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Improved Live Load Deflection Criteria for Steel Bridges

Prepared for:

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Contents

Summary

This research has examined the AASHTO live-load deflection limit for steel bridges. The AASHTO Standard Specification limits live-load deflections to $\frac{L}{800}$ for ordinary bridges and $\frac{L}{1000}$ for bridges in urban areas that are subject to pedestrian use. This limit is also incorporated in the AASHTO LRFD Specifications in the form of an optional serviceability criteria. This limit has not been a controlling factor in most past bridge designs, but it will play a greater role in the design of bridges built with new HPS 70W steel. This study documented the role of the AASHTO live-load deflection limit of steel bridge design, determined whether the limit has beneficial effects on serviceability and performance, and established whether the deflection limit was needed. Limited time and funding was provided for this study, but an ultimate goal was to establish recommendations for new design provisions that would assure serviceability, good structural performance and economy in design and construction.

A literature review was completed to establish the origin and justification for this deflection limit. This review examined numerous papers and reports, and a comprehensive reference list is provided. The work shows that the existing AASHTO deflection limit was initially instituted to control bridge vibration, but deflection limits are not a good method for controlling bridge vibration. Alternate design methods are presented. A survey of state bridge engineers was simultaneously completed to examine how these deflection limits are actually applied in bridge design. The survey also identified bridges that were candidates for further study on this research issue. Candidate bridges either:

- failed to meet the existing deflection limits,
- exhibit structural damage that was attributable to excessive bridge deflection,
- were designed with HPS 70W steel, or
- had pedestrian or vehicle occupant comfort concerns due to bridge vibration.

The survey showed wide variation in the application of the deflection limit in the various states, and so a parameter study was completed to establish the consequences of this variation on bridge design. The effect of different load patterns, load magnitudes, deflection limits, bridge span length, bridge continuity, and other factors were examined. There is wide variation in the application of the existing deflection limit, because of the variation in the actual deflection limits, the variation in the load magnitude and load pattern used to calculate the deflection, the application of load factors and lane load distribution factors, and other effects. The difference between the least restrictive and most restrictive deflection limit may exceed 1,000%. The load pattern and magnitude have a big impact on this variation. Some states use truck loads, some use distributed lane loads, and some use combinations of the above. Truck loads provide the largest deflection for short span bridges. Distributed lane loads provide the largest deflections for long span bridges.

The survey identified a number of bridges which were experiencing structural damage and reduced service life associated with bridge deflections. Design drawings, inspection reports, photographs, and other information was collected on these bridges. They were grouped and analyzed to:

- determine whether the damage was truly caused by bridge deflections,
- determine whether the AASHTO live-load deflection limit played a role in controlling or preventing this damage, and
- examine alternate methods of controlling or preventing this damage.

This analysis showed that a substantial number of bridges are damaged by bridge deformation. This deformation is related to bridge deflection. The deformations that cause the damage are relative deflections between adjacent members, local rotations and deformations, deformation induced by bridge skew and curvature, and similar concerns. None of these deformations are checked with the existing $\frac{L}{800}$ live-load deflection limit.

Additional analyses were performed to examine how the deflection limit interacts with bridge vibration, the span-to-depth ($\frac{L}{D}$) ratio and other design parameters. The study examined the effect these parameters on the economy and performance of bridge design. The AASHTO live-load deflection limit is less likely to influence the design of bridges with small $\frac{L}{D}$ ratios and is more likely to control the superstructure member sizes as the $\frac{L}{D}$ ratio increases. Application of the deflection limit with truck load only shows that the existing AASHTO deflection limits will have a significant economic impact on some steel I-girder bridges built from HPS 70W steel. Simple span bridges are more frequently affected by this limit than continuous bridges. However, continuous bridges are also likely to be more frequently affected by existing deflection limits if the span length, L, is taken as the true span length rather than the distance between inflection points in the application of the deflection limit. The study shows that many bridges the satisfy the existing deflection limit are likely to provide poor vibration performance, while other bridges failing the existing deflection limit will provide good comfort characteristics.

Lastly, this report summarizes major findings and presents proposed design recommendations and further research requirements.

Acknowledgments

This research report describes a cooperative research study completed at the University of Washington and West Virginia University. Funding for this work is provided by the National Cooperative Research Program under NCHRP 20-07/133 and by the American Iron and Steel Institute through project entitled "Vibration and Deflection Criteria for Steel Bridges." The authors gratefully acknowledge this support.

Chapter 1

Introduction

1.1. Problem Statement

The AASHTO Standard Specification (AASHTO, 1996) limits live-load deflections to $\frac{L}{800}$ for ordinary bridges and $\frac{L}{1000}$ for bridges in urban areas that are subject to pedestrian use. These limits are required for steel, prestressed and reinforced concrete, and other bridge superstructure types. Bridges designed by the AASHTO LRFD Specification (AASHTO, 1998) have an optional deflection limit. The specifications and the LRFD commentary do not provide detailed explanations or justification for these limits. Historically, the deflection limit has not affected a significant range of bridge designs. However, recent introduction of high performance steel (HPS) may change this fact. HPS has a higher yield stress than other steels commonly used in bridge design (Fy=70 ksi and higher as opposed to 50 ksi), and the larger yield stress permits smaller cross sections and moments of inertia for bridge members. As a result, deflections may be larger for HPS bridges, and deflection limits are increasingly likely to control the design of bridges built from these new materials. It is therefore necessary to ask:

- How the deflection limit affects bridge performance?
- Whether the deflection limit is justified or needed?
- Whether it achieves its intended purpose?
- Whether it benefits the performance of steel bridges?
- Whether it affects the economy of steel bridges?

This research study was jointly funded under the NCHRP 20-7 program and the American Iron and Steel Institute, and the research was initiated to determine whether the deflection limits for steel bridges are needed or warranted. The study focuses on steel bridges, and the particular goals are -

- to determine how the deflection limits are employed in steel bridge design in the US,
- to determine the rationale behind existing design provisions and to compare AASHTO provisions to design methods used in other countries,
- to evaluate the effect of AASHTO and other existing deflection limits on steel bridge design and performance, and evaluate where existing deflection limits prevent damage and reduced service life,
- to document any problems that have occurred or are prevented by the existing limits,
- and, if problems are found, to evaluate whether the existing limit is the best possible method of achieving the serviceability design objectives.

1.2. Directions of Research

The research started in December 2000. The research contract was awarded to the University of Washington (UW). However, early in the study it was noted that a parallel study was in progress at West Virginia University (WVU) with funding from the American Iron and Steel Institute (AISI) and the West Virginia Department of Highways (WVDOH). The WVU study was interested in bridge deflection limits, but it was also concerned with bridge vibrations and the development of improved methods of vibration control.

This NCHRP 20-7/133 funding was very limited, and so the work had to be done in a way that will provide the maximum benefit at minimum cost. Further, the similarities of the research justified cooperation and coordination between the two research teams. Cooperation initially was arranged through a conference call between the UW and WVU researchers (Charles Roeder for UW and Karl Barth for WVU) and the responsible research program managers (David Beal for NCHRP and Camille Rubeiz for AISI). Cooperation between the two research teams were agreed at that time, and the researchers have had numerous meetings, email exchanges, and conference calls for the duration of this project. The UW issued a subsequent subcontract to WVU to help balance funding with the responsibilities. Through these efforts the researchers have exchanged information throughout the research effort to date. Researchers from both universities are co-authoring this report and all papers resulting from this coordinated effort.

The research was divided into 6 tasks. The first task provided an initial review of existing literature and the state of practice for steel bridge deflection control. Task 2 provided an Interim Report, which summarized the results of Task 1, and proposed directions for the work to be completed during Tasks 3, 4, 5 and 6. The Interim Report was prepared in March 2001, and was reviewed by an NCHRP Project Panel as well as being submitted to AISI. The panel provided advice and guidance on the research progress, and this guidance was used to direct the research of Tasks 3, 4, and 5. Tasks 3, 4 and 5 consisted of follow up analysis to examine the deflection limits. The range of variability in the actual professional practice was determined. Bridges, which had reported damage due to excessive deflection or deformation, were analyzed to determine whether deflections could or do prevent this damage. Design studies were completed to determine when and how deflections would affect steel bridge design. Task 6 included preparation of a final report with the summary and recommendations from the research.

1.3. Report Content and Organization

This is the Final Report required by Task 6 of the project. It describes the progress made throughout the coordinated project. Chapter 1 of this report has introduced the issues of concern. Chapter 2 summarizes the literature review, and Chapter 3 presents the results of a survey of bridge engineering practice. The work in these first 3 chapters was described in somewhat greater detail in the Interim Report submitted to

NCHRP and AISI in March 2001. This material is summarized in this final report so that the reader can develop a complete understanding of the issues at hand.

The details of the work in Tasks 3, 4, and 5 were finalized after obtaining feedback from the Project Panel from the Interim Report. This work is summarized in Chapters 4, 5, 6 and 7 of this report. The survey shows a wide variation of the professional practice, and Chapter 4 summarizes a parameter study completed at the UW to examine and understand the impact of this variation on the application of the deflection limit. The survey of Chapter 3 identified bridges with structural problems that were attributed to bridge deflections or deformations. Comprehensive analyses of these bridges were completed at UW, and the results of these specific bridge analyses are provided in Chapter 5. WVU completed a series of evaluations of recent bridge designs to establish how deflection limits and bridge vibrations affect their performance, this work is summarized in Chapter 6. WVU also completed a design parameter study to determine the effect and economic consequences of the deflection limits on actual bridge design. Chapter 7 provides a summary of this work. Finally, Chapter 8 provides a brief summary of the work completed and a discussion of the conclusions and recommendations from this research study.

Chapter 2

Literature Review

2.1. Overview and Historical Perspective

The original source of the present AASHTO deflection limits was of interest to this study, as the possible existence of a rational basis for the original deflection limits is an important consideration. The source of the present limitations is traceable to the 1905 American Railway Engineering Association (AREA) specification where limits to the span-to-depth ratio, $\frac{L}{D}$, of railroad bridges were initially established. $\frac{L}{D}$ limits indirectly control the maximum live-load deflection, and Table 2.1 shows the limiting $\frac{L}{D}$ ratios that have been incorporated in previous AREA and AASHTO specifications [ASCE, 1958]. Although, initially, live load deflections were not directly controlled, the 1935 AASHTO specification included the following stipulation:

> *If depths less than these are used, the sections shall be so increased that the maximum deflection will be not greater than if these ratios had not been exceeded.*

Table 2.1 Span-to-Depth, $\frac{L}{D}$, ratios in A.R.E.A. and A.A.S.H.O. [ASCE, 1958].

Year (s)	**Trusses**	**Plate Girders**	**Rolled Beams**
A.R.E.A.			
1905	1 / 10	1 / 10	1 / 12
1907, 1911, 1915	1 / 10	1 / 12	1 / 12
1919, 1921, 1950, 1953	1 / 10	1 / 12	1 / 15
A.A.S.H.O.			
1913, 1924	1 / 10	1 / 12	1 / 20
1931	1 / 10	1 / 15	1 / 20
1935, 1941, 1949, 1953	1 / 10	1 / 25	1 / 25

It is valuable to note that, while $\frac{L}{D}$ limits have been employed for many years, the definitions of the span length, L, and the depth, D, have changed over time. Commonly, engineers have used either the center-to-center bearing distance or the distance between points of contraflexure to define span length. The depth has varied between the steel section depth and the total superstructure depth (steel section plus haunch plus concrete deck in the case of a plate or rolled girder). While these differences may appear to be small, they have a significant influence on the final geometry of the section, and they significantly affect the application of the $\frac{L}{D}$ and deflection limits.

