DC1}}{S_{s\_bottom}} + \frac{1.25M_{DC2}}{S_{3n\_bottom}} + \frac{1.5M_{DW}}{S_{3n\_bottom}} + \frac{1.75M_{LL+IM}}{S_{n\_bottom}} \tag{2-26}$$

For Missouri Bridge A6101

$$f_{bu} = \frac{1.25 \cdot 3558k \cdot ft}{1832in^3} + \frac{1.25 \cdot 378k \cdot ft}{2154in^3} + \frac{1.5 \cdot 805k \cdot ft}{2154in^3} + \frac{1.75 \cdot 2663k \cdot ft}{2327in^3} \cdot \frac{12in}{1ft} = 62.5ksi$$

To meet the strength limit state, the following must be met:

$$f_{bu} + \frac{1}{3} f_\ell \leq \phi_f F_{nc} \tag{2-27}$$

where

f_ℓ = the flange lateral bending stress determined as zero, giving

$$f_{bu} = 62.5\ ksi < \phi_f F_{nc} = 68.7\ ksi$$

The strength I design check for the girder is satisfied for both flanges because the section is symmetric.

2.10.4 Service II Limit State Design Check

According to the provisions of Article 6.10.4.2, for permanent deformations, maximum stresses in the flanges should satisfy the following equation:

$$f_f \leq 0.95 R_h F_{yt} \tag{2-28}$$

where

f_f = the stress caused by the Service II load combination.

F_{yt} = the tension flange yield strength.

The stress in the flange is given by

$$f_{f\_bottom} = \frac{M_{DC1}}{S_{s\_bottom}} + \frac{M_{DC2}}{S_{3n\_bottom}} + \frac{M_{DW}}{S_{3n\_bottom}} + \frac{1.3 M_{LL+IM}}{S_{n\_bottom}} \tag{2-29}$$

Missouri Bridge A6101

$$f_{f\_bottom} = \frac{3558k \cdot ft}{1832 in^3} + \frac{378k \cdot ft}{2154 in^3} + \frac{805k \cdot ft}{2154 in^3} + \frac{1.3 \cdot 2663k \cdot ft}{2327 in^3} \cdot \frac{12 in}{1 ft} = 32.1 ksi$$

The service II factored design check for the girder is satisfied because the section is symmetrical and the calculated stress is less than $0.95 R_h F_{yf}$ =68.7 ksi.

2.10.5 Positive Moment Region

Similar to the negative moment region, sample calculations were shown for the design checks of the flexural strength and serviceability limit states using the LRFD design method. These regions act compositely with the concrete deck. The moments used in this analysis are M_{DC1} = 1345 k-ft, M_{DC2} = 168 k-ft, M_{DW} = 352 k-ft and M_{WT+IM} = 3429 k-ft (see Section 2.8). The live-load moment is factored by the lane distribution factor for moment:

$$M_{LL+IM} = M_{WT+IM} \cdot mg$$

For Missouri Bridge A6101

$$M_{LL+IM} = 3429k \cdot ft \cdot 0.646 = 2215k \cdot ft$$

2.10.5.1 Strength I Limit State Flexural Design Check

Article 6.10.7.1 gives the criteria for composite girders in positive bending. Similar to the negative moment regions, the maximum bending strength depends on whether the section is compact or noncompact. Since Missouri Bridge A6101 meets the requirements of a compact section in the positive moment region, according to Article 6.10.6 the maximum strength of the section is

$$M_n = M_p\left(1.07 - 0.7\frac{D_p}{D_t}\right) \tag{2-30}$$

For Missouri Bridge A6101

$$M_n = 9543k \cdot ft\left(1.07 - 0.7\frac{9.82}{70.56}\right) = 9281k \cdot ft$$

M_n = the flexural nominal resistance,

M_p = the plastic moment,

D_p = the distance from the top of the concrete deck to the plastic neutral axis and

D_t = the total depth of the composite section.

To satisfy the Strength I limit state, when f_l is equal to zero, the following must be met:

$$M_u \le \phi_f M_n \tag{2-31}$$

The factored design moment M_u is then given by

$$M_u = 1.25M_{DC1} + 1.25M_{DC2} + 1.5M_{DW} + 1.75M_{LL+IM} \tag{2-32}$$

For Missouri Bridge A6101

$$M_u = 1.25 \cdot 1345k \cdot ft + 1.25 \cdot 168k \cdot ft + 1.5 \cdot 352k \cdot ft + 1.75 \cdot 2215k \cdot ft = 6296k \cdot ft$$

which is less than $\phi_f M_n$ = 9281 k-ft. This section is adequate for strength.

2.10.5.2 Service II Limit State Design Check

According to the provisions of Article 6.10.4.2, for permanent deformations, maximum stresses in the flanges should satisfy the following equations:

$$f_f \leq 0.95 R_h F_{yt} \tag{2-33}$$

where

f_f = the stress caused by the Service II load combination.

F_{yt} = the tension flange yield strength.

The stress in the critical flange is given by

$$f_{f\_bottom} = \frac{M_{DC1}}{S_{s\_bottom}} + \frac{M_{DC2}}{S_{3n\_bottom}} + \frac{M_{DW}}{S_{3n\_bottom}} + \frac{1.3 M_{LL+IM}}{S_{n\_bottom}} \tag{2-34}$$

Missouri Bridge A6101

$$f_{f\_bottom} = \frac{1345k \cdot ft}{999in^3} + \frac{168k \cdot ft}{1333in^3} + \frac{352k \cdot ft}{1333in^3} + \frac{1.3 \cdot 2215k \cdot ft}{1452in^3} \cdot \frac{12in}{1ft} = 44.6ksi$$

The service II factored design check for the girder is satisfied because the section is symmetrical and the calculated stress is less than $0.95 R_h F_{yf}$ =63.4 ksi, where R_h is equal to 0.953.

2.10.5.3 Optional Live-load Deflection Criteria

From the LRFD CONSYS model, the deflection from the live loads used for the optional live-load deflection criteria with impact for Missouri Bridge A6101 (see Section 2.8) is

$$\Delta_{LL+IM} = 2.59\, in$$

To determine the maximum deflection of a girder, the CONSYS deflection value is multiplied by the deflection distribution factor, giving

$$\Delta_{LRFD} = DF_{\Delta} \cdot \Delta_{LL+IM} \tag{2-35}$$

For Missouri Bridge A6101

$$\Delta_{LRFD} = 0.51 \cdot 2.59 = 1.321 in$$

The maximum allowable deflection for a bridge without pedestrian traffic given by AASHTO is

$$\Delta_{ALLOWABLE} = \frac{L}{800} \tag{2-36}$$

For Missouri Bridge A6101

$$\Delta_{ALLOWABLE} = \frac{1654 in}{800} = 2.07 in.$$

where L = span length

2.10.6 Rating Factors

This section will demonstrate the rating factor calculations for Strength I and Service II using Missouri Bridge A6101.

2.10.6.1 Strength Rating Factor Sample Calculations

The strength rating factors are calculated using stresses because the section is non-compact in the negative moment region. The moments previously used are converted to kip-inch in the following equations:

$$RF_{Strength\_Negative} = \frac{S_{bottom}}{1.75 \cdot M_{LL+IM}} \cdot \left(\phi_f F_{nc} - \frac{1.25 M_{DC1}}{S_{s\_bottom}} - \frac{1.25 M_{DC2}}{S_{bottom}} - \frac{1.5 M_{DW}}{S_{bottom}} \right) \qquad (2\text{-}37)$$

For Missouri Bridge A6101

$$RF_{Strength\_Negative} = \frac{1832 in^3}{1.75 \cdot 31{,}956 k \cdot ft} \cdot \left(68.7 - \frac{1.25 \cdot 42{,}696 k \cdot in}{1832 in^3} - \frac{1.25 \cdot 4{,}536 k \cdot in}{1832 in^3} - \frac{1.5 \cdot 9{,}660 k \cdot ft}{1832 in^3} \right)$$

$$RF_{Strength\_Negative} = 0.934$$

The positive rating factor equation, however, is different. Because the positive moment regions strength checks are done with moments instead of stresses, the rating factor must also be calculated using moments instead of stresses. The strength rating factor for the positive section is given by

$$RF_{Strength\_Positive} = \frac{1}{1.75 \cdot M_{LL+IM}} \cdot \left(\phi_f M_n - 1.25 M_{DC1} - 1.25 M_{DC2} - 1.5 M_{DC2} \right) \qquad (2\text{-}38)$$

For Missouri Bridge A6101

$$RF_{Sterngth\_Positive} = \frac{1}{1.75 \cdot 2215 k \cdot ft} \cdot \left(9281 k \cdot ft - 1.25 \cdot 1345 k \cdot ft - 1.25 \cdot 168 k \cdot ft - 1.5 \cdot 352 k \cdot ft \right)$$

$$RF_{Strength\_Positive} = 1.77$$

The bottom section modulus is once again used because the concrete in the composite section takes most of the compressive stresses caused by the loads and raises the neutral axis. This makes the tensile stresses in the steel critical.

2.10.6.2 Service Rating Factor Sample Calculations

The overload rating factors are calculated using stresses for negative and positive sections because overload is a stress requirement. The overload rating factors are calculated by

$$RF_{Service\_Negative} = \frac{S_{bottom}}{1.3M_{LL+IM}} \cdot \left(0.95\phi_f F_{nc} - \frac{M_{DC1}}{S_{s\_bottom}} - \frac{M_{DC2}}{S_{bottom}} - \frac{M_{DW}}{S_{bottom}} \right) \tag{2-39}$$

For Missouri Bridge A6101

$$RF_{Service\_Negative} = \frac{1832in^3}{1.3 \cdot 31{,}956k \cdot ft} \cdot \left(0.95 \cdot 68.7 - \frac{42{,}696k \cdot in}{1832in^3} - \frac{4{,}536k \cdot in}{1832in^3} - \frac{9{,}660k \cdot ft}{1832in^3} \right)$$

$$RF_{Service\_Negative} = 1.77$$

Similarly, the positive moment rating factor is given as

$$RF_{Service\_Positive} = \frac{S_{n\_bottom}}{1.3M_{LL+IM}} \cdot \left(0.95\phi_f F_{nc} - \frac{M_{DC1}}{S_{s\_bottom}} - \frac{M_{DC2}}{S_{3n\_bottom}} - \frac{M_{DW}}{S_{3n\_bottom}} \right) \tag{2-40}$$

For Missouri Bridge A6101

$$RF_{Service\_Positive} = \frac{1452in^3}{1.3 \cdot 26{,}580k \cdot ft} \cdot \left(0.95 \cdot 68.7 - \frac{16{,}140k \cdot in}{999in^3} - \frac{2{,}016k \cdot in}{1333in^3} - \frac{4{,}224k \cdot ft}{1333in^3} \right)$$

$$RF_{Service\_Positive} = 1.51$$

2.10.7 Summary of LRFD Design Requirements

The previous section presented the Strength I, Service II and deflection requirements for the AASHTO LRFD method. It also used Missouri Bridge A6101 to demonstrate the procedure. These design requirements are applied to the 10 bridges in the study set in section 3. The state specific deflection criteria from each of the study states are presented and demonstrated in section 3.

3. STATE LOAD FACTOR DESIGN (LFD) DEFLECTION PRACTICES

3.1 Introduction

This section examines the Load Factor Design (LFD) deflection behavior of the 10 study bridges presented in section 2. AASHTO and specific state deflection criteria are applied to the bridges to compare state practice to AASHTO. Table 3.1 (repeat of Table 2.1) describes the selected state criteria.

Table 3.1 Selected State Criteria for AASHTO LFD Comparisons

				Limits	
State	Factored	Loading	Distribution	w/Ped.	w/o Ped.
AASHTO	No	Truck + I	Equal Distribution	L/1000	L/800
Arizona	No	Truck + I or Lane	Moment Distribution	L/1000	L/800
New Jersey	No	HL93 Truck + I	Moment Distribution	L/1000	L/1000
New York	No	Truck + I or Lane + I	Equal Distribution	L/1000	L/800
Rhode Island	Yes=5/3	Truck + Lane + I	Equal Distribution	L/1100	L/1100
South Dakota	No	Truck + I or Lane + I	Moment Distribution	L/1200	L/1000
Tennesse	No	Truck + I	Equal Distribution	L/1000	L/800

The states were selected based on their conservative (or identical to AASHTO in the case of Tennessee) deflection limits, live loads and distribution factors. Of these states, South Dakota employs the most conservative deflection limit of L/1200 for pedestrian steel bridges (Rhode Island is most conservative for non-pedestrian bridges at L/1100). This conservative limit is 83% of the deflection permitted by AASHTO for live-load deflection of pedestrian bridges, and Rhode Island is 73% of AASHTO for non-pedestrian bridges. Rhode Island is the most conservative in terms of the magnitude of the live-load. Rhode Island uses a factored live load of a HS20 Truck + HS20 Lane + Impact. For some spans, this provides an analysis live-load of more than twice that of the AASHTO HS20 truck + Impact. On the other hand, Tennessee is equal in all aspects to the AASHTO criteria (LRFD Tennessee criteria differ from AASHTO and that is why it is included here). A summary of the defection limits, live loads, and distribution factors for each state are shown in Table 3.1. The AASHTO criteria and limits are provided for comparison.

Table 3.2 is a summary of the 10 selected bridges. There are four one-span, three two-span, two three-span and one four-span bridges. Four of the bridges use HPS in combination with conventional steel in a hybrid design. The bridges range in span length from 54 ft to 200 ft. This variable provides a range of span lengths, which allows different live-loadings to control the deflection calculation. Truck loadings control design on shorter spans while lane loadings can be significantly larger on longer spans. The design lanes range from two to six design lanes. The girder spacing ranges from approximately 5 ft. to a little over 9 ft. Design lanes and girder spacing are variables that determine the amount of live-load that gets applied to each girder line. Three of the bridges carry sidewalks. The bridges are composite in the positive moment regions. For the continuous spans, two are non-composite over the piers and the remaining are composite in the negative moment region.