Actual limits on allowable live-load deflection appeared in the early 1930's when the Bureau of Public Roads conducted a study that attempted to link the objectionable vibrations felt on a sample of bridges built in that era (ASCE, 1958; Oehler, 1970; Wright and Walker, 1971; and Fountain and Thunman, 1987). This study concluded that structures having unacceptable vibrations determined by subjective human response had deflections that exceeded $\frac{L}{800}$, and this conclusion resulted in the $\frac{L}{800}$ deflection design limit. Some information regarding the specifics of these studies is lost in history. However, the bridges included in this early study had wood plank decks, and the superstructure samples were either pony trusses, simple beams, or pin-connected through-trusses. The $\frac{L}{1000}$ deflection limit for pedestrian brides was set in 1960. Literature suggested that this limit was established after a baby was awakened on a bridge. The prominent mother's complaint attributed the baby's response to the bridge vibration, and the more severe deflection limit was established for bridges open to pedestrian traffic (Fountain and Thunman, 1987).

A 1958 American Society of Civil Engineers (ASCE) committee [ASCE, 1958] reviewed the history of bridge deflection criteria, completed a survey to obtain data on bridge vibrations, reviewed the field measurements of bridges subjected to moving loads, and gathered information on human perception to vibration. The committee examined the effect of the deflection limit on undesirable structural effects including:

- Excessive deformation stresses resulting directly from the deflection or from rotations at the joints or supports induced by deflections.
- Excessive stresses or impact factors due to dynamic loads.
- Fatigue effects resulting from excessive vibration.

The committee also considered the measures needed to avoid undesirable psychological reactions of pedestrians, whose reactions are clearly consequences of the bridge motion, and vehicle occupants, whose reactions may be caused by bridge motion or a combination of vehicle suspension/bridge interaction.

The committee noted that the original deflection limit was intended for different bridges than those presently constructed. Design changes such as increased highway live-loads and different superstructure designs such as composite design, pre-stressed concrete, and welded construction were not envisioned when the limit was imposed. The limited survey conducted by the committee showed no evidence of serious structural damage attributable to excessive live-load deflection. The study concluded that human psychological reaction to vibration and deflection was a more significant issue than that of structural durability and that no clear structural basis for the deflection limits were found.

A subsequent study (Wright and Walker, 1971) also investigated the rationality of the deflection limits and the effects of slenderness and flexibility on serviceability. They reviewed literature on human response to vibration and on the effect of deflection and vibration on deck deterioration. This study suggested that bridge deflections did not have a significant influence on structural performance, and that deflection limits alone were not a good method of controlling bridge vibrations or assuring human comfort.

Oehler (Oehler, 1970) surveyed state bridge engineers to investigate the reactions of vehicle passengers and pedestrians to bridge vibrations. Of forty-one replies, only 14 states reported vibration problems. These were primarily in continuous, composite structures due to a single truck either in the span or in an adjacent span. In no instance was structural safety perceived as a concern. The survey showed that only pedestrians or occupants of stationary vehicles objected to bridge vibration. The study noted that objectionable vibration could not be consistently prevented by a simple deflection limit alone. It was suggested that deflection limits and $\frac{L}{D}$ limits in the specifications be altered to classify bridges in three categories with the following restrictions:

1. *Bridges carrying vehicular traffic alone should have only stress restrictions.*

2. *Bridges in urban areas with moving pedestrians and parking should have a minimum stiffness of 200 kips per inch deflection to minimize vibrations.*

3. *Bridges with fishing benches, etc. should have a minimum stiffness of 200 kips per inch of deflection and 7.5% critical damping of the bridge to practically eliminate vibrations.*

Others (Fountain and Thunman, 1987) also suggested the AASHTO live-load deflection limits show no positive effect on bridge strength, durability, safety, maintenance, or economy. They noted that subjective human response to objectionable vibrations determined the $\frac{L}{800}$ in the 1941 AASHTO specifications, which were adopted after the 1930 Bureau of Public Roads study, but deflection limits do not limit the vibration and acceleration that induces the human reaction.

This chapter presents a comprehensive literature review on the dynamic performance of highway bridges subjected to moving loads. More detailed discussion is provided on 3 factors that influence or are influenced by, live-load deflection. These include:

- structural performance, mainly reinforced deck deterioration,
- bridge vibration characteristics, and
- human response to bridge vibration.

2.2. Effect of Bridge Deflections on Structural Performance

Deterioration of reinforced concrete bridge decks is an increasing problem in all types of bridge superstructure, and it is caused by various internal and external factors. Bridge deck deterioration reduces service life by reducing load capacity of the structure and the quality of the riding surface. It is logical to ask whether bridge deterioration is attributable to excessive bridge flexibility and deflection.

There are four main types of deck deterioration: spalling, surface scaling, transverse cracking, and longitudinal cracking. Spalling is normally caused by corrosion of reinforcement and freeze/thaw cycles of the concrete. Scaling is caused by improper finishing and curing of the concrete and the simultaneous effects of freeze-thaw cycles and de-icing salts.

Transverse cracking is the most common form of bridge deck deterioration. Plastic shrinkage of the concrete, drying shrinkage of the hardened concrete combined with deck restraint, settlement of the finished plastic concrete around top mat of reinforcement, long term flexure of continuous spans under service loads, and traffic induced repeated vibrations all contribute to this damage.

Longitudinal cracks occur as a result of poor mix design, temperature changes, live-load effects, or a reflection of shrinkage cracking. Multiple cracks appear on bridge decks that are fatigued or "worn out" from heavy traffic due to pounding caused from the wheel impact on the expansion joints and surface irregularities.

Research has shown that the width and intensity of these cracks tend to be uniformly distributed throughout the entire length of a bridge deck, rather than being concentrated in negative bending regions (Fountain and Thunman, 1987; Kansas State Highway Commission, 1965; and Krauss and Rogalla, 1996). One study (Fountain and Thunman, 1987) question the beneficial influence of the AASHTO deflection criteria, because flexural stresses in the deck of composite bridges are small. Bridge dynamic response changes very little as flexibility increases, because the lateral distribution of loads to adjacent girders increases with flexibility. In the negative moment regions of composite spans, the design flexural stresses in the deck are predictable and reinforcing steel can be provided for crack width control. They also argue that increased stiffness may increase deck deterioration, because the effects of volume change on the tensile stresses due to deck/beam interaction increase as the beam stiffness increases. They examined deck deterioration noted in field survey data accumulated by the Portland Cement Association (PCA) (PCA 1970) in cooperation with Federal Highway Administration (FHWA) from 20 states representative of various climates. The bridges included simple and continuous span concrete T-beams, slabs, box girders, and pre-stressed beams, as well as steel rolled beams, plate girders, deck and through trusses. These bridges were systematically and consistently inspected, and the damage characteristics were noted in detail. Laboratory studies of core samples of deteriorated and non-deteriorated areas were examined. No correlation was found between bridge type and either the amount or degree of deck deterioration.

Others (Krauss and Rogalla, 1996) reviewed literature, surveyed 52 transportation agencies throughout the U.S. and Canada and conducted analytical, field, and laboratory research. The survey was sent to develop an understanding of the magnitude and mechanistic basis of transverse cracking in recently constructed bridge decks. The analytical parametric study examined stresses in more than 18,000 bridge scenarios. Further, one deck replacement was monitored in the field, and laboratory experiments examined the effect of concrete mix and environmental parameters on cracking potential. It was concluded that cracking is more common among multi-span continuous steel girder structures due to restraint and that longer spans are more susceptible than shorter spans. It was felt that reducing deck flexibility may potentially reduce early cracking.

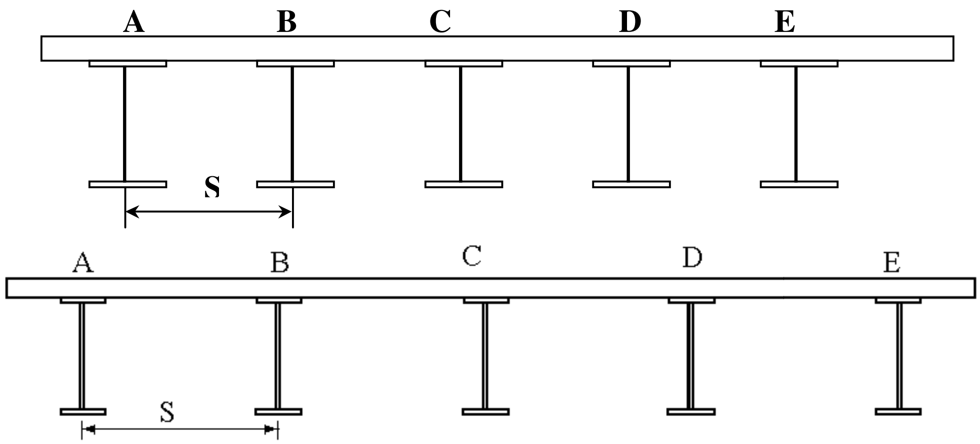
Three studies (Goodpasture and Goodwin, 1971; Wright and Walker, 1971; and Nevels and Dixon, 1973) focus on the relationship between deck deterioration and live-load deflection. Goodpasture and Goodwin studied 27 bridges in Phase I of their research to determine which type of bridges exhibited the most cracking. These bridges including plate girders, rolled beams, concrete girders, pre-stressed girders, and trusses. The effect of stiffness on transverse cracking was evaluated for 10 of the continuous steel bridges in Phase II. No correlation between girder flexibility and transverse cracking intensity could be established.

Wright and Walker show no evidence to associate spalling, scaling or longitudinal cracking with girder flexibility (Wright and Walker, 1971). Transverse deck moments lead to tension at the top of the deck and possible deck cracking, and were of interest to this research. The longitudinal deck moments are small. Figure 2.1 shows the influence of stringer flexibility and span length on transverse moments. The curves give moment per unit width produced by a dimensionless unit force, M/P. The stiffness parameter, *H*, is the ratio of stiffness $E_b I_b$ of the beam and slab stiffness for the span length, *L*.

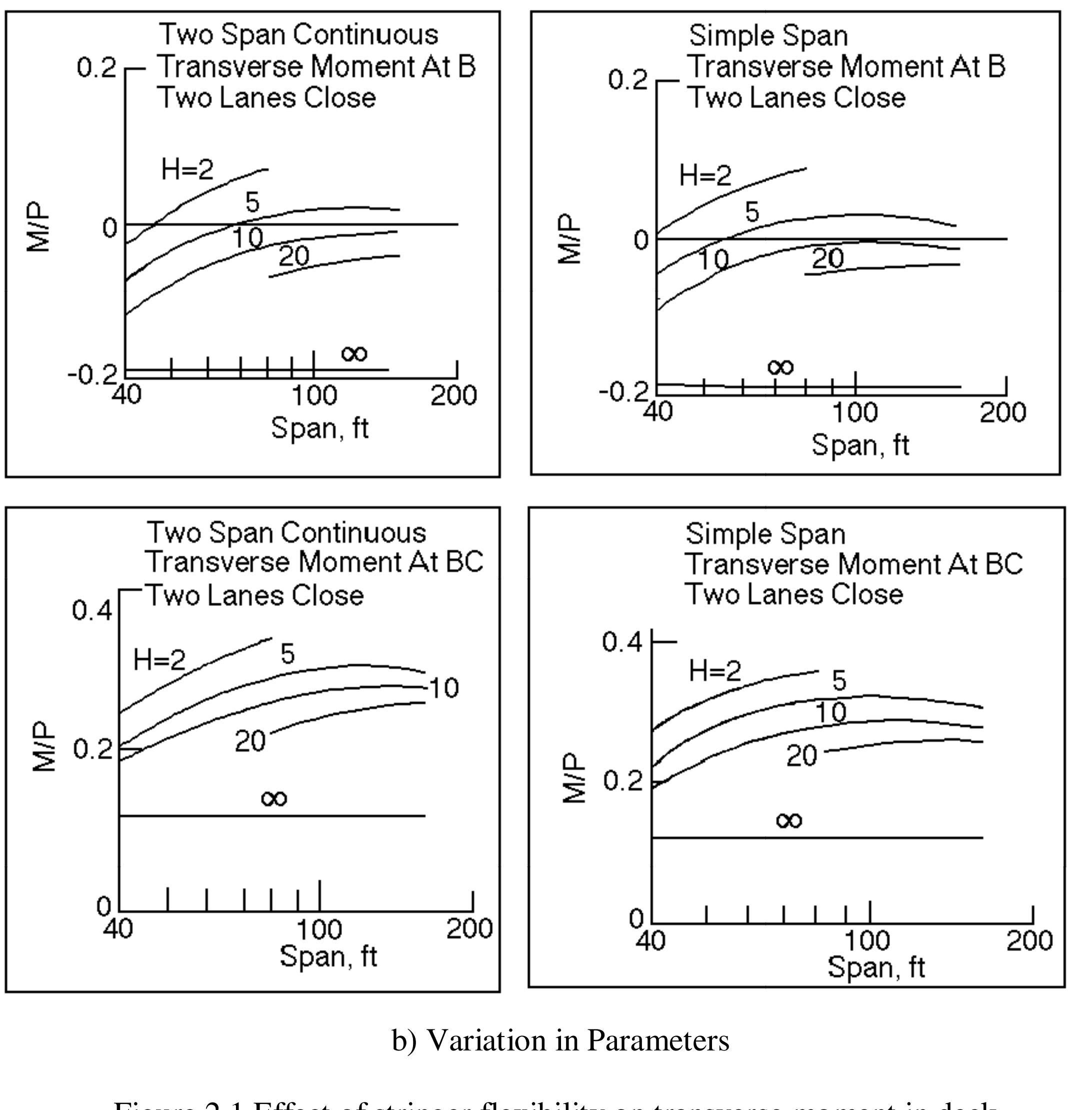
$$H = \frac{E_b\, I_b}{\dfrac{E\, L\, h^3}{12\,(1-\nu)^2}} \qquad \text{(Eqn. 2.1)}$$