Table 3.2 Selected Bridges for the AASHTO LFD Comparisons

Bridge and Abbreviation	Number of Spans	Span Lengths (Ft)	Number of Lanes	Number of Girders	Girder Spacing (ft)	Negative Moment Behavior	Sidewalk	Steel Strength
UT Asay Creek 9 (UT)	1	76	3	6	7.9	N/A	no	50 ksi
MA Billerica (MA-B)	1	197	6	9	9.4	N/A	no	50 ksi
WY Little Laramie River (WY-LL)	1	96.4	2	4	7.67	N/A	no	50 ksi
PA Northampton County (PA)	1	123	3	5	9	N/A	no	Hybrid - 50 ksi and 70 ksi
MA Chelmsford (MA-C)	2	161-161	3	6	7.8	Composite	no	Hybrid - 50 ksi and 70 ksi
ID A010 (ID)	2	70.3-70.3	3	7	6.83	Composite	yes	50 ksi
MO A6101 (MO)	2	138-138	3	5	8.76	Non-Composite	no	Hybrid - 50 ksi and 70 ksi
WV Overpass 10462 (WV)	3	54-80-54	4	7	8	Composite	yes	Hybrid - 50 ksi, 70 ksi, 100 ksi
WY Laramie River (WY-LR)	3	67.2-89-67.2	2	6	9.04	Composite	yes	50 ksi
IL F.A.S. Route 860 (IL)	4	81-130-130-81	2	5	5.17	Non-Composite	no	50 ksi

3.2 AASHTO LFD Design Criteria

Chapter 2 presents the AASHTO LFD design requirements and uses Missouri Bridge A6101 to demonstrate the strength, overload and deflection design calculations. Table 3.3 shows the AASHTO LFD results for strength rating factors, overload rating factors, live-load deflections and the AASHTO deflection limit. The rating factors are shown here to demonstrate that the bridges are adequate in terms of safety ($RF_{Strength} > 1$) and service performance ($RF_{Overload} > 1$). Although rating factors are not used in this interim MPC report, they will be used as work continues for the overall research effort. As noticed in Table 3.3, all 10 bridges meet the AASHTO deflection criteria. The AASHTO criteria does not appear to be a detriment to conventional steel or high performance steel bridge design, at least for these ten bridges.

Table 3.3 AASHTO LFD Design Results for the Selected Bridges

BRIDGE	Strength RF_{LFD}	Overload RF_{LFD}	Deflection (in)	Deflection Limit L/800 or L/1000
UT Asay Creek	1.50/+	1.68/+	0.450	1.14
MA Billerica	3.69/+	3.67/+	0.836	2.95
WY Little Laramie River	1.53/+	1.23/+	0.956	1.40
PA Northampton County	3.39/+	1.92/+	0.675	1.85
MA Chelmsford	1.82/+	2.50/+	1.23	2.41
ID A010	1.24/+	1.66/+	0.765	0.843
MO A6101	1.15/-	1.74/+	1.43	2.07
WV Overpass 10462	1.18/+	1.57/+	0.943	1.20
WY Laramie River	1.48/+	1.93/+	0.444	1.07
IL F.A.S. Route 860	1.45/-	2.52/-	1.45	1.94

3.3 State LFD Deflection Criteria

Deflection values for each of the 10 existing study bridges subject to each of the six state criteria were determined by analysis similar to the AASHTO LFD provisions shown in section 2. The six state criteria for Missouri Bridge A6101 will be used as an example.

3.3.1 Arizona State Deflection Criteria for Missouri Bridge A6101

For the loading, Arizona uses the maximum deflection from the unfactored Truck plus Impact or the Lane Loading for a maximum deflection of Δ_{WL} = 1.324 in. The multi-presence reduction does not contribute since Arizona uses AASHTO LFD moment distribution factors (S/5.5) for a deflection distribution factor DF_{Δ} = 1.629. Therefore, the calculated deflection for LFD becomes:

$$\Delta_{LFD} = DF_{\Delta} \cdot \Delta_{WL} \cdot (1+I) = (1.629)(1.324)(1+0) = 2.16in$$

The impact is zero since the lane load controlled deflections, and impact is not applied to the lane load in the Arizona loading. Missouri Bridge A6101 does not carry sidewalks; therefore, the Arizona deflection limit is:

$$\Delta_{ALLOWABLE} = \frac{L}{800} = 2.07in \le \Delta_{LFD} = 2.16in$$

And Missouri Bridge A6101 does not meet the deflection criteria for Arizona.

3.3.2 New Jersey State Deflection Criteria for Missouri Bridge A6101

For the loading, New Jersey uses the maximum deflection from the unfactored AASHTO LRFD HL93 loading, which consists of the Truck plus Impact or the Lane Loading plus 25% of the Truck plus impact yielding a maximum deflection of Δ_{WL} = 1.278 in. The multi-presence reduction does not contribute since New Jersey uses AASHTO LFD moment distribution factors (S/5.5) for a deflection distribution factor DF_{Δ} = 1.629. Since New Jersey uses the AASHTO LRFD loading, the associated impact factor is higher than for LFD. In the equation below, an LRFD effective impact factor is used for the adjustment. Derivation of the effective LRFD impact factor, and an example calculation for Missouri Bridge A6101, is presented in Chapter 5. Therefore, the calculated deflection for LFD becomes:

$$\Delta_{LFD} = DF_{\Delta} \cdot \Delta_{WL} \cdot (1+I_{Eff-LRFD}) = (1.629)(1.278)(1+0.158) = 2.41in$$

Missouri Bridge A6101 does not carry sidewalks; therefore, the New Jersey deflection limit is:

$$\Delta_{ALLOWABLE} = \frac{L}{1000} = 1.654in \le \Delta_{LFD} = 2.41in$$

And Missouri Bridge A6101 does not meet the deflection criteria for New Jersey.

3.3.3 New York State Deflection Criteria for Missouri Bridge A6101

For the loading, New York uses the maximum deflection from the unfactored Truck plus Impact or the Lane Loading plus Impact for a maximum deflection of Δ_{WL} = 1.143 in. The bridge has three design lanes, which yields a multi-presence reduction i = 0.90. With equal distribution of loads to all girders, the deflection distribution factor DF_{Δ} = 1.08. Therefore, the calculated deflection for LFD becomes:

$$\Delta_{LFD} = DF_{\Delta} \cdot \Delta_{WL} \cdot (1+I) = (1.08)(1.143)(1+0.19) = 1.47in$$

Missouri Bridge A6101 does not carry sidewalks; therefore, the New York deflection limit is:

$$\Delta_{ALLOWABLE} = \frac{L}{800} = 2.07in \geq \Delta_{LFD} = 1.47in$$

And Missouri Bridge A6101 does meet the deflection criteria for New York.

3.3.4 Rhode Island State Deflection Criteria for Missouri Bridge A6101

For the loading, Rhode Island uses the maximum deflection from the factored (factor = 5/3) Truck plus Impact plus Lane Loading plus Impact for a maximum deflection of Δ_{WL} = 3.76 in. The bridge has three design lanes, which yields a multi-presence reduction i = 0.90. With equal distribution of loads to all girders, the deflection distribution factor DF_{Δ} = 1.08. Therefore, the calculated deflection for LFD becomes:

$$\Delta_{LFD} = DF_{\Delta} \cdot \Delta_{WL} \cdot (1+I) = (1.08)(3.76)(1+0.19) = 4.83in$$

Missouri Bridge A6101 does not carry sidewalks; therefore, the Rhode Island deflection limit is:

$$\Delta_{ALLOWABLE} = \frac{L}{1100} = 1.50in \leq \Delta_{LFD} = 4.83in$$

And Missouri Bridge A6101 does not meet the deflection criteria for Rhode Island.

3.3.5 South Dakota State Deflection Criteria for Missouri Bridge A6101

For the loading, South Dakota uses the maximum deflection from the unfactored Truck plus Impact or the Lane Loading plus Impact for a maximum deflection of Δ_{WL} = 1.143 in. The multi-presence reduction does not contribute since South Dakota uses moment distribution factors (S/5.5) for a deflection distribution factor DF_{Δ} = 1.629. Therefore, the calculated deflection for LFD becomes:

$$\Delta_{LFD} = DF_{\Delta} \cdot \Delta_{WL} \cdot (1+I) = (1.08)(1.143)(1+0.19) = 2.22in$$

Missouri Bridge A6101 does not carry sidewalks; therefore, the South Dakota deflection limit is:

$$\Delta_{ALLOWABLE} = \frac{L}{1000} = 1.654in \leq \Delta_{LFD} = 2.22in$$

And Missouri Bridge A6101 does not meet the deflection criteria for South Dakota.

3.3.6 Tennessee State Deflection Criteria for Missouri Bridge A6101

For the loading, Tennessee uses the maximum deflection from the unfactored Truck plus Impact for a maximum deflection of Δ_{WL} = 1.112 in. The bridge has three design lanes, which yields a multi-presence reduction i = 0.90. With equal distribution of loads to all girders, the deflection distribution factor DF_{Δ} = 1.08. Therefore, the calculated deflection for LFD becomes:

$$\Delta_{LFD} = DF_{\Delta} \cdot \Delta_{WL} \cdot (1+I) = (1.08)(1.112)(1+0.19) = 1.43in$$

Missouri Bridge A6101 does not carry sidewalks; therefore, the Tennessee deflection limit is:

$$\Delta_{ALLOWABLE} = \frac{L}{800} = 2.07in \geq \Delta_{LFD} = 1.43in$$

And Missouri Bridge A6101 does meet the deflection criteria for Tennessee.

3.3.7 State Deflection Criteria Analysis

Table 3.4 presents the results for the six state's deflection criteria applied to the 10 study bridges. From Table 3.4, it is clear that the live-load deflection criteria for many of the states in this study are conservative. At times, the live-load deflections produced by the state loading criteria are more than triple those produced using AASHTO. The level of conservatism in the calculated deflections used by each state can be quantified and illustrated in Figure 3.1. This figure provides a graphical representation of the calculated state deflection divided by the calculated AASHTO deflection for each bridge.

In Figure 3.1, the AASHTO live-load deflections are represented by the value of 1 for each of the bridges. Therefore, the remaining values represent how many times larger the state calculated deflections are than the AASHTO deflections. The level of conservatism for each state varies for each bridge due to the loading characteristics and distribution factors used to calculate the deflections. Rhode Island uses the largest applied nominal live-load, and it is factored, and in turn produces the largest deflection values, some of which are near four times that of AASHTO. Several states produce deflections around 1.5 to 2 times the AASHTO deflections. This was from using moment distribution factors in place of equal distribution. The remaining states are fairly close to AASHTO with the calculated deflections. The additional lane loading case do not appear to significantly affect the deflections for New York, and Tennessee is the same as AASHTO since the analysis is identical to AASHTO. However, Figure 3.1 represents only the loading characteristics and does not include the effects of the actual deflection limits.

Figure 3.2 illustrates the comparison of the state deflection limits. The deviations from the constant relationships represent bridges that carry pedestrian traffic and the limits change. Rhode Island and South Dakota have more strict deflection limits for bridges carrying sidewalks. Those two states, plus New Jersey, have more strict deflection limits for bridges without sidewalks. The impact of the deflection limits combined with the differences in loading is shown in Figure 3.3.

Figure 3.3 provides information on the overall conservative nature of the state live-load deflection criteria, including loading criteria, distribution factors, and the state deflection limits. Unlike Figure 3.1, Figure 3.3 includes the effects of the state imposed deflection limits. The plot combines the calculated deflection conservatism (State Deflection / AASHTO Deflection) and the conservatism from the more restrictive deflection limits (State Allowable Limit / AASHTO Allowable Limit) using either L/800 or L/1000 as appropriate for the AASHTO limit.

All the states in this study, except Tennessee, impose more restrictive live-load deflection criteria than AASHTO. The larger live loads and the more restrictive limits create conservatism in the state deflection criteria. Rhode Island is 3 to 5.5 times more conservative than the AASHTO LFD specifications due to the factored larger nominal loads and lower allowable deflections. This means that in Rhode Island bridges would need to be 3 to 5.5 times stiffer (much more structural material) to be equivalent in design to the AASHTO specifications. New Jersey, South Dakota and Arizona are approximately 1.5-2.5 times more conservative caused by a mix of using moment distribution factors are more restrictive deflection limits. New Jersey is at the upper end of these three states due to the larger impact factor associated with its use of the LRFD loading scheme. Tennessee is identical to AASHTO in loading and deflection limits. New York is also the same as AASHTO except that New York adds the Lane Load case when calculating the deflection. This causes a slight conservatism for the few bridges where the lane load controls the maximum deflection.