In equation 2.1, *E, h*, and ν are the modulus of elasticity, thickness, and Poisson's ratio for the deck slab, respectively, and h and L are in like units. Flexible structures result in smaller values of H, and H is varied between 2, 5, 10, 20 and infinity (∞) in the figure, because this range includes practical extremes of flexibility and stiffness. Span lengths of 40, 80, and 160 ft (12.2, 24.4, and 48.8 m) for both simple and continuous span bridges are used. Figure 2.1b shows that low values of *H* (increased girder flexibility) increase the peak positive transverse moment in the deck. In turn, the peak negative live-load moments are decreased with increased flexibility, and this subsequently reduces deck cracking.

Nevels and Hixon (Nevels and Hixon, 1973) completed field measurements on 25 I-girder bridges to determine the causes of bridge deck deterioration. The total sample of 195 bridges consisted of simple and continuous plate girder and I-girder as well as prestressed concrete beams with span lengths ranging from 40 to 115 ft (12.2 to 35 m). The work showed no relationship between flexibility and deck deterioration.



a) Cross-section



b) Variation in Parameters

Figure 2.1 Effect of stringer flexibility on transverse moment in deck
(Wright and Walker, 1971)

An early PCA (PCA 1970) study provides substantial evidence that steel bridges and bridge flexibility have not greater tendency toward deck cracking damage than other bridge systems. However, another recent study (Dunker and Rabbat, 1990 and 1995) funded by PCA appears to contradict earlier PCA results (PCA 1970). This more recent study examines bridge performance on a purely statistical basis. No bridges are inspected. The condition assessment and the statistical evaluation are based entirely upon the National Bridge Inventory data. They show that steel bridges have greater damage levels than concrete bridges, and imply that this is caused by greater flexibility and deflection. There are several reasons for questioning this inference. First, the damage scale in the inventory data is very approximate, and the scale is not necessarily related to structural performance. Second, the age and bridge construction methods are not considered in the statistical evaluation. It is likely that the average age of the steel bridges is significantly older than the prestressed concrete bridges used for comparison. Therefore, any increased damage noted with steel bridges may be caused by greater wear and age and factors such as corrosion and deterioration. Finally, there are numerous other factors that affect the bridge inventory condition assessment. As a consequence, the results of this study must be viewed with caution.

The preponderance of the evidence indicates no association between bridge girder flexibility and poor bridge performance (ASCE, 1958; Wright and Walker, 1971; and Goodpasture and Goodwin, 1971). While the literature shows no evidence that bridge deck deterioration is caused by excessive bridge live-load deflections, other factors are known to influence bridge deck deterioration. High temperature, wind velocity, and low humidity during placement and curing accelerate cracking (Krauss and Rogalla, 1996). Further, the deck casting sequence has been found to have a significant effect on the deterioration of concrete at early ages (Issa, Mo., 1999; and Issa, Ma. et. al. 2000). Concrete material factors important in reducing early cracking include low shrinkage, low modulus of elasticity, high creep, low heat of hydration, and the use of shrinkage compensating cement. Variables in the design process that affect cracking include the size, placement and protective coating of reinforcement bars. Smaller diameter reinforcement, more closely spaced, is recommended to reduce cracking (Krauss and Rogalla, 1996; French, et. al. 1999). Increased deck reinforcement helps reduce cracking, but the reinforcement must have a sufficient cover, between 1 and 3 inches. However, a CALTRANS study reported placement as having no effect on transverse cracking (Poppe, 1981). In general, existing research provides little support for deflection limits as a method of controlling damage in bridges.

2.3. Effect of Bridge Deflection on Superstructure Bridge Vibration

There is considerable evidence that the existing deflection limits are motivated by vibration control, so research into bridge vibrations is relevant to this study.

2.3.1. Human Response to Vibration

Research (Nowak and Grouni, 1988) has shown that deflection and vibration criteria should be derived by considering human reaction to vibration rather than structural performance. The important parameters that effect human perception to vibration are the acceleration, deflection, and period (or frequency) of the response. Human reactions to vibrations are classified as either physiological or psychological. Psychological discomfort results from unexpected motion, but physiological discomfort results from a low frequency, high amplitude vibration such as seasickness. Vertical bridge acceleration is of primary concern, since it is associated with human comfort (Shahabadi, 1977).

In 1931, Reiher and Meister (Shahabadi, 1977) produced 6 tolerance ranges based on reactions of 25 adult subjects between the ages of 20 to 37 years. In a laboratory setting, subjects were exposed to sinusoidal movements in the vertical or horizontal directions for 10-minute periods. The tolerance ranges are classified as imperceptible, slightly perceptible, distinctly perceptible, strongly perceptible or annoying, unpleasant or disturbing, and very disturbing or injurious as shown in Fig. 2.2. Goldman (Goldman, 1948) reviewed the problem and produced from several different sources, including Reiher and Meister, a set of revised averaged curves corresponding to three tolerance levels classified as perceptible, unpleasant, and intolerable.

A 1957 study (Oehler, 1957) cites empirical amplitude limits developed by Janeway to control intolerable levels of vibration amplitude. Janeway's limits recommended that af^3 equal 2 for bridges with frequency of 1 to 6 cps where a is the amplitude and f is the frequency of vibration, and af^2 equal to 1/3 for higher bridge frequencies. Bridge deflection, vibration amplitude and frequency of vibration were measured for 34 spans of 15 bridges to determine which bridge type was more susceptible to excessive vibrations. Simple-spans, continuous spans, and cantilever spans of reinforced concrete, steel plate girder and rolled beam superstructures were investigated. The observed amplitude and frequency data was compared to Janeway's recommended limits. The amplitude of vibration is shown with the test truck on the span and off the bridge in Fig. 2.3. The test vehicle produced vibration amplitudes that exceeded Janeway's human comfort limits in 7 cantilever-span and 7 simple-span bridges, but this amplitude of vibration never lasted more than one or two cycles. Reactions from personnel performing the tests disagreed with the limits set by Janeway. They perceived the vibration on the simple and continuous spans but noted that it was not disturbing. They felt discomfort at high amplitude, low frequency vibration. It was concluded the cantilever spans were more prone to longer periods of vibration and larger amplitudes than the simple or continuous spans. Further, increasing bridge stiffness does not decrease the vibration amplitude sufficiently to remove it from the perceptible range presented by Reiher and Meister and Goldman (Oehler, 1970).

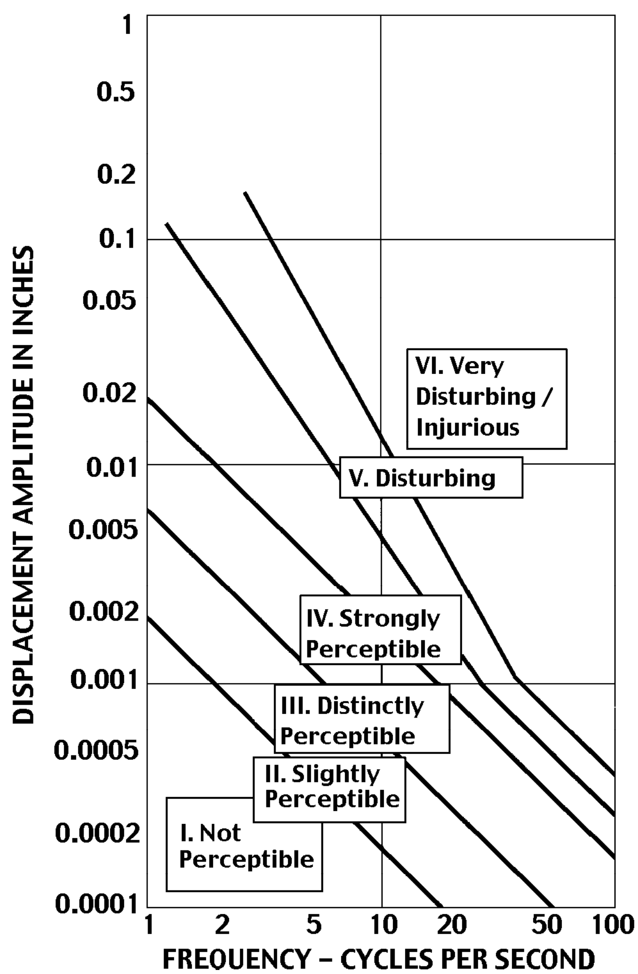


Figure 2.2. Six Human Tolerance Levels by Reiher and Meister (Shahabadi 1977)

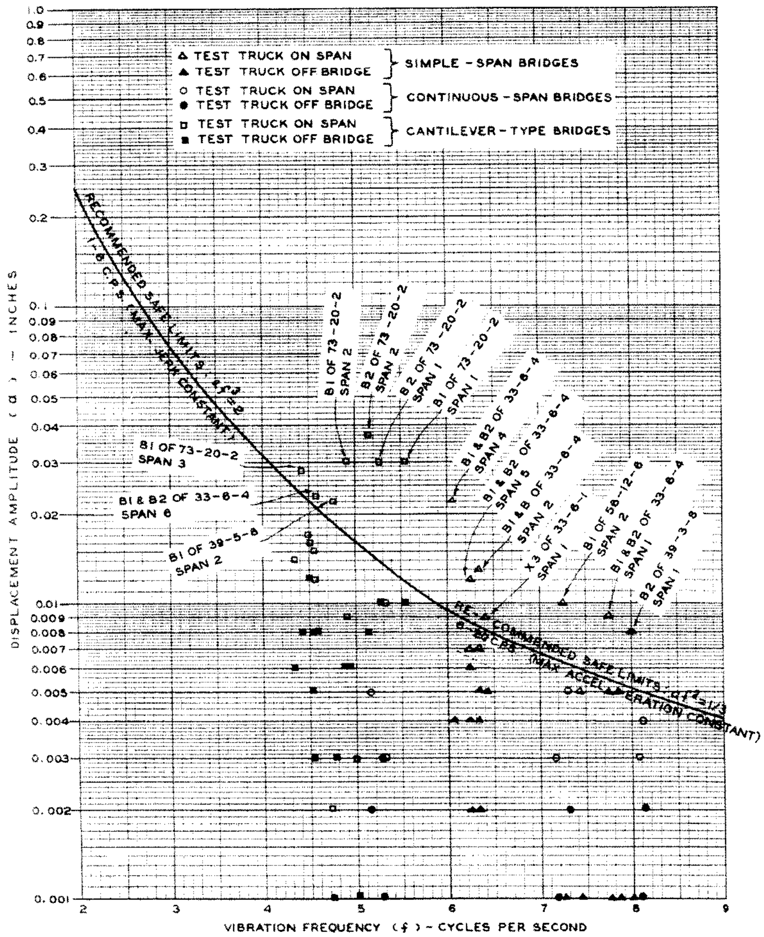


Figure 2.3. Observed Bridge Amplitude and Frequency with Human Tolerance Limits Developed by Janeway (Oehler, 1957)

Wright and Green (Wright and Green, 1964) compared the peak levels of vibration from 52 bridges to levels based on Reiher and Meister scale and Goldman's work. They showed that 25% of the bridges reached the intolerable level indicated by subjects in the Reiher, Meister, and Goldman's work. They concluded that low natural frequencies, up to 3 Hz, are not the only parameter that will reduce vibrations.