Table 3.4 LFD State Deflection Criteria for Study Bridges

Bridge	ST/AASHTO	Δ (in)	Δ_{limit} (in)	Remarks Comments
Utah	AASHTO	0.451	1.142	
	Arizona	0.789	1.142	
	New Jersey	0.841	0.913	
	New York	0.451	1.142	
	Rhode Island	1.304	0.830	not adequate
	South Dakota	0.788	0.913	
	Tennessee	0.451	1.142	
Billerica, MA	AASHTO	0.773	2.953	
	Arizona	1.560	2.953	
	New Jersey	1.651	2.36	
	New York	1.055	2.953	
	Rhode Island	3.046	2.147	not adequate
	South Dakota	1.803	2.362	
	Tennessee	0.773	2.953	
Little Laramie River, WY	AASHTO	0.956	1.401	
	Arizona	1.333	1.401	
	New Jersey	1.443	1.121	not adequate
	New York	0.956	1.401	
	Rhode Island	2.913	1.019	not adequate
	South Dakota	1.333	1.121	not adequate
	Tennessee	0.956	1.401	
Pennsylvania	AASHTO	0.679	1.845	
	Arizona	1.030	1.845	
	New Jersey	1.140	1.476	
	New York	0.679	1.845	
	Rhode Island	2.223	1.342	not adequate
	South Dakota	1.031	1.476	
	Tennessee	0.679	1.845	
Chelmsford, MA	AASHTO	1.229	2.411	
	Arizona	1.938	2.411	
	New Jersey	2.194	1.929	not adequate
	New York	1.403	2.411	
	Rhode Island	4.385	1.754	not adequate
	South Dakota	2.213	1.929	not adequate
	Tennessee	1.229	2.411	
A010 Idaho	AASHTO	0.766	0.843	
	Arizona	1.233	0.843	not adequate
	New Jersey	1.306	0.843	not adequate
	New York	0.766	0.843	
	Rhode Island	2.175	0.766	not adequate
	South Dakota	1.234	0.703	not adequate
	Tennessee	0.766	0.843	

Table 3.4 LFD State Deflection Criteria for Study Bridges (continued)

Bridge	ST/AASHTO	Δ (in)	Δ_{limit} (in)	Remarks Comments
A6101 Missouri	AASHTO	1.429	2.067	
	Arizona	2.157	2.067	not adequate
	New Jersey	2.410	1.654	not adequate
	New York	1.469	2.067	
	Rhode Island	4.833	1.503	not adequate
	South Dakota	2.216	1.654	not adequate
	Tennessee	1.429	2.067	
Culloden, WV	AASHTO	0.930	0.960	
	Arizona	1.578	0.960	not adequate
	New Jersey	1.667	0.960	not adequate
	New York	0.930	0.960	
	Rhode Island	2.678	0.873	not adequate
	South Dakota	1.578	0.800	not adequate
	Tennessee	0.930	0.960	
Laramie River, WY	AASHTO	0.452	1.068	
	Arizona	1.114	1.068	not adequate
	New Jersey	1.190	1.068	not adequate
	New York	0.452	1.068	
	Rhode Island	1.339	0.971	not adequate
	South Dakota	1.114	0.890	not adequate
	Tennessee	0.452	1.068	
Illinois	AASHTO	1.447	1.943	
	Arizona	1.700	1.943	
	New Jersey	1.900	1.554	not adequate
	New York	1.447	1.943	
	Rhode Island	4.802	1.413	not adequate
	South Dakota	1.701	1.554	not adequate
	Tennessee	1.447	1.943	

3.3.8 State Practice Effect on Economy and Design

Figures 3.1 through 3.3 and Table 3.4 illustrate the conservatism in many state deflection practices. To demonstrate what this means in terms of design and economy for these 10 study bridges, Figure 3.4 quantifies how many times larger the state calculated deflections are than the state allowable deflection. The calculated Rhode Island deflection for the Illinois Bridge would have to be reduced by nearly four times to meet the Rhode Island deflection limit. The steel girder sizes of these bridges would have to be increased significantly to increase the stiffness four times to produce calculated deflections below the state limit. In fact, there are 28 cases where, for these 10 bridges, they do not meet state criteria. In these 28 cases, the states would need to add steel material, weight and cost to meet the state criteria. All the bridges meet the AASHTO requirements (along with Tennessee, which is the same as AASHTO, and New York, which is just slightly conservative). However, none of the remaining states could build all these bridges and meet their state criteria. The problem with this is, besides the additional cost, these 10 bridges are in service and performing well. There are no apparent deficiencies in either user comfort or deformation induced damage. The conclusion that can be drawn is that these conservative states are expending unnecessary materials and costs.

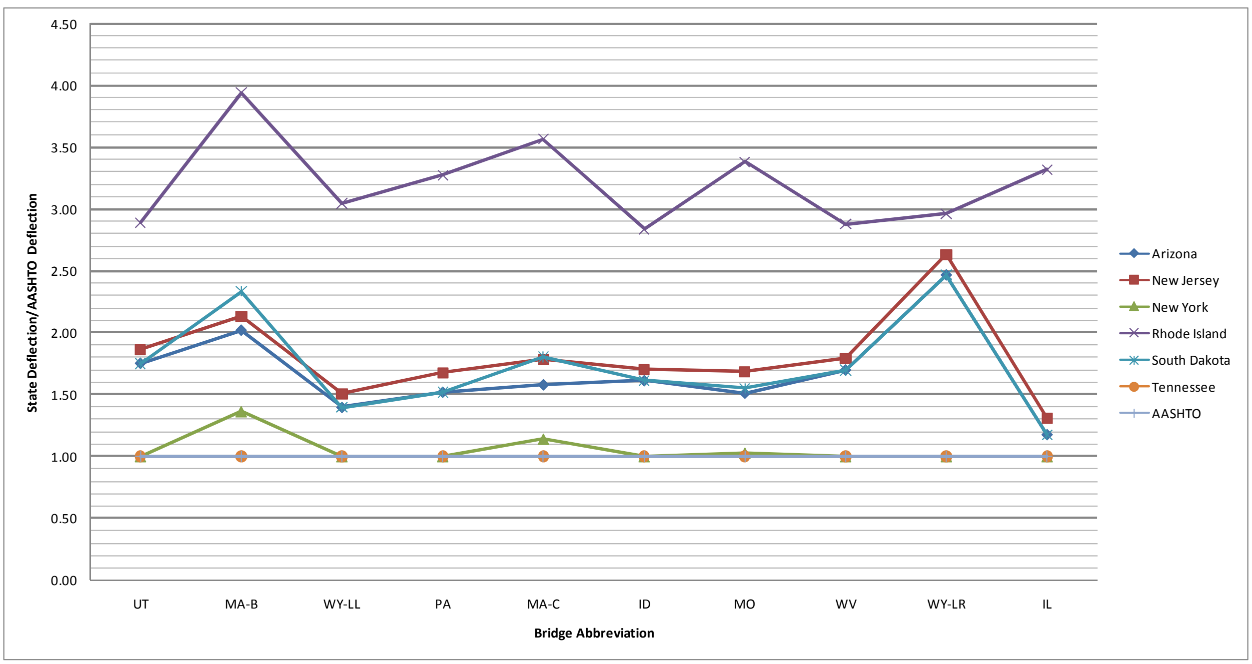


Figure 3.1 Comparison of LFD State Loading Practice

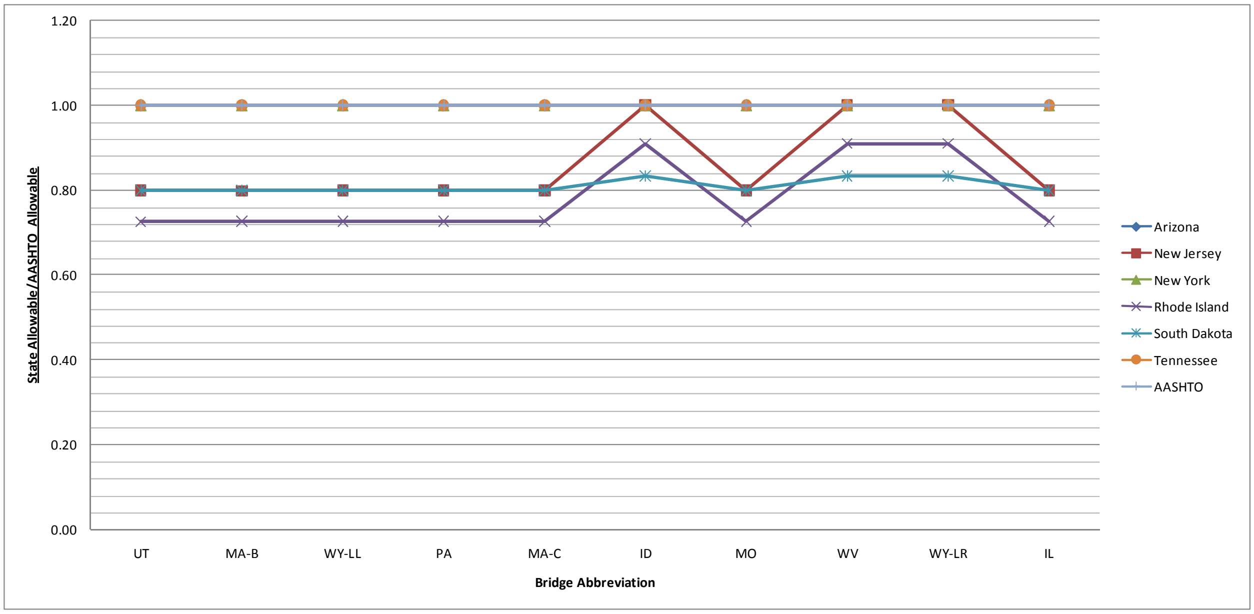


Figure 3.2 Comparison of LFD State Deflection Limit Practice

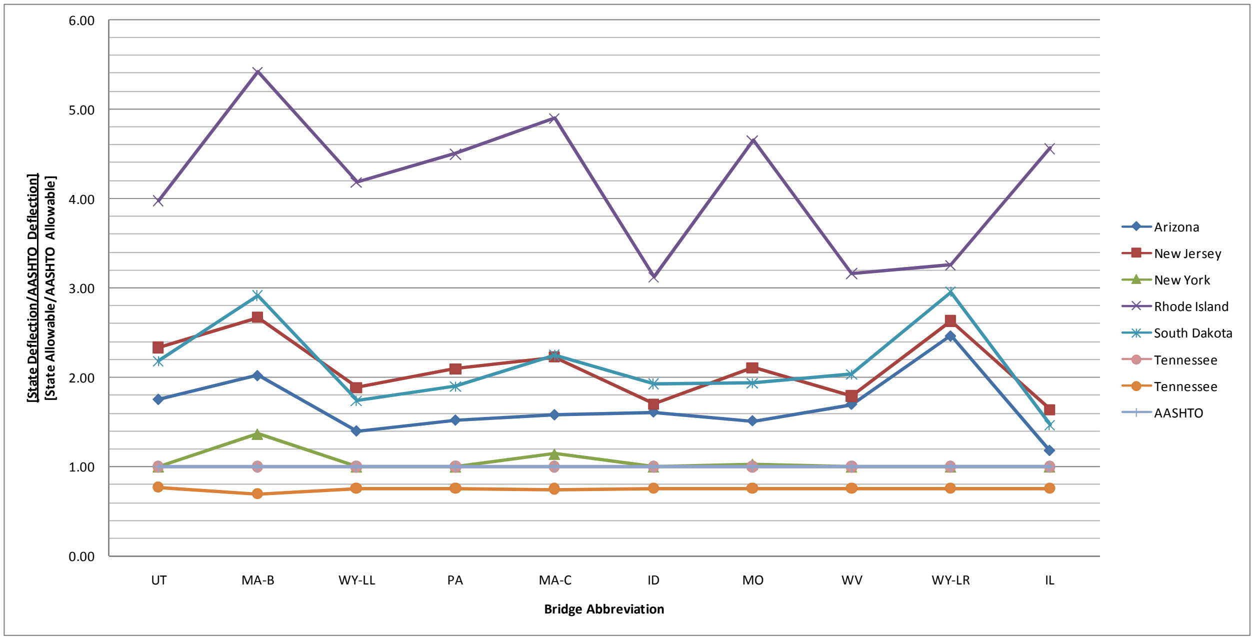


Figure 3.3 Comparison of LFD State Practice Conservatism

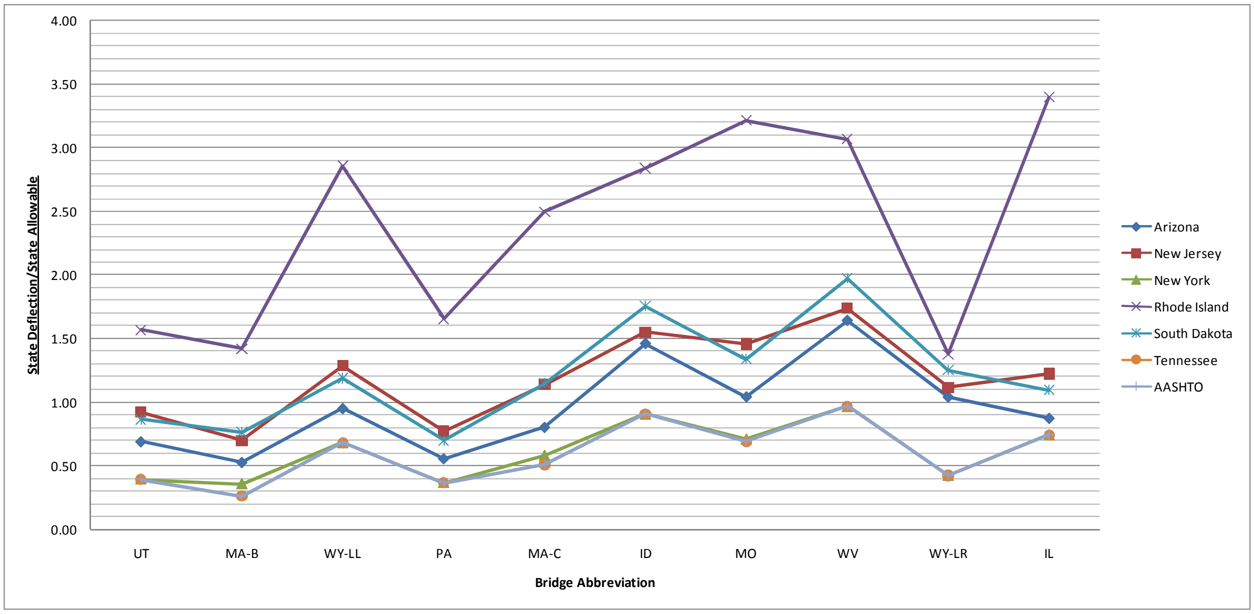


Figure 3.4 Design Effect of LFD State Deflection Practice

4. STATE LOAD & RESISTANCE FACTOR DESIGN (LRFD) DEFLECTION PRACTICES

4.1 Introduction

This chapter examines the Load & Resistance Factor Design (LRFD) deflection behavior of the 10 study bridges presented in Chapter 2. AASHTO and specific state deflection criteria are applied to the bridges to compare state practice to AASHTO. Table 4.1 (repeat of Table 2.2) describes the selected state criteria.