DeWolf and others (DeWolf, Kou, and Rose, 1986) conducted a field study on a four-span noncomposite continuous bridge with two nonprismatic steel plate girders. This 30-year old structure had reported objectionable vibrations, when one direction of traffic was stopped on the bridge while the other lane was moving. Accelerations were determined and compared to human tolerance limits developed by Bolt, Beranek and Newman, Inc. The maximum values recorded on the bridge, seen in Fig. 2.4, exceed those accelerations tolerable by most people. However, the bridge structural performance and the resulting stresses, based on the initial analysis of the data, are within acceptable limits.

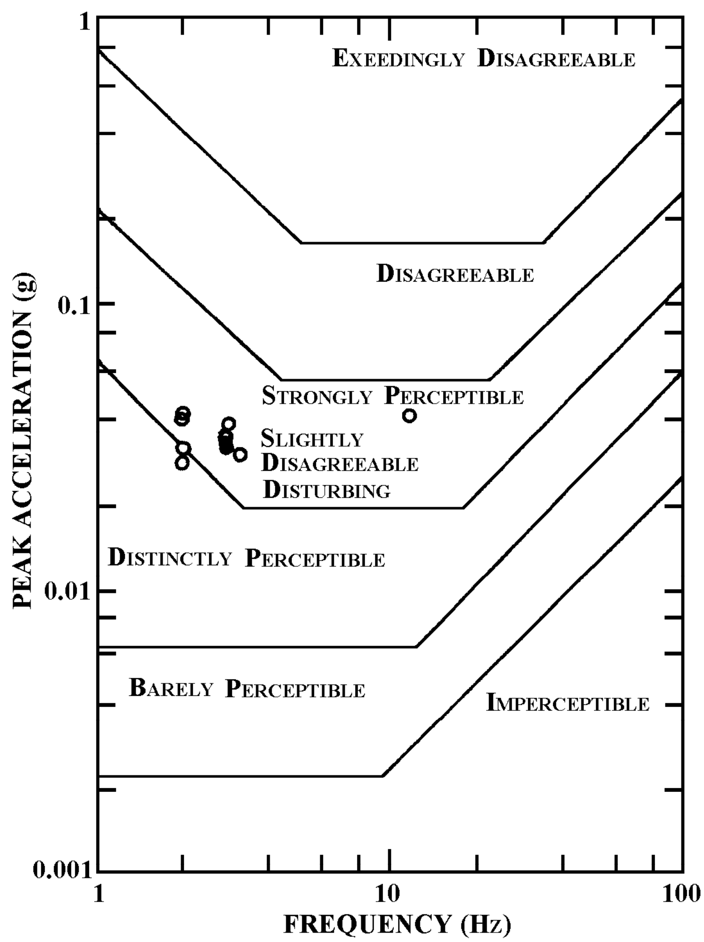


Figure 2.4. Measured Acceleration Compared to Human Tolerance Limits by Bolt, Baranek and Newman (DeWolf et al. 1986)

2.3.2. Field Studies

Many factors influence the dynamic behavior of bridges including the following:

- vehicle properties,

- bridge geometric and material properties, and
- vehicle / structure interaction

Many early dynamic studies [Biggs et al., 1959, Cantieni, 1983] were directed primarily toward development of impact factors and understanding bridge dynamic response. The dynamic response of the Jackson and the Fennville Bridges (Foster and Oehler, 1954) were monitored under normal commercial traffic, a controlled two-axle truck, and a special three-axle truck. The Jackson Bridge is an eight-span plate girder bridge with 5 simple and 3 continuous spans. The Fennville Bridge consists of 6simple-spans of rolled beam construction of which only one span exhibits composite action. Measured deflections were compared to theoretical predictions, and the effect of vehicle weight, vehicle type, axle arrangement, speed, and surface roughness on vibration was studied. Deck surface irregularities were simulated by boards placed on the bridge deck in the path of the test vehicle, and they caused increased amplitude of bridge vibration. Increasing span flexibility increased the observed amplitude and duration of vibration. Computed deflections were consistently larger than the measured deflections. Vibrations increased when the natural period of vibration of the span nearly coincided with the time interval between axles passing a reference point on the span.

Midspan deflections for all spans due to a 3-axle truck with axle loads of 5.6, 18.1, and 15.5 kips (24.9, 80.5, and 70 kN) were measured (Oehler, 1957) for 15 bridges built between 1947 and 1957. Several spans showed appreciable vibration although live-load plus impact deflections were less than $\frac{L}{1000}$. The dynamic behavior of 52 representative Ontario highway bridges that vibrate under normal traffic were measured (Wright and Green, 1964). Each bridge was inspected to determine traffic conditions, road surface condition and bridge details. A wide variety of differing types, spans and cross-sectional geometry were chosen including beam or plate girder and truss systems, simple and continuous spans. Span lengths ranged from 50 to 320 feet. In all cases, the actual stiffness of the bridge was larger than that of the calculated stiffness, and as a consequence the measured frequency was always larger than the computed frequency as shown in Fig. 2.5. One bridge was selected for further evaluation of the influence of surface roughness on the dynamic response. A test was performed on that bridge before the final asphalt pavement was laid and after the pavement was laid while normal traffic operated on the bridge under both cases. The deck couldn't be considered rough or smooth before the pavement was placed but was smooth immediately after the pavement was placed. Comparison of the results of the two tests showed great improvement in the dynamic performance with the smooth deck.

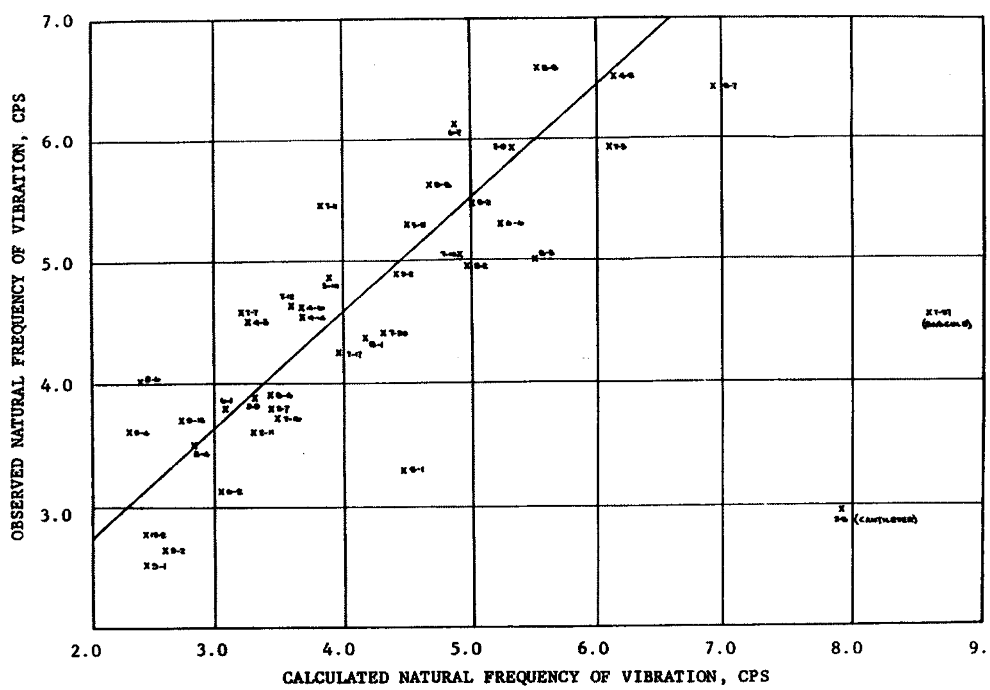


Figure. 2.5. Measured Bridge National Frequency Versus Calculated Natural Frequency
(Wrignt and Green, 1959)

Live-load deflections were measured (Nevels and Hixon, 1973) on 25 bridges with an HS20 vehicle, with wheel loadings of 7.29 and 32.36 kips (32.4 and 144 kN) and an axle spacing of 13.25 ft (4.03 m), and compared to calculated deflections. The calculated deflection was approximately 50 percent larger than the actual values.

Dynamic responses of 40 steel, 19 reinforced concrete and 3 pre-stressed concrete bridges were measured under normal traffic and loaded with a 21 kip (93.5 kN) test vehicle (Kropp, 1977). Measured frequencies compared roughly well with analytical predictions made for 900 of the more than 13,000 records accumulated during testing. Of the 900 records, 65 percent were normal trucks, 30 percent were the test vehicle and 5 percent were light traffic. Only 5% of the measured responses exceeded the comfort limit proposed by Wright and Walker (Wright and Walker, 1971).

Other field studies of dynamic response of typical bridge structures were carried out in Ontario, Canada (Green, 1977). For each structure, one of two dominant frequencies of vibration was generally observed for the free vibration. Many types of bridge geometry, ages, and conditions were included in the study. A relationship between observed frequency, f_{obs}, and calculated frequency, f_{cal} was determined.

$$f_{obs} = 0.95 f_{cal} + .072 \qquad \text{(Eqn. 2.2)}$$

where the frequency values are in Hz.

$$f_{cal} = \frac{\pi}{2\ L^2}\sqrt{\frac{E_b\ I_b\ g}{w}} \qquad \text{(Eqn. 2.3)}$$

where E_b is the modulus of elasticity of steel, I_b is the moment of inertial of the beam of cross-section, g is the acceleration due to gravity, and w is the weight per unit length of the stringer and its share of deck. Consistent units must be employed for all variables. This equation was validated for structures with 2 Hz < f_{cal} < 7 Hz.

Haslebacher [(Haslebacher, 1980)] measured deflections on steel superstructures, and suggested that intolerable dynamic conditions may result if the ratio of forcing frequency to bridge natural frequency is in the range of 0.5 to 1.5. He defined intolerable movements as those adversely affecting structural integrity or human perception. He notes that by choosing a critical value of forcing frequency and comparing this value to the natural frequency of the structure, the designer can determine if the structure has enough mass and stiffness to prevent excessive dynamic deflections.

Static deflections using present AASHTO Load Factor Specifications, natural frequencies and mode shapes were estimated and compared to field measurements for another bridge [(DeWolf, Rose, and Kou, 1986)]. Twenty-three test runs were completed with 2-axle dump trucks that weighed 30.52 and 36.4 kips (135.8 and 162 kN). The maximum determined deflection of 0.64 in (16.2 mm) was approximately 25 percent of the AASHTO limit, but the bridge had unacceptable vibrations at that load level.

Field vibration analyses on 17 steel girder, 6 reinforced concrete slab, and 2 reinforced concrete box-girder bridge spans were performed [(Dusseau, 1996)]. Accelerometers were used to measure ambient vibration, and a spectrum analyzer was used to determine the fundamental natural frequency for the 25 bridges. Calculated bridge natural frequency compared very well with the measured frequency.