Table 4.1 Selected State Criteria for AASHTO LRFD Comparisons

				Limits	
State	Factored	Loading	Distribution	w/Ped.	w/o Ped.
AASHTO	No	Truck + I or Lane + 25%Truck + Impact	Equal w/ multiple lane reduction	L/1000	L/800
Arizona	Yes=1.75	Turck + I or Lane + 25%Truck + Impact	Equal w/ multiple lane reduction	L/800	L/800
New Jersey	Yes=1.75	Truck + I or Lane + 25%Truck + Impact	Multiple Lane Moment Distribution	L/1000	L/1000
New York	No	Truck + Impact	Multiple Lane Moment Distribution	L/1000	L/800
Rhode Island	Yes=1.75	Lane + Truck + Impact	Equal w/ multiple lane reduction	L/1100	L/1100
South Dakota	No	Truck + I or Lane + 25%Truck + Impact	Equal w/out lane reduction	L/1200	L/1000
Tennesse	No	Truck Only	Equal w/ multiple lane reduction	L/1000	L/800

These states were selected based on their conservative (or unconservative in the case of Tennessee) deflection limits, live loads and distribution factors. Of these states, South Dakota employs the most conservative deflection limit of L/1200 for pedestrian steel bridges (Rhode Island is most conservative for non-pedestrian bridges at L/1100). This conservative limit is 83% of the deflection permitted by AASHTO for live-load deflection of pedestrian bridges, and Rhode Island is 73% of AASHTO for non-pedestrian bridges. Rhode Island is the most conservative in terms of the magnitude of the live-load. Rhode Island uses a factored live load of a Truck+ Impact plus Lane Load. For some spans, this provides an analysis live-load of more than twice that of the AASHTO Truck + Impact or Lane plus 25% Truck + Impact. Arizona and New Jersey also factor the loads, but use the convention Truck + Impact or Lane plus 25% Truck + Impact. These two states apply 1.75 times the AASHTO loads. New Jersey uses moment distribution factors that would increase the 1.75 factor. On the other hand, Tennessee applies only the truck load without impact, which is a load less than the AASHTO criteria. A summary of the defection limits, live-loads, and distribution factors for each state are shown in Table 4.1. The AASHTO criteria and limits are provided for comparison.

Table 4.2 is a summary of the 10 selected bridges. There are four one-span, three two-span, two three-span and one four-span bridges. Four of the bridges use HPS in combination with conventional steel in a hybrid design. The bridges range in span length from 54 ft to 200 ft. This variable provides a range of span lengths, which allows different live-loadings to control the deflection calculation. Truck loadings control design on shorter spans while lane loadings can be significantly larger on longer spans. The design lanes range from two to six design lanes. The girder spacing ranges from approximately 5 ft. to a little over 9 ft. Design lanes and girder spacing are variables that determine the amount of live-load that gets applied to each girder line. Three of the bridges carry sidewalks. All of the bridges are composite in the positive moment regions. For the continuous spans, two are non-composite over the piers, and the remaining are composite in the negative moment region.

Table 4.2 Selected Bridges for the AASHTO LRFD Comparisons

Bridge and Abbreviation	Number of Spans	Span Lengths (Ft)	Number of Lanes	Number of Girders	Girder Spacing (ft)	Negative Moment Behavior	Sidewalk	Steel Strength
UT Asay Creek 9 (UT)	1	76	3	6	7.9	N/A	no	50 ksi
MA Billerica (MA-B)	1	197	6	9	9.4	N/A	no	50 ksi
WY Little Laramie River (WY-LL)	1	96.4	2	4	7.67	N/A	no	50 ksi
PA Northampton County (PA)	1	123	3	5	9	N/A	no	Hybrid - 50 ksi and 70 ksi
MA Chelmsford (MA-C)	2	161-161	3	6	7.8	Composite	no	Hybrid - 50 ksi and 70 ksi
ID A010 (ID)	2	70.3-70.3	3	7	6.83	Composite	yes	50 ksi
MO A6101 (MO)	2	138-138	3	5	8.76	Non-Composite	no	Hybrid - 50 ksi and 70 ksi
WV Overpass 10462 (WV)	3	54-80-54	4	7	8	Composite	yes	Hybrid - 50 ksi, 70 ksi, 100 ksi
WY Laramie River (WY-LR)	3	67.2-89-67.2	2	6	9.04	Composite	yes	50 ksi
IL F.A.S. Route 860 (IL)	4	81-130-130-81	2	5	5.17	Non-Composite	no	50 ksi

4.2 AASHTO LRFD Design Criteria

Section 2 presents the AASHTO LRFD design requirements, and uses Missouri Bridge A6101 to demonstrate the strength, overload and deflection design calculations. Table 4.3 shows the AASHTO LRFD results for Strength I rating factors, Service II rating factors, live-load deflections and the AASHTO deflection limit. The rating factors are shown here to demonstrate that the bridges, except for Missouri Bridge A6101, are adequate in terms of safety ($RF_{Strength\ I} > 1$) and service performance ($RF_{Service\ II} > 1$). Although rating factors are not used in this interim MPC report, they will be used as work continues for the overall research effort. As noticed in Table 4.3, all the 10 bridges meet the AASHTO deflection criteria. The AASHTO criteria does not appear to be a detriment to conventional steel or high performance steel bridge design, at least for these 10 bridges.

Table 4.3 AASHTO LRFD Design Requirements for the Selected Bridges

BRIDGE	Strength RF_{LRFD}	Service RF_{LRFD}	Deflection (in)	Deflection Limit L/800 or L/1000 (in)
UT Asay Creek	1.45/+	1.86/+	0.455	1.14
MA Billerica	3.5/+	3.57/+	0.86	2.95
WY Little Laramie River	1.42/+	1.21/+	1.04	1.45
PA Northampton County	2.15/+	1.82/+	0.709	1.85
MA Chelmsford	2.07/+	2.27/+	1.25	2.41
ID A010	1.67/+	1.72/+	0.634	0.843
MO A6101	0.934/-	1.51/-	1.32	2.07
WV Overpass 10462	1.53/-	2.04/+	0.698	0.96
WY Laramie River	1.06/-	1.52/-	0.39	1.07
IL F.A.S. Route 860	0.956/-	1.38/-	1.31	1.94

4.3 State LRFD Deflection Criteria

Deflection values for each of the 10 existing study bridges subject to each of the six state criteria were determined by analysis similar to the AASHTO LRFD provisions shown in section 2. The six state criteria for Missouri Bridge A6101 will be used as an example,.

4.3.1 Arizona State Deflection Criteria for Missouri Bridge A6101

For the loading, Arizona uses the maximum deflection from the factored (factor = 1.75) Truck plus Impact or the Lane +25%Truck plus Impact Loading for a maximum deflection of Δ_{WT+I} = 4.533 in. The bridge has three design lanes, which yields a multi-presence reduction m = 0.85. With equal distribution of loads to all girders, the deflection distribution factor DF_{Δ} = 0.51. Therefore, the calculated deflection for LRFD becomes:

$$\Delta_{LRFD} = DF_{\Delta} \cdot \Delta_{WT+I} \cdot = (0.51)(4.533) = 2.31 in$$

Missouri Bridge does not carry sidewalks; therefore, the Arizona deflection limit is:

$$\Delta_{ALLOWABLE} = \frac{L}{800} = 2.07 in \le \Delta_{LRFD} = 2.31 in$$

And Missouri Bridge A6101 does not meet the deflection criteria for Arizona.

4.3.2 New Jersey State Deflection Criteria for Missouri Bridge A6101

For the loading, New Jersey uses the maximum deflection from the factored (factor = 1.75) Truck plus Impact or the Lane +25%Truck plus Impact Loading for a maximum deflection of Δ_{WT+I} = 4.533 in. The multi-presence reduction does not contribute since New Jersey uses AASHTO LRFD moment distribution factors for a deflection distribution factor DF_{Δ} = 0.646. Therefore, the calculated deflection for LRFD becomes:

$$\Delta_{LRFD} = DF_{\Delta} \cdot \Delta_{WT+I} \cdot = (0.646)(4.533) = 2.93 in$$

Missouri Bridge does not carry sidewalks; therefore, the Arizona deflection limit is:

$$\Delta_{ALLOWABLE} = \frac{L}{1000} = 1.654 in \le \Delta_{LRFD} = 2.93 in$$

And Missouri Bridge A6101 does not meet the deflection criteria for New Jersey.

4.3.3 New York State Deflection Criteria for Missouri Bridge A6101

For the loading, New York uses the maximum deflection from the unfactored Truck plus Impact for a maximum deflection of Δ_{WT+I} = 2.59 in. The multi-presence reduction does not contribute since New York uses AASHTO LRFD moment distribution factors for a deflection distribution factor DF_{Δ} = 0.646. Therefore, the calculated deflection for LRFD becomes:

$$\Delta_{LRFD} = DF_{\Delta} \cdot \Delta_{WT+I} \cdot = (0.646)(2.59) = 1.67 in$$

Missouri Bridge does not carry sidewalks; therefore, the New York deflection limit is:

$$\Delta_{ALLOWABLE} = \frac{L}{800} = 2.07in \geq \Delta_{LRFD} = 1.67in$$

And Missouri Bridge A6101 does meet the deflection criteria for New York.

4.3.4 Rhode Island State Deflection Criteria for Missouri Bridge A6101

For the loading, Rhode Island uses the maximum deflection from the factored (factor = 1.75) Lane plus plus Truck plus Impact for a maximum deflection of Δ_{WT+I} = 7.135 in. The bridge has three design lanes, which yields a multi-presence reduction m = 0.85. With equal distribution of loads to all girders, the deflection distribution factor DF_{Δ} = 0.51. Therefore, the calculated deflection for LRFD becomes:

$$\Delta_{LRFD} = DF_{\Delta} \cdot \Delta_{WT+I} \cdot = (0.51)(7.135) = 3.64in$$

Missouri Bridge does not carry sidewalks; therefore, the Rhode Island deflection limit is:

$$\Delta_{ALLOWABLE} = \frac{L}{1100} = 1.503in \leq \Delta_{LRFD} = 3.64in$$

And Missouri Bridge A6101 does not meet the deflection criteria for Rhode Island.

4.3.5 South Dakota State Deflection Criteria for Missouri Bridge A6101

For the loading, South Dakota uses the maximum deflection from the unfactored Truck plus Impact or the Lane +25%Truck plus Impact Loading for a maximum deflection of Δ_{WT+I} = 2.59 in. South Dakota does not apply the multi-presence reduction. With equal distribution of loads to all girders, the deflection distribution factor DF_{Δ} = 0.60. Therefore, the calculated deflection for LRFD becomes:

$$\Delta_{LRFD} = DF_{\Delta} \cdot \Delta_{WT+I} \cdot = (0.60)(2.59) = 1.55in$$

Missouri Bridge does not carry sidewalks; therefore, the South Dakota deflection limit is:

$$\Delta_{ALLOWABLE} = \frac{L}{1000} = 1.654in \geq \Delta_{LRFD} = 1.55in$$

And Missouri Bridge A6101 does meet the deflection criteria for South Dakota.

4.3.6 Tennessee State Deflection Criteria for Missouri Bridge A6101

For the loading, Tennessee uses the maximum deflection from the unfactored Truck without Impact for a maximum deflection of Δ_{WT+I} = 1.95 in. The bridge has 3 design lanes which yields a multi-presence reduction m = 0.85. With equal distribution of loads to all girders, the deflection distribution factor DF_{Δ} = 0.51. Therefore, the calculated deflection for LRFD becomes:

$$\Delta_{LRFD} = DF_{\Delta} \cdot \Delta_{WT+I} \cdot = (0.51)(1.95) = 1.00in$$

Missouri Bridge does not carry sidewalks; therefore, the Tennessee deflection limit is:

$$\Delta_{ALLOWABLE} = \frac{L}{800} = 2.07in \geq \Delta_{LRFD} = 1.00in$$

And Missouri Bridge A6101 does meet the deflection criteria for Tennessee.

4.3.7 State Deflection Criteria Analysis

Table 4.4 presents the results for the six states' deflection criteria applied to the 10 study bridges. From Table 4.4, it is clear that the live-load deflection criteria for many of the states in this study are conservative. At times the live-load deflections produced by the state loading criteria are more than triple those produced using AASHTO. The level of conservatism in the calculated deflections used by each state can be quantified and illustrated in Figure 4.1. This figure provides a graphical representation of the calculated state deflection divided by the calculated AASHTO deflection for each bridge.

In Figure 4.1, the AASHTO live-load deflections are represented by the value of 1 for each of the bridges. Therefore, the remaining values represent how many times larger the state calculated deflections are than the AASHTO deflections. The level of conservatism for each state varies for each bridge due to the loading characteristics and distribution factors used to calculate the deflections. Rhode Island and New Jersey use the largest applied nominal live-load, and it is factored, and in turn produces the largest deflection values, some of which are near 3.5 times that of AASHTO. New Jersey does not have as much nominal load, but it uses moment distribution factors that increase the deflections. Arizona factors the load, but, other than that, its process is the same as AASHTO so that its deflections are 1.75 times that of AASHTO. New York has a lower nominal loading, but it uses moment distribution factors that make its deflections overall conservative. South Dakota is conservative due to not applying the multiple lane reduction. Tennessee is actually less conservative than AASHTO. Its process is the same except that the nominal load is only the truck without even the impact. Tennessee deflections are approximately 75% of the AASHTO deflections. However, Figure 4.1 represents only the loading characteristics and does not include the effects of the actual deflection limits.

Figure 4.2 illustrates the comparison of the state deflection limits. The deviations from the constant relationships represent bridges that carry pedestrian traffic and the limits change. Rhode Island and South Dakota have more strict deflection limits for bridges carrying sidewalks. Arizona actually has less restrictive limits (L/800) than AASHTO (L/1000) for bridges with sidewalks. Those two states plus New Jersey have more strict deflection limits for bridges without sidewalks. The impact of the deflection limits combined with the differences in loading is shown in Figure 4.3.

Figure 4.3 provides information on the overall conservative nature of the state live-load deflection criteria, including loading criteria, distribution factors, and the state deflection limits. Unlike Figure 4.1, Figure 4.3 includes the effects of the state imposed deflection limits. The plot combines the calculated deflection conservatism (State Deflection / AASHTO Deflection) and the conservatism from the more restrictive deflection limits (State Allowable Limit / AASHTO Allowable Limit) using either L/800 or L/1000 as appropriate for the AASHTO limit.

All the states in this study, except Tennessee, impose more restrictive live-load deflection criteria than AASHTO. The larger live loads and the more restrictive limits create conservatism in the state deflection criteria. Rhode Island is 2.5 to 4.5 times more conservative than the AASHTO LRFD specifications due to the factored larger nominal loads and lower allowable deflections. This means that in Rhode Island bridges would need to be 2.5 to 4.5 times stiffer (much more structural material) to be equivalent in design to the AASHTO specifications. New Jersey is not much better due to the factored loads and moment distribution factors. South Dakota, New York and Arizona are approximately 1.5 times more conservative caused by a mix of using moment distribution factors and more restrictive deflection limits. Tennessee is actually less conservative than AASHTO due to the lesser loading and and all other aspects the same as AASHTO.

Table 4.4 LRFD State Deflection Criteria for Study Bridges

Bridge	ST/AASHTO	Δ (in)	Δ_{limit} (in)	Remarks Comments
Utah	AASHTO	0.445	1.142	
	Arizona	0.796	1.142	
	New Jersey	1.215	0.913	not adequate
	New York	0.694	1.142	
	Rhode Island	1.071	0.830	not adequate
	South Dakota	0.535	0.913	
	Tennessee	0.341	1.142	
Billerica, MA	AASHTO	0.836	2.953	
	Arizona	1.464	2.953	
	New Jersey	2.347	2.362	
	New York	1.237	2.953	
	Rhode Island	2.750	2.147	not adequate
	South Dakota	1.287	2.362	
	Tennessee	0.581	2.953	
Little Laramie River, WY	AASHTO	1.035	1.446	
	Arizona	1.811	1.446	not adequate
	New Jersey	2.163	1.157	not adequate
	New York	1.236	1.446	
	Rhode Island	2.58	1.052	not adequate
	South Dakota	1.035	1.157	
	Tennessee	0.78	1.446	
Pennsylvania	AASHTO	0.709	1.845	
	Arizona	1.241	1.845	
	New Jersey	1.664	1.476	not adequate
	New York	0.954	1.845	
	Rhode Island	1.905	1.342	not adequate
	South Dakota	0.834	1.476	
	Tennessee	0.535	1.845	
Chelmsford, MA	AASHTO	1.245	2.411	
	Arizona	2.179	2.411	
	New Jersey	3.092	1.929	not adequate
	New York	1.767	2.411	
	Rhode Island	3.629	1.754	not adequate
	South Dakota	1.465	1.929	
	Tennessee	0.935	2.411	
A010 Idaho	AASHTO	0.634	0.843	
	Arizona	1.109	1.054	not adequate
	New Jersey	1.857	0.843	not adequate
	New York	1.061	0.843	not adequate
	Rhode Island	1.462	0.766	not adequate
	South Dakota	0.746	0.703	not adequate
	Tennessee	0.477	0.843	

Table 4.4 LRFD State Deflection Criteria for Study Bridges (continued)

Bridge	ST/AASHTO	Δ (in)	Δ_{limit} (in)	Remarks Comments
A6101 Missouri	AASHTO	1.234	2.067	
	Arizona	2.312	2.067	not adequate
	New Jersey	2.928	1.654	not adequate
	New York	1.673	2.067	
	Rhode Island	3.639	1.503	not adequate
	South Dakota	1.554	1.654	
	Tennessee	0.995	2.067	
Culloden, WV	AASHTO	0.698	0.960	
	Arizona	1.222	1.200	not adequate
	New Jersey	2.092	0.690	not adequate
	New York	1.196	0.960	not adequate
	Rhode Island	1.635	0.873	not adequate
	South Dakota	1.074	0.800	not adequate
	Tennessee	0.524	0.960	
Laramie River, WY	AASHTO	0.390	1.068	
	Arizona	0.682	1.335	
	New Jersey	1.364	1.068	not adequate
	New York	0.779	1.068	
	Rhode Island	0.940	0.971	
	South Dakota	0.390	0.890	
	Tennessee	0.294	1.068	
Illinois	AASHTO	1.312	1.943	
	Arizona	2.296	1.943	not adequate
	New Jersey	2.572	1.554	not adequate
	New York	1.469	1.943	
	Rhode Island	3.543	1.413	not adequate
	South Dakota	1.312	1.554	
	Tennessee	0.988	1.943	

4.3.8 State Practice Effect on Economy and Design

Figures 4.1 through 4.3 and Table 4.4 illustrate the conservatism in many state deflection practices. To demonstrate what this means in terms of design and economy for these ten study bridges, Figure 4.4 quantifies how many times larger the state calculated deflections are than the state allowable deflection. The calculated Rhode Island deflection for the Illinois Bridge would have to be reduced by 2.5 times to meet the Rhode Island deflection limit. The steel girder sizes of these bridges would have to be increased significantly to increase the stiffness 2.5 times to produce calculated deflections below the state limit. In fact, there are 27 cases where, for these 10 bridges, they do not meet state criteria. In these 27 cases, the states would need to add steel material, weight and cost to meet the state criteria. All the bridges meet the AASHTO requirements (along with Tennessee, which is less conservative than AASHTO). However, none of the remaining states could build all these bridges and meet their state criteria. The problem with this is, besides the additional cost, these 10 bridges are in service and performing well. There are no apparent deficiencies in either user comfort or deformation-induced damage. The conclusion that can be drawn is that these conservative states are expending unnecessary materials and costs. Another point of interest is why Arizona and New Jersey went from unfactored loading in their LFD criteria to factored loading in their LRFD criteria. Hopefully through dissemination of this research and conversations with the states, this problem can be rectified.

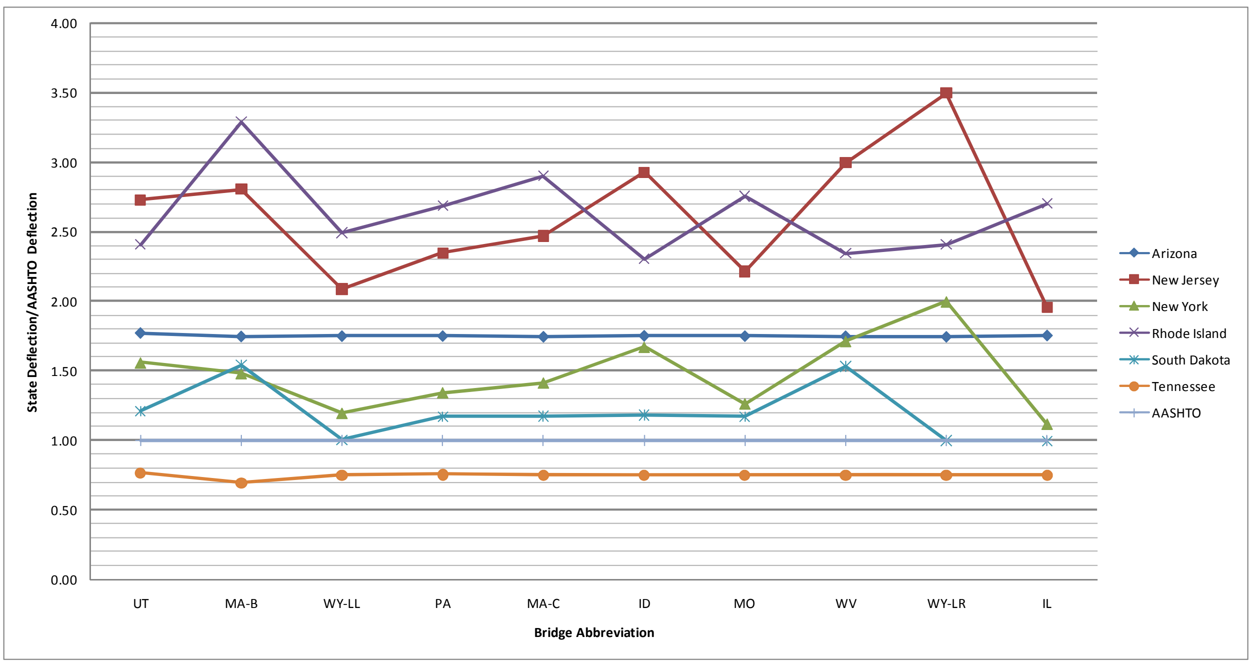


Figure 4.1 Comparison of LRFD State Loading Practice

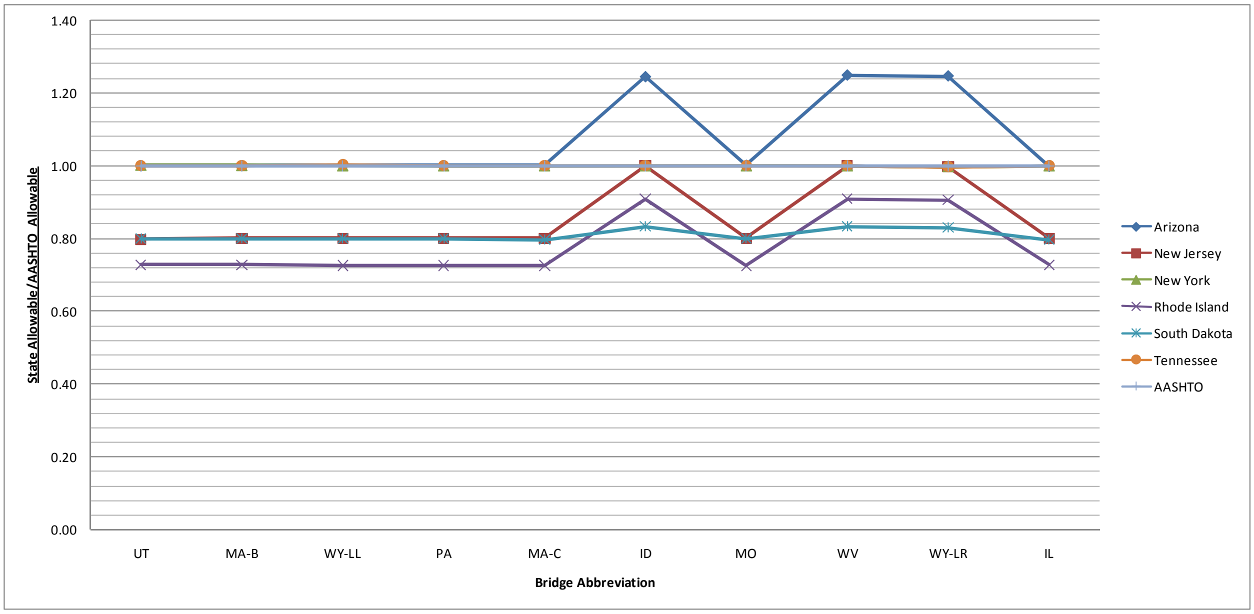


Figure 4.2 Comparison of LRFD Deflection Limit Practice

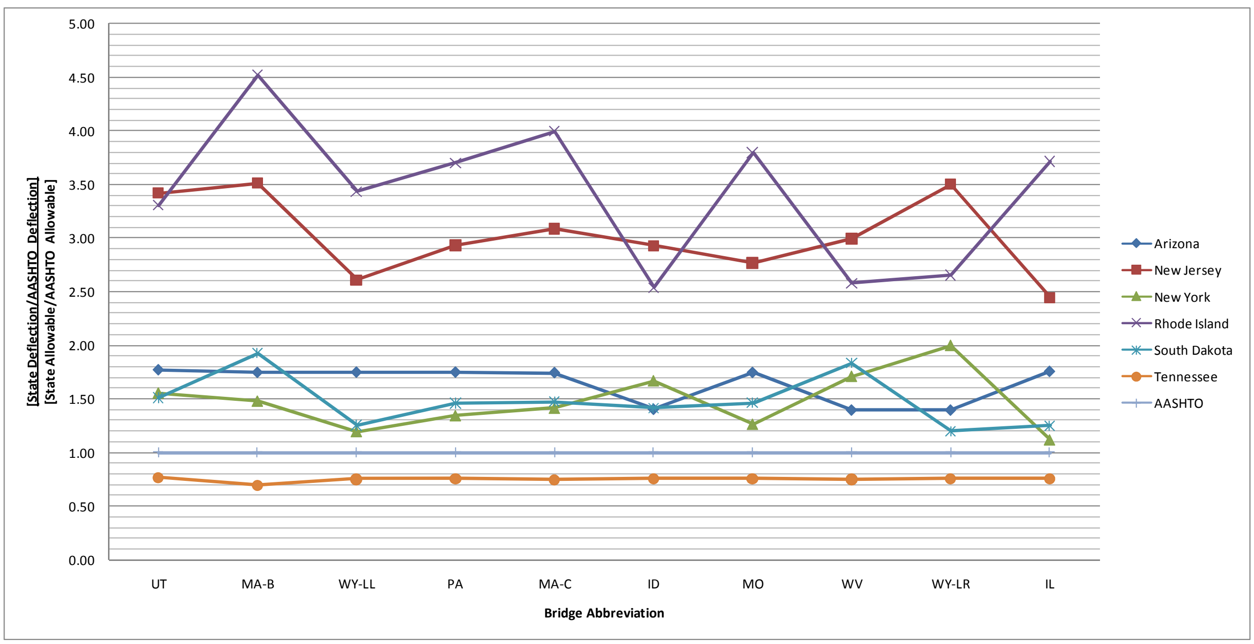


Figure 4.3 Comparison of LRFD State Practice Conservatism

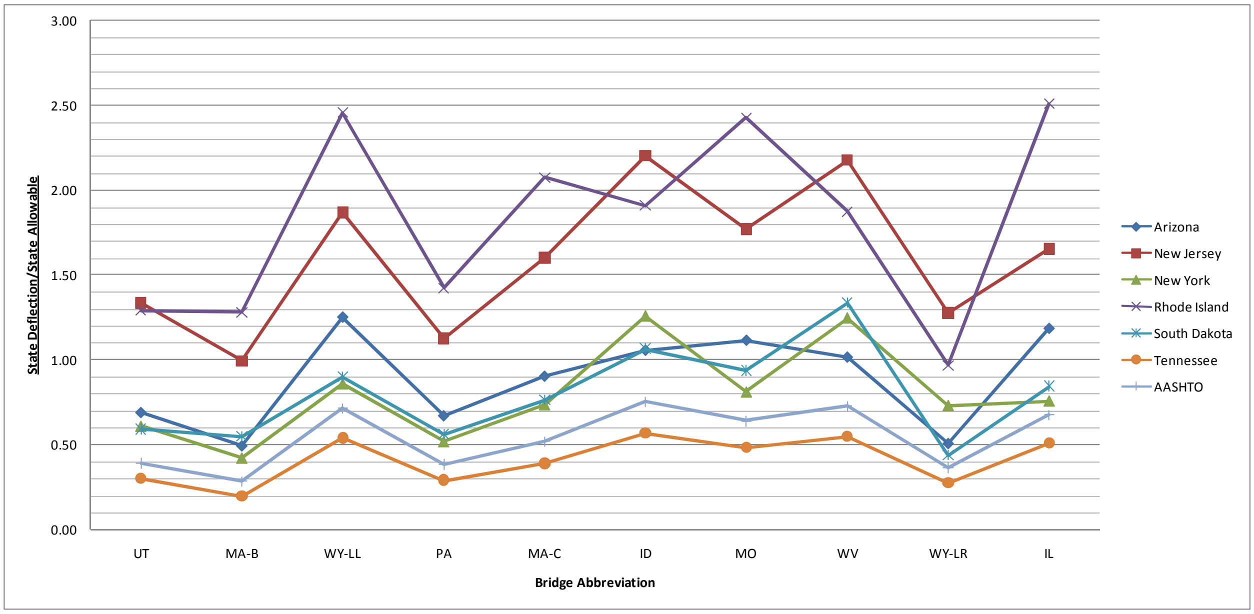


Figure 4.4 Design Effect of LRFD State Deflection Practice

5. LRFD DEFLECTION CRITERIA VS. LFD DEFLECTION CRITERIA

5.1 Introduction

To anticipate the impact with the adoption of LRFD specifications, it is necessary to compare LRFD to LFD deflection criteria directly. To develop a method of comparison, it is imperative to understand where the two specifications differ, and then derive a method where the two specifications can be quantitatively compared both overall and categorically. This section will detail the differences between AASHTO LRFD and AASHTO LFD deflection criteria, and the following section will derive a method used to quantitatively compare the two.