2.3.3. Analytical Studies

Finite element studies of representative noncomposite simple span and continuous multi-girder bridges investigated the effects of bridge span length and stiffness, deck surface roughness, axle spacing and number of axles on bridge acceleration [(Amaraks, 1975)].

Surface roughness produced the most significant effect on acceleration for both the simple and continuous span bridges. The maximum accelerations with a rough roadway surface were found to be as much as five times those for the same bridge with a smooth deck. Maximum accelerations increased as the span length decreased. Maximum acceleration also increased when the stiffness was reduced, but this increase was significantly less severe than noted with the surface roughness variations as may be seen in Fig. 2.6 and 2.7, respectively. Aramraks observed that vehicle speed greatly influences peak acceleration. The maximum accelerations were approximately the same for two and three axle vehicle models, but were about two thirds of the magnitudes produced by the single axle vehicle model. An investigation of the influence of initial oscillation of the vehicle suspension on bridge acceleration was also conducted. Initial oscillation causes a 30 to 50 percent increase in maximum accelerations for the bridge assumed to have a smooth deck surface.

Aramraks [(Aramraks, 1975)] evaluated maximum accelerations for varied ratios of bridge natural frequency to vehicle frequency, in the range of 0.5 to 2.0, as can be seen in Fig. 2.8. The vehicle frequency, using an HS20-44 loading, is the tire frequency of the rear axles. For the two-span bridge and three-span bridge, the fundamental natural frequency is 3.53 and 3.0 Hz, respectively. Commonly, the acceleration magnitudes were approximately the same but increased slightly in the midspan when the vehicle and bridge had the same natural frequency.

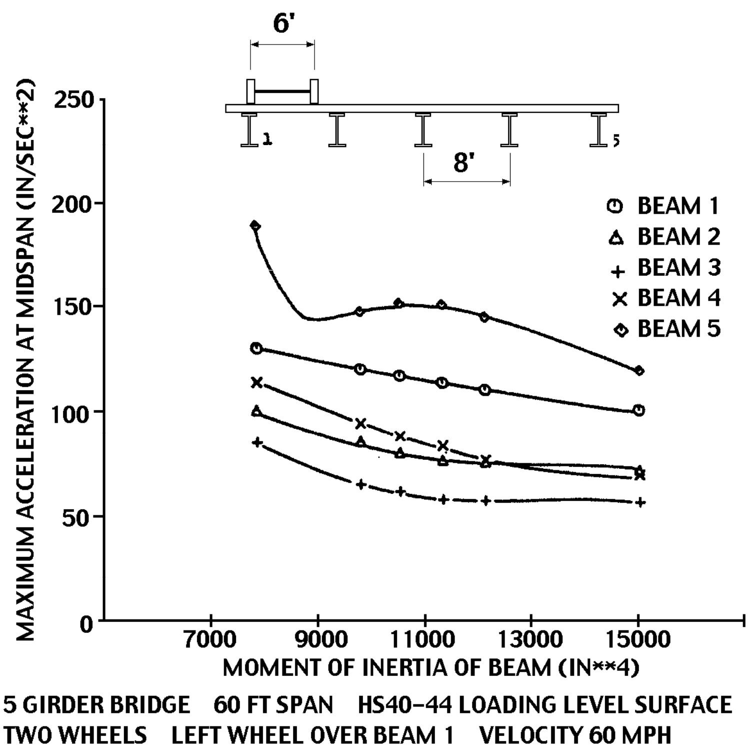


Figure 2.6. Effect of Flexibility on Acceleration (Aramraks, 1975)

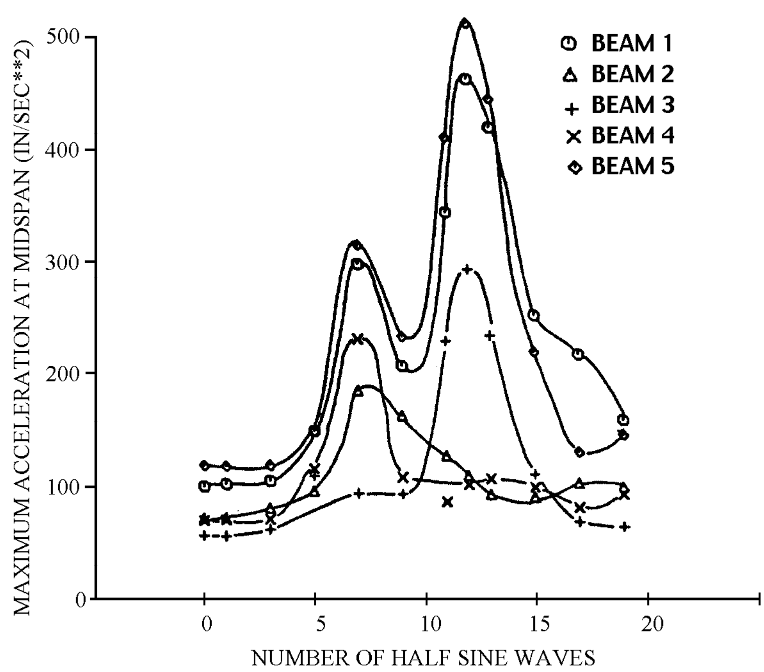


Figure 2.7. Effect of Surface Roughness on Acceleration (Aramraks, 1975)

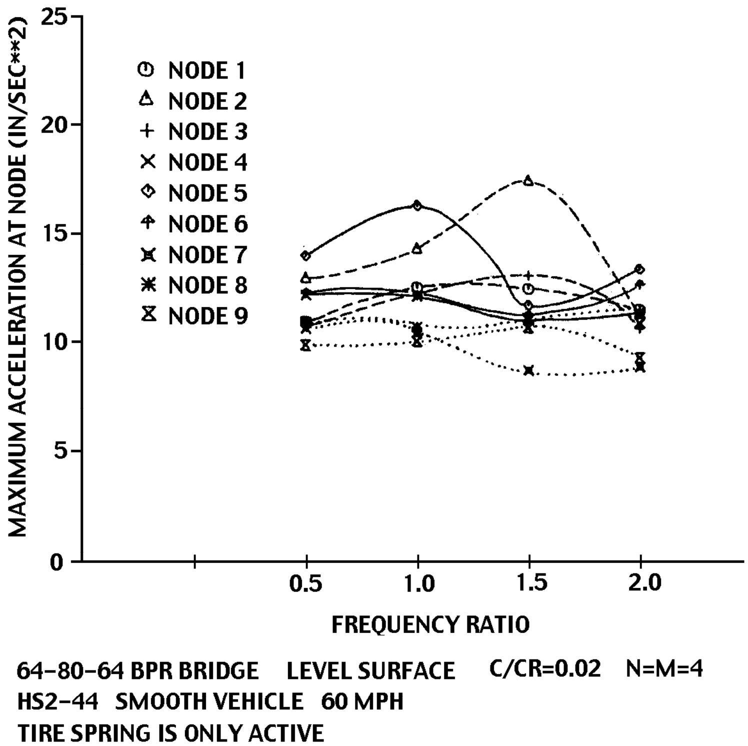


Figure 2.8. Effect of Frequency Ratio on Acceleration (Aramraks, 1975)

Another study (DeWolf and Kou, 1997) demonstrated the influence of the vehicle speed, vehicle weight, bridge surface roughness, initial vehicle oscillation, deck thickness and girder flexibility using a three-dimensional finite element model. The bridge was previously monitored in the field (DeWolf, Kou, and Rose, 1986) , and it was a composite continuous four-span bridge with nonprismatic steel plate girders. They found that maximum displacement in different spans changed by only 5 to 12 percent but maximum acceleration increased by 50 to 75 percent when road surface roughness changed from smooth to one inch surface roughness amplitude. They found only minor influence of girder flexibility on overall dynamic bridge behavior. The maximum displacement increased with increased vehicle speed. The increase was up to 40 percent in extreme cases. However, vehicle speed was found to have the greatest effect on the maximum girder acceleration. Additionally, they showed that initial vehicle oscillation had the greatest effect on maximum deflections increasing 2.5 times, while the maximum girder acceleration showed a minimal increase with an increase in oscillations.

2.4. Alternate Live-Load Deflection Serviceability Criteria

Three alternative methods of providing for the serviceability limit state are found and discussed here.

2.4.1. Canadian Standards and Ontario Highway Bridge Code

Both the Canadian Standard and the Ontario Highway Bridge Code use a relationship between natural frequency and maximum superstructure static deflection to evaluate the acceptability of a bridge design for the anticipated degree of pedestrian use (Ontario Ministry of Transportation, 1991; and Canadian Standards, 1988). Figure 2.9 shows the plot of the first flexural frequency (Hz) versus static deflection (mm) at the edge of the bridge, which the natural frequency is calculated using Eqn. 2.2 (Ontario Ministry of Transportation, 1991). The superstructure deflection limits are based on human perception to vibration.

Three types of pedestrian use of highway bridge are considered for serviceability:

- very occasional use by pedestrians or maintenance personnel of bridges without sidewalks,
- infrequent pedestrian use (generally do not stop) of bridges with sidewalks, and
- frequent use by pedestrians who may be walking or standing on bridges with sidewalks.

This relationship was developed from extensive field data collection and analytical models conducted by Wright and Green in 1964. For highway bridges, acceleration limits were converted to equivalent static deflection limits to simplify the design process. For pedestrian traffic, the deflection limit applies at the center of the sidewalk or at the inside face of the barrier wall or railing for bridges with no sidewalk.

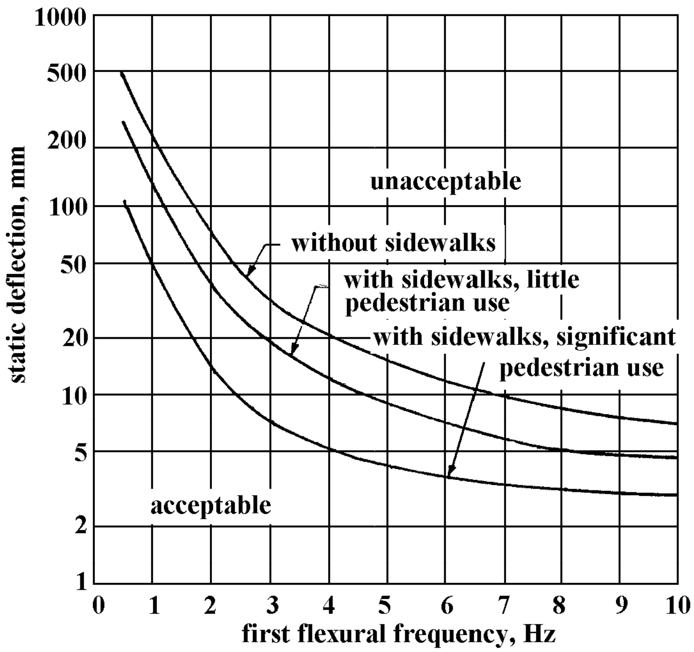


Figure 2.9. First Flexural Frequency versus Static Deflection (Ministry of Transportation, 1991)

More recent studies by Billings conducted over a wide range of bridge types and vehicle loads, loads ranging from 22.5 kip to 135 kips (100 KN to 600 KN), confirm the results of the initial study (Ontario Ministry of Transportation, 1991).

For both the Canadian Standards and the Ontario Code, only one truck is placed at the center of a single traveled lane and the lane load is not considered. The maximum deflection is computed due to factored highway live-load including the dynamic load allowance, and the gross moment of inertia of the cross-sectional area is used (i.e. for composite members, use the actual slab width). For slab-and-girder construction, deflection due to flexure is computed at the closest girder to the specified location if the girder is within 1.5m of that location.