The AASHTO LFD method differs from the AASHTO LRFD method in regards to design load application, multiple presence factors, calculation and application of impact factors, and in longitudinal analysis techniques.

5.1.1 LFD vs. LRFD Design Load Application

The AASHTO LFD and AASHTO LRFD methods differ in the application of loads to calculate deflections. The LFD method uses a single load case of a HS20 Truck plus Impact, whereas the LRFD method uses two load cases, which consist of either a HS20 Truck plus Impact or a quarter of an HS20 truck plus impact plus the lane without impact. If the Truck plus Impact load controls LRFD design, then the applied deflection loads without impact are the same for the LFD and LRFD method. However, if the 25% Truck plus Impact plus lane loading controls the LRFD design, then a more severe loading will be applied by the LRFD method than the truck plus impact loading applied by the LFD method. Also, while LRFD applies a whole truck to a girder line in design, LFD applies a wheel line load to a girder line in design. However, this difference cancels out when the deflection distribution factors are applied since LFD is twice the value for LRFD (2·wheel line = whole truck).

5.1.2 Multiple Presence Factors

The AASHTO LFD and AASHTO LRFD use different multiple presence factors for their respective methods. Table 5.1 shows a comparison of the multiple presence factors.

Table 5.1 Multiple Presence Factors

Lanes	LFD	LRFD
1	1	1.2
2	0.9	1
3	0.75	0.85
4 or more	0.75	0.65

Based on this difference alone, multiple presence factors will increase deflections in bridges designed with LRFD instead of LFD with one lane; the multiple presence factors would have no influence for bridges with two lanes; and bridges with three or more lanes will have increases in deflections if designed with LFD versus LRFD.

5.1.3 Calculation and Application of Impact Factors

AASHTO LRFD differs from AASHTO LFD in the calculation and application of impact factors. When using LFD, the designer calculates an impact factor based on span length as previously demonstrated in Section 2.9.2. This impact factor is applied to both truck and lane loads. In LRFD, the designer uses a constant impact factor with a value of 0.33 and applies this impact factor to truck loads only. Impact factors do not apply to lane loads in LRFD.

5.1.4 Longitudinal Analysis Techniques

AASHTO LRFD utilizes a prismatic analysis for composite bridges versus AASHTO LFD's non-prismatic analysis (Section 2.8). In essence, although it is the same bridge, two different structures are modeled. One structure, prismatic, is analyzed in LRFD while a different structure, non-prismatic, is analyzed in LFD. The difference in the analyzed structures stem from how the concrete deck is treated. In LFD, the concrete is assumed to have zero tensile strength, and the deck is only accounted for in areas with positive moment. If the bridge is composite over the piers, the steel rebar in the assumed cracked concrete can be used in stiffness calculations. LRFD assumes that the strains in the deck are small, below the strain required to crack the concrete, and the deck is considered in both positive and negative moment regions. This results in LFD and LRFD cross sections of equal stiffness in positive moment regions, and a larger cross section stiffness in negative moment regions for LRFD. In single span structures with no negative moment regions, no difference exists. In multi-span bridges with a negative moment region, the same bridge, assuming all other factors constant, designed with LRFD will have a lower deflection due to the stiffer bridge than if it was designed with LFD.

5.2 Comparing LRFD vs. LFD

To meaningfully compare AASHTO LRFD to AASHTO LFD, a method must be derived so the two can be compared both as a whole and per category of influence quantitatively. This section steps through the derivation of this method and then shows a design example using Missouri Bridge A6101.

5.2.1 Equation for Calculating LFD Deflections

To calculate the maximum LFD deflection of a girder, the wheel line output obtained from CONSYS™, $\Delta_{LRFD\text{-}WL\text{-}NP}$, is placed in Equation 5-1. Equation 5-1 multiplies the wheel line deflection obtained from the CONSYS™ LFD model by the summation of "1" plus the LFD impact factor. This accounts for the impact of the load on the bridge. This product is then multiplied by the ratio of design lanes to girders; the ratio of design lanes to girders serves as the LFD deflection distribution factor. The factor of "2" converts the deflection from a wheel line deflection to a whole truck deflection, with two wheel lines per whole truck. This product is then multiplied by the LFD multi presence factor i.

$$\Delta_{LFD-NP} = (\Delta_{LFD-WL-NP}) \cdot (1 + I_{LFD}) \cdot \left(\frac{2n_{designlanes}}{n_{girder}} \right) \cdot i \qquad (5\text{-}1)$$

where:
$\Delta_{LFD\text{-}NP}$ = maximum LFD design deflection of girder in a non-prismatic structure,
$\Delta_{LFD\text{-}WL\text{-}NP}$ = LFD wheel line deflection from one wheel line on a single girder of a non-prismatic structure, value obtained from CONSYS™ LFD bridge models, no impact, no distribution,
I_{LFD} = LFD impact factor,

$n_{designlane}$ = number of design lanes located on bridge structure,
n_{girder} = number of girders in bridge structure and
i = the LFD multi-presence factor from AASHTO LFD Article 3.12.1.

5.2.2 Equation for Calculating LRFD Deflections

The method for calculating the maximum LRFD deflection is similar to the method used for calculating the maximum LFD deflection. Equation (5-2) shown below calculates the maximum LRFD deflection. The whole truck live load plus impact deflection is multiplied by "1" plus the effective impact factor. This product is then multiplied by the distribution factor, which consists of the ratio of design lanes to girders. This product is then multiplied by the multiple presence factor m.

$$\Delta_{LRFD-P} = (\Delta_{LRFD-WT-P}) \cdot (1 + I_{LRFD-EFF}) \cdot \left(\frac{n_{designlanes}}{n_{girder}} \right) \cdot m \qquad (5\text{-}2)$$

where:
Δ_{LRFD-P} = maximum LRFD deflection of girder in a prismatic structure,
$\Delta_{LRFD-WT-P}$ =LRFD deflection of whole truck on a single girder in a prismatic structure, obtained from CONSYS™ LRFD bridge models, no impact, no distribution factor,
I_{LRFD_EFF} = effective impact factor and
m = LRFD multiple presence factor from AASHTO LRFD Table 3.6.1.1.2-1.

Note the use of an effective impact factor in Equation (5-2) instead of the constant 0.33 impact factor. Since the impact factor applies only to truck loads in LRFD, it is not mathematically correct to factor the live-load deflection by the impact factor when the quarter truck plus lane plus impact loading controls. When designing or analyzing a bridge, the designer applies the impact factor directly to the truck load in the computer analysis program, such as CONSYS™. This does not allow for a direct comparison of the different impact factors; therefore, an effective impact factor is calculated using Equation (5-3) below. This enables the direct comparison to be made between LFD and LRFD.

$$I_{LRFD-EFF} = \frac{M_{LL+IM}}{M_{LL}} - 1 \qquad (5\text{-}3)$$

where:
$I_{LRFD-EFF}$ = effective LRFD impact factor,
M_{LL+IM} = positive maximum live load plus impact LRFD moment, obtained from CONSYS™ and
M_{LL} = positive maximum live-load LRFD moment with no impact factor, obtained from CONSYS™.

5.2.3 Deriving LRFD/LFD Deflection Ratio

To compare LRFD to LFD in a meaningful manner, it is necessary to compare equivalent values. This is a challenge because LRFD uses whole truck loads and prismatic design, while LFD uses wheel line loads and non-prismatic design. Therefore, algebraic operations must be performed so the LRFD and LFD terms can be compared.

The first step is to convert the LFD wheel line deflections, $\Delta_{LFD\text{-}WL\text{-}NP}$, to an LFD whole truck deflection, $\Delta_{LFD\text{-}WT\text{-}NP}$. This is done by factoring the two term from the LFD deflection distribution factor and placing it as a coefficient to the LFD wheel line deflection. This is shown below in Equation 5-1a.

$$\Delta_{LFD-NP} = (2\Delta_{LFD-WL-NP})\cdot(1+I_{LFD})\cdot\left(\frac{2n_{designlanes}}{n_{girder}}\right)\cdot i \tag{5-4}$$

Since a whole truck loading consists of two wheel line loadings, Equation 5-5 is substituted into Equation 5-4 to derive Equation 5-6.

$$\Delta_{LFD-WT-NP} = (2\Delta_{LFD-WL-NP}) \tag{5-5}$$

where: $\Delta_{LFD\text{-}WT\text{-}NP}$ = LFD deflection from a whole truck on a single girder, no impact factor, no distribution factor

Equation (5-4) becomes:

$$\Delta_{LFD-NP} = (\Delta_{LFD-WT-NP})\cdot(1+I_{LFD})\cdot\left(\frac{n_{designlanes}}{n_{girder}}\right)\cdot i \tag{5-6}$$

Now that the equation for maximum LFD deflection in a girder is in terms of whole truck loading, Equation 5-2 is divided by Equation 5-6, which produces Equation 5-7.

$$\frac{\left[\Delta_{LRFD-P} = (\Delta_{LRFD-WT-P})\cdot(1+I_{LRFD-EFF})\cdot\left(\frac{n_{designlanes}}{n_{girder}}\right)\cdot m\right]}{\left[\Delta_{LFD-NP} = (\Delta_{LFD-WT-NP})\cdot(1+I_{LFD})\cdot\left(\frac{n_{designlanes}}{n_{girder}}\right)\cdot i\right]} \tag{5-7}$$

The distribution factors reduce to a factor of "1", and the impact factors and the multiple presence factors can be isolated into their own respective terms. This is shown in Equation 5-8.

$$\left[\frac{\Delta_{LRFD-P}}{\Delta_{LFD-NP}}\right] = \left[\frac{(1+I_{LRFD-EFF})}{(1+I_{LFD})}\right]\cdot\left[\frac{m}{i}\right]\cdot\left[\frac{\Delta_{LRFD-WT-P}}{\Delta_{LFD-WT-NP}}\right]\cdot\frac{\left(\frac{\cancel{n}_{designlane}}{\cancel{n}_{girder}}\right)}{\left(\frac{\cancel{n}_{designlane}}{\cancel{n}_{girder}}\right)} \tag{5-8}$$

Equation 5-8 simplifies into Equation 5-9.

$$\left[\frac{\Delta_{LRFD-P}}{\Delta_{LFD-NP}}\right] = \left[\frac{(1+I_{LRFD-EFF})}{(1+I_{LFD})}\right]\cdot\left[\frac{m}{i}\right]\cdot\left[\frac{\Delta_{LRFD-WT-P}}{\Delta_{LFD-WT-NP}}\right] \tag{5-9}$$

Equation 5-9 is still not in a form to qualify the differences into the four differences described in Section 5.1. While the first term covers the difference in impact factors, and the second term covers the difference in multiple presence factors, the third term is comparing two different structures, one prismatic the other non-prismatic and two different loading methods, LFD and LRFD at the same time. The third term needs to be converted into two terms in which one term compares the LFD and LRFD loadings of the same structure, and the second term comparing a prismatic structure to a non-prismatic structure of the same loading method. This is accomplished my multiplying the right side of Equation 5-9 by a factor the LRFD deflection of a whole truck in a non-prismatic structure divide by itself. This is shown in Equation 5-10.

$$\left[\frac{\Delta_{LRFD-P}}{\Delta_{LFD-NP}}\right]=\left[\frac{(1+I_{LRFD-EFF})}{(1+I_{LFD})}\right]\cdot\left[\frac{m}{i}\right]\cdot\left[\frac{\Delta_{LRFD-WT-P}}{\Delta_{LFD-WT-NP}}\right]\cdot\left[\frac{\Delta_{LRFD-WT-NP}}{\Delta_{LRFD-WT-NP}}\right] \quad (5\text{-}10)$$

where

$\Delta_{LRFD\text{-}WT\text{-}NP}$ = LRFD deflection of a whole truck on a single girder on a non-prismatic structure, obtained from CONSYS™, no impact, no distribution factor

Equation 5-5c is then rearranged into Equation 5-11.

$$\left[\frac{\Delta_{LRFD-P}}{\Delta_{LFD-NP}}\right]=\left[\frac{(1+I_{LRFD-EFF})}{(1+I_{LFD})}\right]\cdot\left[\frac{m}{i}\right]\cdot\left[\frac{\Delta_{LRFD-WT-NP}}{\Delta_{LFD-WT-NP}}\right]\cdot\left[\frac{\Delta_{LRFD-WT-P}}{\Delta_{LRFD-WT-NP}}\right] \quad (5\text{-}11)$$