2.4.2. Codes and Specifications of Other Countries

A brief review of the codes and specifications used in other countries were also examined. Most European Common Market countries base their design specifications upon the Eurocodes (Dorka, 2001). The Eurocodes are only a framework for national standards. Each country must issue a "national application document (NAD)" which specifies the details of their procedures. A Eurocode becomes a design standard only in connection with the respective NAD. Thus, there is considerable variation in the design specifics from country to country in Europe. If an NAD exists for a specific Eurocode, then this design standard is enforced when it is applied to a building or bridge. Often, the old national standards are also still valid and are applied. There is the rule though, that the designer cannot mix specifications. The designer must make an initial choice and then use this in all design documents for the structure. However, in general, the full live-loads are factored with a "vibration factor" to account for extra stresses due to vibrations in European bridge codes. No additonal checks (frequency, displacements etc.) are then required. For long span or slender pedestrian bridges, a frequency and mode shape analysis also is usually performed. Special attention is always paid to cables, since vibrations are common, and some European bridges have problems with wind induced cable vibration. Deflection limits are not normally applied in European bridge design.

In New Zealand, the 1994 Transit NZ Bridge Manual limits the maximum vertical velocity to 0.055 m/s (2.2 in/sec) under two 120 kN (27 kip) axles of one HN unit if the bridge carries significant pedestrian traffic or where cars are likely to be stationary (Walpole, 2001). Older versions of this Bridge Manual also employed limits on $\frac{L}{D}$ and deflection, but these are no longer used in design.

2.4.3. Wright and Walker Study

A 1971 study conducted by the American Iron and Steel Institute (AISI) reviewed AASHTO criteria and recommended relaxed design limits based on vertical acceleration to control bridge vibrations (Wright and Walker, 1971). The proposed criteria requires that:

1. Static deflection, δ_s, is the deflection as a result of live-loads, with a wheel load distribution factor of 0.7, on one stringer acting with its share of the deck.

2. Natural frequency, f_b (cps), is computed for simple or equal spans

$$f_b = \frac{\pi}{2\ L^2}\sqrt{\frac{E_b\ I_b\ g}{w}} \qquad \text{(Eqn. 2.5)}$$

3. The speed parameter, α, is determined by

$$\alpha = \frac{v}{2\ f_b\ L} \qquad \text{(Eqn. 2.6)}$$

where,

v = vehicle speed, fps.

4. The Impact Factor, *DI*, is determined as

$$DI = \alpha + 0.15 \qquad \text{(Eqn. 2.7)}$$

5. Dynamic Component of Acceleration, a (in/sec^2)

$$a = DI\ \delta_s\ (2\,\pi\, f_b)^2 \qquad \text{(Eqn. 2.8)}$$

6. Acceleration limit must not exceed the limit

a = 100 in./sec^2

7. If the Dynamic Component of Acceleration exceeds the acceleration limit, a redesign is needed.

2.5. Summary

The specification requires that deflections be controlled by limiting span-to-depth ratio and by limiting the maximum unfactored deflection to:

- $\frac{L}{800}$ for most design situations
- $\frac{L}{1000}$ for urban areas where the structure may be used in part by pedestrian traffic

where *L* is the span length of the girder.

The justification for the existing AASHTO deflection limits are not clearly defined in the literature, but the best available information indicates that they initiated as a method of controlling undesirable bridge vibration. The limits are based on undetermined loads, and the bridges used for this initial limit state development are very different from those used today. The research has shown that reduced bridge deflections and increased bridge stiffness will reduce bridge vibrations, but this is clearly not the best way to control bridge vibration. Bridge vibration concerns are largely based upon human perception. Human perception of vibration depends upon a combination of maximum deflection, maximum acceleration and frequency of response. Several models have been proposed for establishing acceptable limits for perception of vibration, but there does not appear to be a consensus regarding acceptable limits at this point. Bridge surface roughness and vehicle speed interact with the dynamic characteristics of the vehicle and the bridge (such as natural frequency) to influence the magnitude of bridge response. Field measurements of bridges show that the actual bridge live-load deflections are often smaller than computed values for a given truck weight.

Initial vehicle suspension oscillation tends to significantly increase bridge accelerations and displacements. As the ratio of natural frequency of the bridge to the natural frequency of the vehicle suspension approach unity (i.e. a resonant condition), the bridge response increases. Various estimates on the fundamental frequency for slab on girder bridges range from 1 to 10 Hz, but vehicle natural frequency has been estimated between 2 to 5 Hz (typically closer to the lower value).

Past research shows no evidence that bridge live-load deflections cause significant damage to bridge decks. In general, the strain in bridge decks due to normal bridge flexure is quite small, and damage is unlikely to occur under these conditions. On the contrary, other attributes such as quality and material characteristics of concrete clearly influence deck deterioration and reduced deck life. Past research has relatively little consideration to the possibility that large bridge deflections cause other types of bridge structural damage. Furthermore, local deformations may well cause structural damage, but the $\frac{L}{800}$ deflection limit is not typically applied in such a way to control this damage.

Within this framework, it is not surprising that the bridge design specifications of other countries do not commonly employ deflection limits. Instead vibration control is often achieved through a relationship between natural bridge frequency, acceleration and live-load deflection.

Chapter 3

Survey of Professional Practice

3.1. Description of the Survey

A survey was completed to better understand the professional practice with regard to the bridge deflection limit. The survey was completed by telephone and was directed toward bridge engineers from the 50 states. The survey sought specific information about the application of deflection limits for steel bridges in that state. The survey interview started with a brief statement of the goals of the research project, and requested that the bridge engineer answer a series of questions or nominate someone who is well suited to address the relevant issues.

Upon starting the survey, general information about the affiliation and title of the interviewee was obtained. The survey then consisted of 10 general questions. Depending upon the response to a question, any one general question potentially led to prepared follow-up questions that were needed to fully define the response. The first general question established the deflection limits that are applied to steel bridges in that state and the circumstances under which they are used. The second general question determined the loads used to compute these deflections for steel-stringer bridges, and the third question extended this information to other steel bridge types. The fourth question determined the calculation methods and the stiffness considered in the deflection calculation. Deflection limits and span-to-depth ratio ($\frac{L}{D}$ ratio) limits appear to accomplish similar objectives in deflection control, and question 5 addressed the role of the $\frac{L}{D}$ ratio limits in that state.

Questions 6 through 9 identified candidate bridges for more detailed study that was to be completed in later stages of the research. The economy of HPS bridges may be adversely affected by the existing deflection limit, and question 6 sought information on HPS applications. The seventh question identified bridges with structural damage that engineers attributed to excessive bridge deflections. Question 8 sought information regarding deflection serviceability resulting from live-load induced vibrations. Bridges that fail to satisfy the existing deflection limit but still provide good bridge performance are also strong candidates for further study, because these bridges provide a basis for modifying present serviceability limits. Question 9 identified these bridges.

Question 10 sought comments on the use and suitability of present live-load deflection limits and research reports or other information that was relevant to the study. Field measurements and research reports related to this study were requested.

3.2. Results of Survey

Phone calls were made to bridge engineers in all 50 states, and 48 valid responses were obtained. Only 47 responses are discussed here, because one state indicated that they had not designed a steel bridge in more than 30 years and had no position on steel bridge deflection issues. The survey and details of the state by state responses to the survey are provided in Appendix A.

The AASHTO Standard Specification limits the maximum live-load deflection to $\frac{L}{800}$ for steel bridges, which do not carry pedestrians, but the survey shows that there is wide variation in the deflection limit employed by the various states. Of 47 states reporting deflection limits for bridges without pedestrian access -

- 1 state employs a $\frac{L}{1600}$ limit,
- 1 state uses a $\frac{L}{1100}$ limit,
- 5 states employ a $\frac{L}{1000}$ limit,
- 1 state expresses a preference for $\frac{L}{1000}$ but requires $\frac{L}{800}$ limit, and
- 39 states employ a $\frac{L}{800}$ limit.

Of the states reporting deflection limits for bridges with pedestrian access -

- 1 state employs a $\frac{L}{1600}$ limit,

- 2 states use a $\frac{L}{1200}$ limit,
- 1 state employs a $\frac{L}{1100}$ limit,
- 39 states use a $\frac{L}{1000}$ limit,
- 3 states employ a $\frac{L}{800}$ limit.

There is very wide variation in these deflection limits, since the largest deflection limit is twice as large as the smallest deflection limit. Two of the 47 states treat the deflection limit as a recommendation rather than a design requirement.

The AASHTO Specification indicates that deflections due to live-load plus impact are to be limited by the deflection limit. Within this context, there is ambiguity in the loads and load combinations that should be used for the deflection calculations, because design live-loads are expressed as both individual truck loads and uniform lane loads. The survey showed that the loads used to compute these deflections have even greater variability than observed in the deflection limits.

- 1 state employs the HS (or in some cases LRFD HL) truck load only,
- 16 states use the truck load plus impact,
- 1 state uses distributed lane load plus impact,
- 1 state uses truck load plus distributed lane load without impact,
- 7 states use the larger deflection caused by either truck load plus impact or the distributed lane load with impact,
- 17 states use truck load plus distributed lane load plus impact, and
- 4 states consider deflections due to some form of military or special permit vehicle.

The combination of the variability of the load and the variability of the deflection limit results in considerable difficulty in directly comparing the various state deflection limits. For example, Wisconsin uses the smallest deflection limit, but it also employs smaller loads than most other states. However, the relative importance of the lane load and design truck load are likely to be different for long and short span bridges, and so the $\frac{L}{1600}$ limit used in Wisconsin may be more restrictive for short span bridges. Conversely, the Wisconsin limit may be a generous deflection limit for very long span bridges, because the truck load becomes relatively smaller with longer bridge spans despite the small deflection limit.

The actual methods used to calculate deflections are not defined in the AASHTO Specification. In typical engineering practice, deflection limits are based upon deflections caused by service loads under actual service conditions. Load factors or other factors used to arbitrarily increase design loads are not normally used in these deflection calculations, and the actual expected stiffness of the full structure is used. The survey shows that this is a further source of variability in the application of the deflection limits. Load factors and lane load distribution factors are employed in some states while they are neglected in others. Lane load distribution factors can significantly affect the magnitude of the loads used to compute the deflections. The survey shows that 26 states use lane load distribution factors from the AASHTO Standard Specifications in calculating these deflections. Three states report that they use the LRFD lane load distribution factors. Thirteen states indicate that they effectively apply the loads uniformly to the traffic lanes by the AASHTO multiple presence lane load rules. They then compute the deflections of the bridge as a system without any increase for load factors, girder spacing or lane load distribution. These states effectively use an equal distribution of deflection principle. One state uses its own lane load distribution factor that is comparable to system deflection calculations. Several states indicate some flexibility in the calculation method, and a few states indicate a reluctance to permit the bridge deflection limit to control the design. The effect of the lane load distribution factor can be quite significant. Depending upon the spacing of bridge girders, the load used for bridge deflection calculations can be 40% to 100% larger than the load used for states where deflections are computed for the bridge as a system or where the loads are uniformly distributed to girders.

Load factors may also be an issue of concern. Five states report that they apply load factors to the load used for the deflection calculation. These load factors also increase the loads used to compute bridge deflections, and they increase the variability in the application of the deflection limit between different states.

Span-to-depth, $\frac{L}{D}$, ratio limits were also examined because they also have interrelation with deflection limits. Seven states indicate that they employ no $\frac{L}{D}$ limits, while 34 indicate that they use the AASHTO design limits. Of these 34 states, 6 indicate that they strictly employ the limit, but 8 indicate that they employ it only as a guideline. The impact of this observation is not immediately clear, because some states that have no limit or a loose $\frac{L}{D}$ limit have relatively tight deflection limits. Some states that strictly apply the AASHTO $\frac{L}{D}$ ratio limits have relatively less restrictive deflection limits.

The combined variability of the deflection limit, the methods of calculating deflections, and the loads used to calculate deflection indicates that the resulting variability of the practical deflection limits used in the different states are huge. On the surface, it appears that variations of at least 200% to 300% are possible. However, the comparison is neither simple nor precise.