Equation (5-11) now contains five terms with one term on the left and four terms on the right side of the equation. The third term on the right side now compares the LRFD deflection of a whole truck on a single girder in a non-prismatic structure with the LFD deflection of a whole truck on a single girder in a non-prismatic structure. The fourth term compares the LRFD deflection of a whole truck on a single girder in a prismatic structure with the LRFD deflection of a whole truck on a single girder in a non-prismatic structure. This final equation shown in final form as Equation 5-12 now compares the overall difference between LRFD maximum deflection and LFD maximum deflection, and Equation 5-12 qualifies and quantifies the four key differences between LRFD specifications and LFD specifications.

$$\left[\frac{\Delta_{LRFD-P}}{\Delta_{LFD-NP}}\right]=\left[\frac{(1+I_{LRFD-EFF})}{(1+I_{LFD})}\right]\cdot\left[\frac{m}{i}\right]\cdot\left[\frac{\Delta_{LRFD-WT-NP}}{\Delta_{LFD-WT-NP}}\right]\cdot\left[\frac{\Delta_{LRFD-WT-P}}{\Delta_{LRFD-WT-NP}}\right] \quad (5\text{-}12)$$

Equation 5-12 consists of the following five terms:

$$\left[\frac{\Delta_{LRFD-P}}{\Delta_{LFD-NP}}\right]$$

This term is the ratio of the LRFD maximum deflection of a girder in a prismatic structure to the LFD maximum deflection of a girder in a non-prismatic structure. This ratio represents the overall difference between the LRFD specifications and the LFD specifications.

$$\left[\frac{(1+I_{LRFD-EFF})}{(1+I_{LFD})}\right]$$

This term is the ratio of the effective LRFD impact factor to the LFD impact factor. This term quantifies the difference between LRFD and LFD calculation and application of impact factors.

$$\left[\frac{m}{i}\right]$$

This term is the ratio of the LRFD multiple presence factor to the LFD multiple presence factor. This term quantifies the difference between LRFD and LFD in the application of multiple presence factors.

$$\left[\frac{\Delta_{LRFD-WT-NP}}{\Delta_{LFD-WT-NP}}\right]$$

This term is the ratio of the LRFD deflection from a whole truck on a single girder of a non-prismatic structure to the LFD deflection from a whole truck on a single girder of a non-prismatic structure. Since neither of these deflections is altered by impact, distribution or multiple presence factors, and they are both whole truck loads on the same non-prismatic structure, the only difference between the two are the differences present in LRFD loading versus LFD loading. This term, therefore, is a quantification of the difference between LRFD and LFD loading methods.

$$\left[\frac{\Delta_{LRFD-WT-P}}{\Delta_{LRFD-WT-NP}}\right]$$

This term is the ratio of the LRFD deflection from a whole truck on a single girder of a prismatic structure to the LRFD deflection from a whole truck on a single girder of a non-prismatic structure. Since both the loads are LRFD, the only difference between the two is in how the bridge is modeled. One is modeled as a prismatic bridge, while the other is modeled as a non-prismatic bridge. This ratio quantifies the difference that arises from the different longitudinal analysis methods of LRFD modeling of a bridge as a prismatic structure versus the LFD specifications modeling the bridge as a non-prismatic structure. Incidentally, since both values are LRFD models, they are going to have the same effective impact factors, multiple presence factors and distribution factors. This term could then be replaced with the following term and still achieve the same ratio.

$$\left[\frac{\Delta_{LRFD-P}}{\Delta_{LRFD-NP}}\right]$$

This term is the ratio of the maximum LRFD deflection of a girder in a prismatic structure to the maximum LRFD deflection of a girder in a non-prismatic structure. Using this term yields Equation 5-13.

$$\left[\frac{\Delta_{LRFD-P}}{\Delta_{LFD-NP}}\right]=\left[\frac{(1+I_{LRFD-EFF})}{(1+I_{LFD})}\right]\cdot\left[\frac{m}{i}\right]\cdot\left[\frac{\Delta_{LRFD-WT-NP}}{\Delta_{LFD-WT-NP}}\right]\cdot\left[\frac{\Delta_{LRFD-P}}{\Delta_{LRFD-NP}}\right] \qquad (5\text{-}13)$$

5.3 Sample Calculation Using Missouri Bridge A6101

A sample of calculating the differences between LRFD and LFD specifications is shown using Missouri Bridge A6101.

5.3.1 Initial Data from CONSYS™

Three different CONSYS™ models are used to obtain the necessary data to begin comparing LRFD specifications to LFD specifications. The first model is a prismatic LRFD model of Missouri Bridge A6101 used to obtain $\Delta_{LRFD\text{-}WT\text{-}P}$, M_{LL}, and M_{LL+IM} of:

$\Delta_{LRFD\text{-}WT\text{-}P} = 2.237$ in
$M_{LL} = 1992$ kip·ft
$M_{LL+IM} = 2306$ kip·ft

The second model developed in CONSYS™ is a LFD non-prismatic model of Missouri Bridge A6101, which is used to obtain a $\Delta_{LFD\text{-}WL\text{-}NP}$ of:

$\Delta_{LFD\text{-}WL\text{-}NP} = 1.112$in

The third model developed in CONSYS™ is a LRFD non-prismatic model of Missouri Bridge A6101, which is used to obtain a $\Delta_{LRFD\text{-}WT\text{-}NP}$ of:

$\Delta_{LRFD\text{-}WT\text{-}N} = 2.555$ in

5.3.2 Multiple Presence Factors

Since Missouri Bridge A6101 is a three-lane bridge the multiple presence factors of m and i are:

$i = 0.90$
$m = 0.85$

5.3.3 Impact Factors

From previous analysis in Section 2.9, I_{LFD} is found to be:

$I_{LFD} = 0.19$

By using M_{LL+IM} and M_{LL} in Equation 5-3, $I_{LRFD\text{-}EFF}$ is found to be:

$$I_{LRFD-EFF} = \frac{M_{LL+IM}}{M_{LL}} - 1$$

$$I_{LRFD-EFF} = \frac{2306 kip \cdot ft}{1992 kip \cdot ft} - 1$$

$I_{LRFD\text{-}EFF} = 0.158$

Since $I_{LRFD\text{-}EFF}$ is less than 0.33, the lane plus 25% Truck plus Impact controls the maximum deflection.

5.3.4 Maximum Deflections

Using Equation 5-1, $\Delta_{LFD\text{-}NP}$ is calculated:

$$\Delta_{LFD-NP} = (\Delta_{LFD-WL-NP}) \cdot (1 + I_{LFD}) \cdot \left(\frac{2n_{designlanes}}{n_{girder}} \right) \cdot i$$

$$\Delta_{LFD-NP} = (1.112in) \cdot (1 + 0.19) \cdot \left(\frac{2 \cdot 3lanes}{5girders} \right) \cdot 0.90$$

$$\Delta_{LFD-NP} = 1.43in$$

Using Equation 5-2, $\Delta_{LRFD\text{-}P}$ is calculated:

$$\Delta_{LRFD-P} = (\Delta_{LRFD-WT-P}) \cdot (1 + I_{LRFD-EFF}) \cdot \left(\frac{n_{designlanes}}{n_{girder}} \right) \cdot m$$

$$\Delta_{LRFD-P} = (2.237in) \cdot (1 + 0.158) \cdot \left(\frac{3lanes}{5girders} \right) \cdot 0.85$$

$$\Delta_{LRFD-P} = 1.321in$$

Using Equation 5-2, refitted for a non-prismatic structure, $\Delta_{LRFD\text{-}NP}$ is calculated:

$$\Delta_{LRFD-NP} = (\Delta_{LRFD-WT-NP}) \cdot (1 + I_{LRFD-EFF}) \cdot \left(\frac{n_{designlanes}}{n_{girder}} \right) \cdot m$$

$$\Delta_{LRFD-NP} = (2.555in) \cdot (1 + 0.158) \cdot \left(\frac{3lanes}{5girders} \right) \cdot 0.85$$

$$\Delta_{LRFD-NP} = 1.51in$$

Equation 5-13 is then used to calculate the different ratios for each of the differences between LRFD and LFD specifications.

$$\left[\frac{\Delta_{LRFD-P}}{\Delta_{LFD-NP}} \right] = \left[\frac{(1 + I_{LRFD-EFF})}{(1 + I_{LFD})} \right] \cdot \left[\frac{m}{i} \right] \cdot \left[\frac{\Delta_{LRFD-WT-NP}}{\Delta_{LFD-WT-NP}} \right] \cdot \left[\frac{\Delta_{LRFD-P}}{\Delta_{LRFD-NP}} \right] \qquad (5\text{-}13)$$

where:

The overall LRFD to LFD ratio is calculated:

$$\left[\frac{\Delta_{LRFD-P}}{\Delta_{LFD-NP}} \right] = \left[\frac{1.321in}{1.43in} \right] = 0.924$$

The impact factor ratio is calculated:

$$\left[\frac{(1 + I_{LRFD-EFF})}{(1 + I_{LFD})} \right] = \left[\frac{(1 + 0.158)}{(1 + 0.19)} \right] = 0.973$$

The multiple presence factor ratio is calculated:

$$\left[\frac{m}{i}\right] = \left[\frac{0.85}{0.9}\right] = 0.944$$

The longitudinal analysis ratio is calculated:

$$\left[\frac{\Delta_{LRFD-P}}{\Delta_{LRFD-NP}}\right] = \cdot\left[\frac{1.321in}{1.51in}\right] = 0.875$$

Equation 5-7 is then used to back calculate the load effect ratio:

$$\left[\frac{\Delta_{LRFD-P}}{\Delta_{LFD-NP}}\right] = \left[\frac{(1+I_{LRFD-EFF})}{(1+I_{LFD})}\right]\cdot\left[\frac{m}{i}\right]\cdot\left[\frac{\Delta_{LRFD-WT-NP}}{\Delta_{LFD-WT-NP}}\right]\cdot\left[\frac{\Delta_{LRFD-P}}{\Delta_{LRFD-NP}}\right]$$

$$[0.924] = [0.973]\cdot[0.944]\cdot\left[\frac{\Delta_{LRFD-WT-NP}}{\Delta_{LFD-WT-NP}}\right]\cdot[0.875]$$

$$\left[\frac{\Delta_{LRFD-WT-P}}{\Delta_{LRFD-WT-NP}}\right] = 1.15$$

5.4 Results from Analysis Study Bridges

Analysis was performed on the 10 selected bridges. The results, with a discussion, follows.

5.4.1 LRFD/LFD Deflection Ratio Results

For each bridge the ratio of LRFD maximum prismatic deflection to LFD maximum non-prismatic deflection is calculated and shown in Figure 5.1.

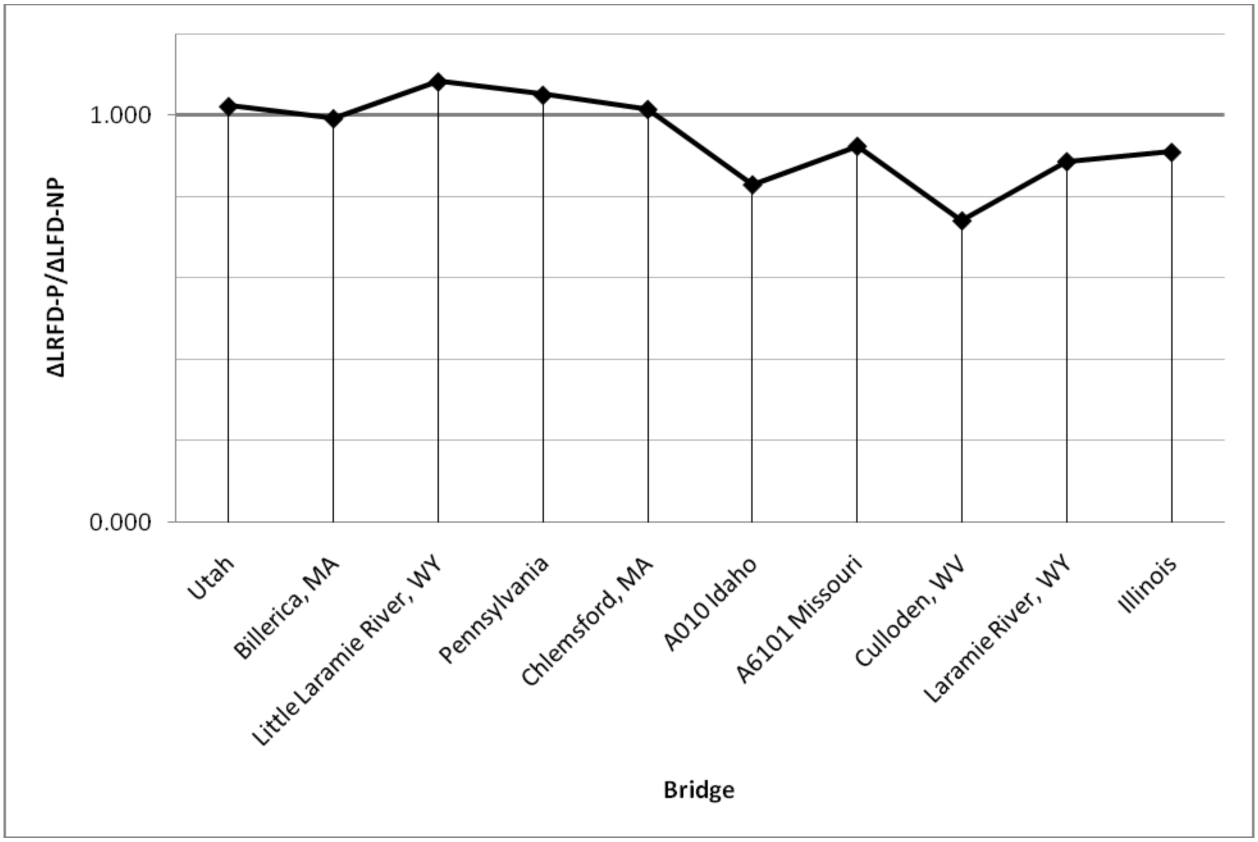


Figure 5.1 LRFD vs. LFD Deflection

Six of the selected 10 bridges have a deflection ratio of less than one. These bridges are Billerica, MA; A010 Idaho; A6101 Missouri; Culloden, WV; Laramie River, WY; and Illinois. A deflection ratio of less than one shows that the LRFD method calculated a smaller maximum deflection value than the maximum deflection value calculated by the LFD method.