3.3. Bridges for Further Study

The survey identified a number of bridges that serve as candidate bridges for further analysis. These candidate bridges fall into one of 4 basic categories including:

- Bridges experiencing structural damage associated with large deflections,
- Bridges having passenger or pedestrian discomfort due to vibration,
- Bridges constructed of HPS steel, and
- Bridges failing existing deflection limits but still providing good performance.

Very few bridges that fail existing deflection limits but still provide good structural performance were identified in this survey. A small number of bridges with vibration problems was also identified. A number of HPS bridges were identified and information regarding these bridges was obtained for possible further evaluation. The identification of bridges with structural damage that is caused by bridge deflection provided somewhat confusing results. A number of damaged bridges were identified, but most state bridge engineers did not believe that they had any bridges with damage due to excessive deflections. A few states were very clear that they had a significant number of bridges with structural damage that was apparently associated with large deflections. This damage was usually deck cracking and steel cracking or other damage due to differential deflection and out-of-plane bending. However, some of the damage relates to cracking of bolts or other steel elements. It must be emphasized that even states reporting damage note that the damaged bridges were a small minority of their total inventory.

Nevertheless, the fact that some engineers felt that they had a significant number of bridges with the reported damage, while others felt that they had absolutely none was a source of concern. This contradiction may mean that some states have much better bridge performance than other state, or it may indicate that bridge engineers may have widely disparate views as to what constitutes bridge damage. As a result, a limited follow-up survey was directed toward maintenance and inspection engineers to better understand and address these results. This survey was limited to 11 states. The states were selected to represent all geographical parts of the United States, to include populous and lightly populated states, and to include states with a wide range of vehicle load limits. The selected states were -

California	Florida
Illinois	Michigan
Montana	New York
Pennsylvania	Tennessee

Texas Washington

Wyoming

The results of this follow-up survey showed that the contradictions in reported bridge behavior are caused by differences in engineer perspective, and there are not likely to be significant differences in bridge performance from state to state. Most state bridge engineers are intimately involved in the design and construction of new highway bridges, but they have limited contact with the repair, maintenance and day to day performance of most of the bridges in their inventory. Maintenance and inspection engineers often have a different perspective of bridge performance than the design engineers for their state. They note a significant number of bridges with cracked steel and cracked concrete decks, and they are more conscience of the potential causes of this damage. As a result, a number of damaged bridges were identified from a number of different states, and the damage of these bridges is usually attributable to some form of bridge deflection. However, none of this deflection damage can be attributed to the direct deflections that are evaluated in the AASHTO deflection check. Instead the damage is caused by differential deflections or relative deflections and other forms of local deformation. As a result, a significant number of candidate bridges were located for this category, it must be clearly recognized that the damage noted in those bridges is often different than what some engineers would regard as bridge deflection damage.

Bridges that were identified as viable candidates by the above criteria were investigated in much greater detail. Design drawings, inspection reports, and photographs were obtained for these candidate bridges, and this information was used for the bridge analysis discussed in Chapter 5.

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Chapter 4

Evaluation of the Variation in Practice

4.1. Introduction and Purpose

The survey results in Chapter 3 showed considerable variation among states in the application of the AASHTO deflection limits. The variation was caused by the use of different deflection limits, different loads used for deflection calculations, changes to these loads through load factors and lane load distribution factors, and different methods used for the calculation of deflections. The primary objective of this chapter is to present the results of a parameter study focused on examining the influence of variations between key design variables and between various deflection limits employed by different transportation departments.

A specialized computer program was developed for this purpose. The program was used to determine the maximum relative moment of inertia, I_{rel}, required to satisfy various state live-load deflection limit criteria. The conservatism of each deflection limit criteria can then be determined based on comparison of the I_{rel} values. The structural stiffness matrix approach for beam elements was used for the analysis procedure.

The deflection limit criteria depends upon the load geometry, the load magnitude, and the applied deflection limit. Four basic types of load patterns were examined:

- a two-axle truck as in the Standard AASHTO H truck loading.
- a three axle truck as in the Standard AASHTO HS or AASHTO LRFD HL truck loading,
- a distributed lane load, and
- combinations of truck loading and distributed lane loading.

The AASHTO HS and LRFD HL loads are combined, because the geometry and magnitude of these loads are similar. The AASHTO H truck loading is not discussed here, because it provides little added insight into the deflection issue.

4.2. Program Operation

Since many highly repetitive calculations that are not well suited to standard structural analysis computer packages were required, a special computer program was developed for these evaluations. The goal was to determine the relative stiffness (EI_{rel}) required to meet the various deflection limits. Beams were analyzed using a constant moment of inertia, I. While it is recognized that most I-girder bridges have variable I value due to flange transitions and other geometric effects, it is not feasible to incorporate such variations in a parameter study.

SAP 2000 Non-linear (Wilson and Habibullah, 2001) was used to check the program used for this study. Built in truck loading and influence line values were used in SAP to insure that the study program was in fact finding the points of maximum influence, and models were set up in SAP and manually run in order to check moment diagrams and deflections. The results were checked for each load and bridge geometry with the 100 ft (30.5 m) span length. In all checks the deflection vs. span length values calculated in SAP 2000 were within 0.5 % of the values calculated by the program developed in this research. Both the SAP model and the developed computer mode employed a 1 ft (.305 m) element discretization for this verification.

The computer program operates in two steps. The first step uses a courser finite element mesh to determine the approximate points of maximum influence in each span of the bridge structure. To do this the program moves a unit point load along the length of the bridge. As the point load is moved the program creates a simple structural model with one beam element in each unloaded span and two beam elements in the loaded span in which the point load is occurring. The loads were advanced in 1 ft (.305 m) increments. The computed deflection for each load point was recorded, and the program ultimately finds the location in each span in which the deflection is greatest.

The second step used the points of maximum influence to apply the appropriate loading to the same structure with a more refined mesh. The refined mesh permitted accurate determination of the moment diagram and deflected shape of the structure, since these were needed to establish the minimum possible moment of inertia required to resist the loading and pass the various deflection vs. span length, $\frac{\delta}{L}$, check. For this second step, a structural stiffness model using a 1 ft. (.305 m) element discretization was assembled. Boundary conditions were applied at the member ends and supports. The load geometry is selected, the load is applied and deflections are computed for each nodal point along the bridge length. The axles for the truck loading are spaced at a constant 14 feet and the centroid of the truck load is placed at approximately the point of maximum influence allowing the axle loads to be placed at the nearest nodes in the structure. This is done for each span separately and the deflections and bending moments were calculated for the entire structure due to loading in that span. For HS truck loading the axle loads had ratios of 0.2, 0.8, and 0.8. This resulted in a total unit load of 1.8. This is done so that HS truck loading can be directly compared by multiplying the deflections by the gross weight of the front two axles. For example, the HS20-44 loading can be compared simply by multiplying the deflection by 40 kips (178 kN).

For distributed lane loading, loading is only applied in spans where it will increase the deflection in the span of interest. The lane loading is applied using equivalent nodal loads at all appropriate nodes and the magnitude is also scaled to permit direct comparison of uniform lane loading and truck loading. Standard HS20-44 lane loads are 0.640 kip/ft (9.34 kN/m) and in the program the lane load has been scaled so that it is $\frac{1}{62.5}$. If the deflection results are multiplied by 40 kips (178 kN), the resulting lane load magnitude would be 0.640 kip/ft (9.34 kN/m), and the resulting deflections will be the same as those for HS20-44 lane loading. The combination of uniform lane loading and the truck loading simply combines the truck and lane loading using the same scaling factors and load positions noted above.

The deflected shape and bending moment diagram are calculated for the maximum influence in each span, and the ratio of the maximum deflection to the span length is established. For simple span beams, the span length, L, is determined by taking the distance between supports, and the maximum deflection is the maximum deflection of beam span relative to the points of zero moment. If a consistent and comparable measure is employed for continuous multiple span beams, the span length for the deflection comparison should be taken as the distance between any points of contraflexure as illustrated in Fig. 4.1. For continuous bridge girders, the maximum deflection should also be determined by taking the maximum deflection measured from the chord joining the zero moment points as shown in the figure.

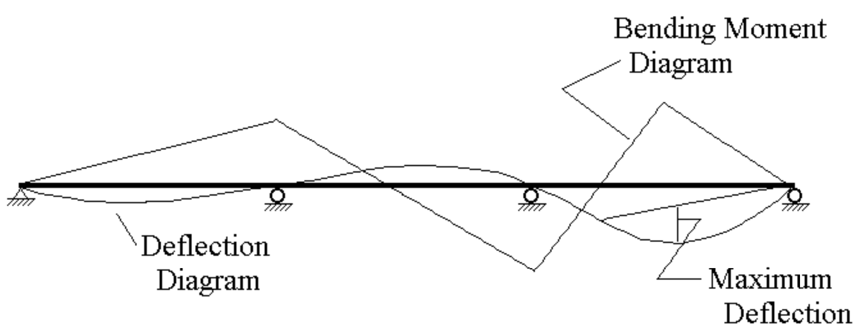

Figure 4.1. Geometry for Deflection Check of Multiple Span Beams

Once the program has calculated the maximum deflection vs. span length the inherent linearity of the structural system can be used to calculate properties of interest. The system is modeled by using a stiffness matrix approach. Since the computer program in this study incorporates a constant beam stiffness the matrix formulation may be written as follows.

$$\{P\}p = EI\,[K]\,\{U\} \qquad \text{(Eqn. 4.1)}$$

The parameter, p, represents the load magnitude (in kips), and the load vector, {P}, provides the load pattern. The column matrix or vector, {P}, is assembled using the methods described earlier. The bending stiffness of the beam, EI, is a constant, and E= 29,000 ksi (201,500 MPa), and [K] is the stiffness matrix. The system of equations can then be solved by normal matrix inversion or solution techniques, and the deflection vector, $\{U\}$, is determined by

$$\{U\} = \{\Phi\}\frac{p}{E\,I} \qquad \text{(Eqn. 4.2)}$$

The vector, $\{\Phi\}$, is the deflected shape of the girder resulting from a unit load, p, and EI. The maximum deflection, δ, is then

$$\delta = \frac{\phi\,p}{E\,I} \qquad \text{(Eqn. 4.3)}$$

where, ϕ, is the maximum value of the shape vector. The deflection is limited by a ratio, R, which is a deflection limit such as $\frac{L}{800}$. Therefore, the relative stiffness, I_{rel}, may be computed as follows

$$I_{rel} = I_{base}\frac{p\,DF\,IF}{R} \geq \frac{\phi\,p}{29000\,R}\,DF\,IF. \qquad \text{(Eqn. 4.4)}$$

where IF is an impact factor, I_{base} is the base moment of inertia, and DF is the lane load distribution factor used in the analysis. I_{base} is the moment inertia required when R, p, DF, and IF all equal to 1. It should be noted that several states include load factors in their bridge deflection evaluation, and if load factors are used they may be incorporated in the right hand side of Eqn. 4.4. However, load factors are not normally considered in deflection limit calculations and are not included in this parameter study. I_{rel} can be calculated for any magnitude of loading or $\frac{\delta}{L}$ limit. I_{rel} represents the minimum possible moment of inertia required in order to satisfy a specific deflection vs. span length value under a specific load geometry and magnitude.

4.3. Application of the Deflection Limits

The load vector, $\{P\}$, considers the load geometry or pattern, and for comparison in this report they are categorized as:

Category A. HS or HL Truck Loading

Category B. Lane Loading

Category C. Combination HS or HL Truck Loading and Lane Loading

There are a series of sub-categories within each of these main categories that differ only in the eventual magnitude of the applied load and deflection limit. This categorization reduces the number of analyses required for the evaluation, and it permits more direct comparison of some parameter effects. The analyses were completed for four main bridge span types:

- simply supported,
- two-span continuous with equal spans,
- three-span continuous with equal spans, and
- three-span continuous with outside spans equal to 80 percent of the center span.

The nominal spans were varied from 50 ft to 300 ft (15.24 m to 91.44 m) in 50 ft (15.24 m) increments. For each analysis, I_{base} was obtained assuming an elastic modulus of 29000 ksi (201,500 MPa) and a R, p, DF, and IF equal to 1.0. The resulting value is in units of in^4 / kip.