From Figure 5.1 a conclusion can be made that the deflection ratio seems to be influenced by the number of spans in a bridge. This leads to impacts on single-span design and multi-span design.

5.4.1.1 Impact on Single-Span Bridges

Not surprisingly, three of the four bridges that have deflection ratios greater than one are single-span bridges (Utah; Little Laramie, WY; Pennsylvania). A single span bridge has only positive moment regions, and therefore the prismatic analysis has no effect. This prismatic analysis is one of the key advantages the LRFD specifications have over LFD specifications. Only one single-span bridge had a deflection ratio less than one, Billerica, MA, and this bridge had a deflection ratio only slightly less than one. The trend indicates that using the LRFD specifications may result in greater deflections for single-span bridges. The future impact of this trend as states adopt LRFD specifications could be that single-span bridge design will result in higher deflections than if LFD specifications are used. This trend seems to indicate that the LRFD deflection criteria may negatively influence the design economy of single-span bridges.

5.4.1.2 Impact on Multi-Span Bridges

Of the six multi-span bridges, only one has a deflection ratio greater than one, and, of the bridges with three spans or more, none of the three bridges has a deflection ratio greater than one. This trend indicates that the LRFD deflection criteria for multi-span bridges may yield smaller deflections than LFD deflection criteria. This trend seems to indicate that the LRFD deflection criteria may positively influence the design economy of multi-span bridges.

5.4.2 Qualified and Quantified LRFD/LFD Deflection Ratio Results

While the previous conclusions could be made with Figure 5.1, it is necessary to look at qualified and quantified characteristics of the relation. This analysis is shown in Table 5.2.

Table 5.2 LRFD vs. LFD Deflection Comparison Equation for Serviceability

Bridge	$\frac{\Delta_{LRFD-P}}{\Delta_{LFD-NP}}$	$\frac{(1+I_{LRFD-EFF})}{(1+I_{LFD})}$	$\frac{m}{i}$	$\frac{\Delta_{LRFD-P}}{\Delta_{LRFD-NP}}$	$\frac{\Delta_{LRFD-WT-P}}{\Delta_{LRFD-WT-NP}}$
Utah	1.022	1.067	0.944	1.000	1.02
Billerica, MA	0.992	0.959	0.867	1.000	1.19
Little Laramie River, WY	1.083	1.082	1.000	1.000	1.00
Pennsylvania	1.050	1.107	0.944	1.000	1.00
Chlemsford, MA	1.015	0.914	0.944	0.948	1.24
A010 Idaho	0.829	1.044	0.944	0.828	1.02
A6101 Missouri	0.924	0.973	0.944	0.875	1.00
Culloden, WV	0.740	1.056	0.867	0.820	1.00
Laramie River, WY	0.886	1.067	1.000	0.807	1.03
Illinois	0.909	0.984	1.000	0.881	1.14
Average	0.945	1.025	0.945	0.916	1.06

Several items from Table 5.2 will be discussed for each ratio.

5.4.2.1 LRFD/LFD Deflection Ratio

The average of the deflection ratios for the ten bridges is 0.945; this means that on average, for these 10 bridges, the LRFD deflection criteria lowers the deflection criteria by 5.5% compared to LFD deflection criteria. The deflection values range from a high of 1.083, Little Laramie River, to a low of 0.740, Culloden WV. This means that the LRFD deflection criteria has a maximum increase in deflection of 8.3% versus LFD deflection criteria, and a maximum decrease of 26% versus LFD deflection criteria. The average deflection ratio of 0.884 for multi-span bridges and the average deflection ratio of 0.845 for bridges with three or more spans reinforce the previous statement that the LRFD specifications may help in design economy for multi-span bridges.

5.4.2.2 Impact Factor Ratio

Six of the 10 bridges analyzed have an impact factor ratio greater than one. An impact factor ratio of greater than one is a result of the effective LRFD impact factor being greater than the impact factor of the LFD deflection criteria. The average impact factor ratio for the 10 bridges is 1.025, which translates to mean that, on average, the LRFD specifications have a slightly more severe effective impact ratio than the LFD specifications by 2.5%.

5.4.2.3 Multiple Presence Ratio

The average multiple presence ratio for the 10 bridges is 0.945. This ratio is based solely on the number of lanes on a bridge. If a bridge has one lane, the LRFD multiple presence factors are 20% more severe than LFD multiple presence factors (none of the study bridges has only one lane). If a bridge has two lanes, the multiple presence factors are equivalent for LRFD and LFD. If a bridge has three lanes, the LRFD multiple presence factors are 5.55% less than LFD multiple presence factors. If a bridge has four or more lanes, the LRFD multiple presence factors are 13.3% less than LFD multiple presence factors. This behavior is represented in the individual bridge multiple presence ratios.

5.4.2.4 Longitudinal Analysis Ratio

The longitudinal analysis ratio is the ratio of prismatic to non-prismatic modeling for a bridge. Since single span bridges contain only positive moment regions, the prismatic cross section is the same as the non-prismatic region. This is why, for the four single-span bridges: Utah; Billerica, MA; Little Laramie River, WY; and Pennsylvania, the longitudinal analysis ratio are equal to one. For multi-span bridges, in the negative moment regions the prismatic cross section will be larger and, therefore, stiffer than the non-prismatic cross section. This is why all the longitudinal analysis ratios are less than one for the multi-span bridges. The average longitudinal analysis ratio of the 10 bridges is 0.916, while the average longitudinal analysis for the six multi-span bridges is 0.860. For multi-span bridges, the modeling of the bridge as prismatic is a key reason why LRFD deflections tend to be less than LFD deflections.

5.4.2.5 Load Effect Ratio

The only difference between LRFD deflection criteria loading and LFD deflection criteria loading is LRFD deflection criteria contain an additional load case: 25% truck plus impact plus lane. In bridges where the truck load is controlled, the load effect ratio is equal to one. In bridges where the 25% Truck plus Impact plus lane load controlled, the load effect ratio is greater than “1.” This ratio can therefore counteract other ratios, such as the longitudinal analysis ratio, that tend to lower the LRFD deflection compared to the LFD deflection.

5.5 Summary

To anticipate the effect that will occur as states begin to adopt LRFD deflection criteria requires the derivation of a method that could qualify and quantify the differences that exist between the specifications. This derivation compares the differences between the LRFD and LFD specifications. Four key differences are identified: design load application, multiple presence factors, calculation and application of impact factors, and longitudinal analysis techniques. Equation 5-12 and Equation 5-13 qualify and quantify the differences between the LRFD and LFD specifications. A sample calculation is performed on Missouri Bridge A6101. Finally the results of the analysis for the 10 selected bridges is presented and discussed. It is observed that the LRFD specifications tend to produce slightly higher deflections in single-span bridges and lower deflections in multi-span bridges when compared to the LFD specifications. The possible effect could be worse design economy in single-span bridges and better design economy in multi-span bridges.

6. SUMMARY, CONCLUSIONS AND FUTURE WORK

6.1 Introduction and MPC Research Objectives

This interim report presents the work and findings of the MPC project as work continues toward the overall research objectives to produce rational serviceability provisions for steel girder bridges. The specific objectives of the MPC project are to conduct serviceability comparisons between state live-load deflection criteria and the AASHTO LRFD and AASHTO LFD standards to provide information on the conservative nature of state serviceability criteria and loss of economical benefits for steel bridge design. The relationship between the LRFD and LFD methods and the impact of moving toward LRFD is also examined. A final report will be submitted to the MPC with the comprehensive research results from the overall research effort in the summer of 2008.

6.2 MPC Research Conclusions

Many state transportation departments have established conservative live-load deflection criteria for steel bridges. The self-imposed state deflection criteria can be more conservative than AASHTO LFD or LRFD requirements in several ways. Conservative deflection limits, applied live-load and lane distribution factors produce conservative deflection criteria. This study determines serviceability comparisons between six state deflection criteria and the AASHTO standards. Figures 3.1 and 4.1 provide calculated state criteria deflections divided by the AASHTO deflection for each of the bridges examined in this study. This figure shows that many states produce live-load deflections significantly larger than the AASHTO deflections for all the sample bridges. One state produced deflections that are as high as four times the AASHTO criteria deflection.

States also have more restrictive deflection limits than the L/800 and L/1000 limits required by the AASHTO specifications. The combination of larger live-load deflections and more restrictive limits would require a significantly stiffer bridge to meet state limits than it would to meet AASHTO limits (Figures 3.4 and 4.4). For this reason, there are concerns that conservative state deflection limits will control bridge designs and especially those using high performance steel. Costs for conventional steel bridges would significantly increase for these states, and the economic benefits of HPS would not be experienced when subjected to the conservative state deflection limits.

The importance of these results is that these ten existing bridges have performed well and do not have deflection problems for either user comfort or structural damage. However, these bridges would be deemed as unacceptable in the states with the conservative deflection criteria. In those states, the bridges would require additional steel over and above that required for safety to meet the deflection criteria. Thus, the economy of steel bridges, and especially HPS 70W bridges, would suffer. Changes to conservative state live-load deflection criteria should be considered to allow for the cost effective use of steel and high performance steel.

Results show that the current AASHTO LFD and LRFD deflection criteria typically do not control in design and, therefore, do not have a negative impact on economy of conventional or high performance steel bridges. As state move from using LFD procedures to LRFD, the AASHTO deflection requirements should usually not hamper steel bridge economy. Figure 5.1 shows the relation between LRFD and LFD deflections, and Table 5.1 quantifies the characteristics of the relationship. It is observed that the LRFD specifications tend to produce slightly higher deflections in single-span bridges and lower deflections in multi-span bridges when compared to the LFD specifications.

6.3 Future Work

In section 1, four issues with bridge deflection limits that the engineering community is currently attempting to address are discussed:

1. Current deflection limits are intended to limit user discomfort and limit deflection induced structural damage. However, past practice and research has shown that limiting deflections may not be adequate for either user comfort or damage.

2. The recent LRFD provisions include optional deflection limit criteria similar to the LFD provisions. As states move toward adoption of LRFD, most states have decided to apply the optional limits or even more conservative limits. However, the loading and analysis procedures have changed from LFD to LRFD and the impact of the newer LRFD criteria are unknown on the role of deflection limits for design economy.

3. The steel industry has developed a high performance steel (HPS) for steel bridges that has improved the quality of the steel material and led to cost savings through weight savings. However, if deflections control in the design, which may happen with the higher strength HPS, these benefits are not realized.

4. State's application of deflection criteria vary significantly across the country and many states have adopted more restrictive deflection criteria which inherently impacts economy of steel bridges. This is especially true when using HPS, but with the more restrictive deflection limits, conventional steel bridges would also be more costly if deflections control the design.

Section 1 also discusses the overall research effort for serviceability of bridges and describes three research efforts underway to address the issues above. The MPC project primarily addressed issues 2 and 4. The MPC contract was leveraged to expand the scope (to meet the overall research objectives and address all four issues) with other research entities. Issues 1 and 3 are currently being addressed in two contracts, one with an HDR/FHWA research project and the other with an American Iron & Steel Institute (AISI) and Idaho DOT contract. All the research projects are inter-related in the overall effort to produce rational serviceability/deflection criteria for steel girder bridges. In addition, there needs to be an overall comprehensive report that disseminates the results from the total research effort. The reporting requirements for the HDR/FHWA and AISI/IdDOT contracts are not conducive to the dissemination requirements. The overall research dissemination report will be the final MPC report.

The HDR/FHWA project includes four tasks that expand the work from the MPC project and advances the overall research effort. Task 1 is a national survey of states on their application of LRFD deflection requirements. This task has been completed, and the survey results were used in the MPC project for studying state LRFD criteria. Task 2 is to further assess the implications on design from the state practices. This is similar to objectives in the MPC project; however, the HDR/FHWA project will use the results of the MPC work and extend the analysis. This will include projecting the analysis past the "as-built" condition of the MPC work into evaluation at design limits (both strength and deflection limits). The HDR/FHWA project will also investigate the mechanistic strains expected in the deck over the piers at various load levels to possibly develop more adequate provisions for deformation-induced structural damage. Task 3 is to develop more rational specifications that assure a more unified application of deflection (serviceability) limits in current practice. The proposed criteria will be based on the work of the MPC and HDR/FHWA research. The results will be compared to past research and current codes that limit bridge deflections. The outcome and benefit of this work will be improved serviceability specifications, improved consistency of design across the states, and more economical use of high performance materials in bridges such as HPS.

The AISI/IDDOT project entails field testing a bridge in Idaho to determine the service performance and serviceability behavior. The design is "out-of-the-box" in terms of typical bridges, and deflections and in-service performance becomes more important to predict and understand. The recommended procedures and the results of the MPC and HDR/FHWA projects will be applied and tested on the Idaho bridge. The outcome should be confirmation of the recommendations and a demonstration of rational procedures to other states, especially those that implement conservative deflection criteria.

6.4 Summary and Final Report

The objective of the overall research work is to produce rational deflection criteria (or a form of serviceability criteria) to limit user discomfort and deformation-induced structural damage in steel girder bridges. Current AASHTO criteria do not effectively meet that purpose. Additionally, many states have chosen to use more conservative deflection criteria than AASHTO. This results in more costly bridges and inconsistent design procedures. It also impedes the use of high performing materials such as HPS when deflections and not strength (safety) controls the design. Implementation of realistic and appropriate deflection limits over the nation's bridge inventory will result in more efficient and less costly bridges. Conventional steel and high performance steel bridge design will be more consistent and cost effective across the country.

This interim report for the MPC portion of the overall research effort includes the results of the MPC research contract work plan. The MPC program will be used for final dissemination of the overall research effort with a final report to be submitted in summer 2009. The final report will include the work of the MPC project in addition to the work and results of the HDR/FHWA and AISI/IDDOT research.

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