Figure 4.2, 4.3, 4.4, and 4.5 show the I_{base} for the three load pattern categories for a simple span bridge, 2 span continuous bridge, a 3 span continuous bridge with equal span lengths, and a 3 span continuous bridge with the exterior span lengths equal to 80% of the interior span length, respectively. An increase in span length yields an overall increase in the base moment of inertia for any bridge geometry or loading, but it is interesting to note the difference in the base moment of inertia for the different load patterns. The combined truck and uniform lane loadings require the largest moment of inertia in all cases. The HS truck load geometry always requires a larger moment of inertia for short span bridges than does the uniform lane load for all bridge span types. However, as the span length increases the uniform lane load has a more rapidly increasing impact on the bridge deflection than does the truck loading. A crossover between the two load patterns occurs around 175 ft (53.3 m). Comparison of Figs. 4.2 through 4.5 shows that continuous girders require a smaller I_{base} than simple spans. This is partly caused by the added stiffness due to continuity of the girder, but the more rational method for defining span length, L, in Fig 4.1 also contributes to this beneficial effect. The difference between 2 span continuous and 3 span continuous with equal spans is negligible.

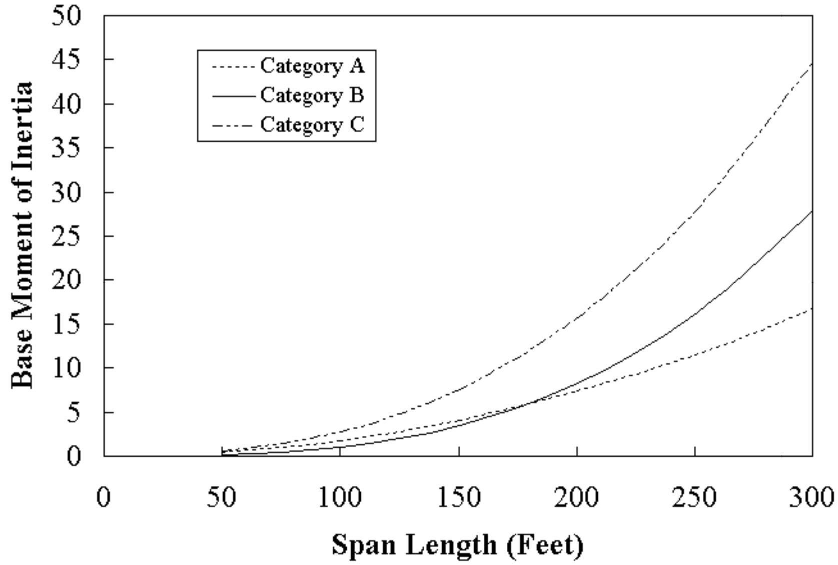

Figure. 4.2. I_{base} for Simply Supported Bridges

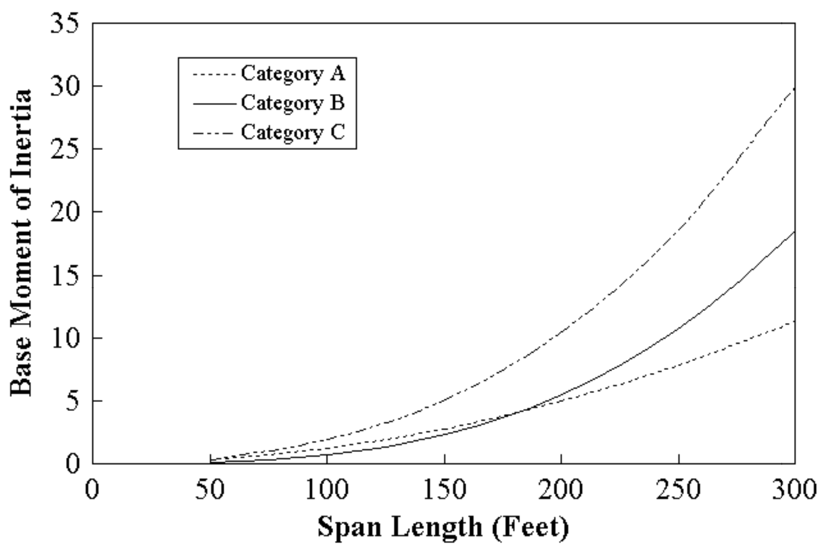

Figure 4.3. I_{base} for 2 Span Continuous Bridges

As shown in Eqn. 4.4, I_{base} can be multiplied by 40 kips (178 kN) to obtain I_{rel} for HS20-44 loading or multiplied by 50 kips (222.5 kN) to obtain I_{rel} for HS25-44. The effects of the distribution factor or dynamic impact factor can also be achieved by multiplying these values by DF and IF in Eqn. 4.4 as appropriate. The effect of individual deflection limits can be applied by dividing by R as shown in the equation.

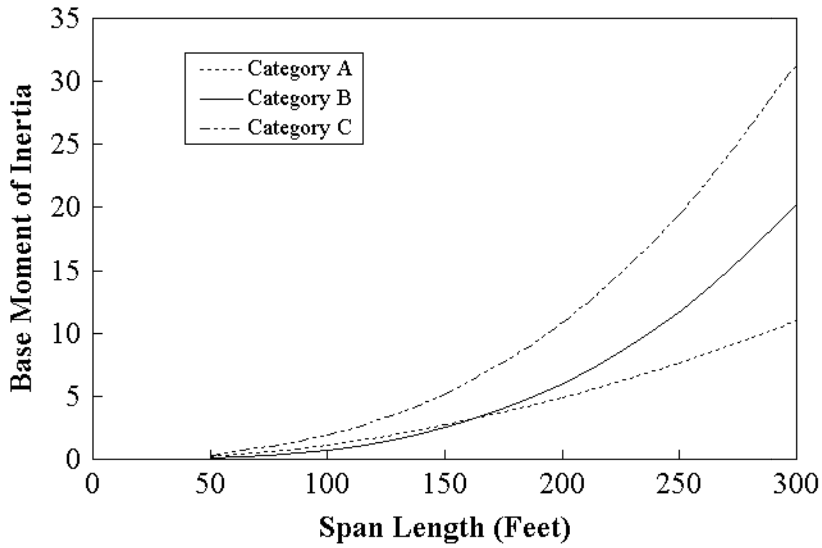

Figure 4.4. I_{base} for 3 Span Continuous Bridges with Equal Span Length

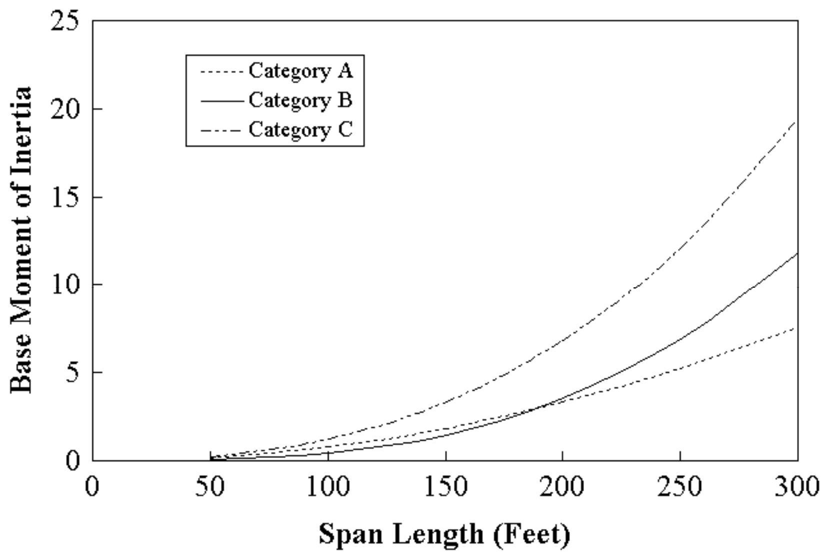

Figure 4.5. I_{base} for 3 Span Continuous Bridge with Unequal Span Lengths (80%-100%-80%)

Lane load distribution factors play a major role in the application of the deflection limit. The survey established two widely used methods of determining a lane load distribution factor. Some states employ lane load distribution factor from the AASHTO

Standard or LRFD Specifications. AASHTO Standard Specifications state that the DF should be calculated as $\frac{S}{7.0}$ if the girder spacing, S, is less than or equal to 10 ft (3.05 m) and there is only one traffic lane. DF is $\frac{S}{5.5}$ if S is less than or equal to 14 ft (4.27 m) and there are two or more traffic lanes. In both cases, if the girder spacing is greater than the limit, the deck is analyzed as a beam to determine the reaction to the girders. Other states employ an equal distribution of deflection principle. In these bridges, DF is no larger than the ratio of S to the lane width.

Figure 4.6 shows the effect of the AASHTO lane load distribution factors on I_{rel} for a 4 lane bridge with different road widths and different numbers of girders as compared to the equal distribution hypothesis. Similar curves were developed for other bridge widths and geometry. The difference decreases as the number of girders increases and as the bridge width decreases for a constant number of lanes of traffic. However, the difference between the two methods can range anywhere from about 55 percent to 345 percent (for a 2 lane bridge). It should be noted that the difference shown in this figure constitutes a percentage increase in the DF factor shown in Eqn. 4.4.

The AASHTO dynamic impact factor is calculated as $\frac{50}{L + 125}$ and is not to exceed 0.3, where L is the bridge length. This factor can be added to 1.0 to obtain IF in Eqn. 4.4. Figure 4.7 below, shows the variation in the dynamic impact factor with span length. The numbers in the plot represent the scaling factor that would be used if the dynamic impact factor were used. For instance, the plot shows a value of 0.22 for a 100 ft (30.48 m) bridge. In the previous analysis, a value of 1.22 would be multiplied to the base required moment of inertia to account for the use of the dynamic impact factor. The plot of the dynamic impact factor bears a strong resemblance to a theoretical acceleration response spectrum plot taking into account the fact that longer bridges will have higher periods and thus the dynamic effect of a truck crossing the bridge will be less critical for these longer period structures.

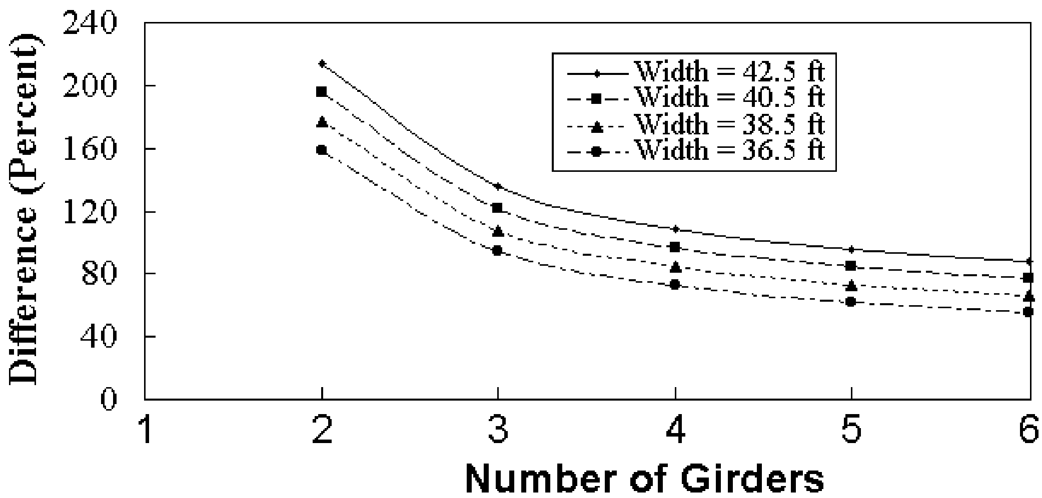

Figure 4.6. Difference in DF Factor for AASHTO Lane Load Distribution Factors as Compared to the Equal Distribution Method for a Four Lane Bridge

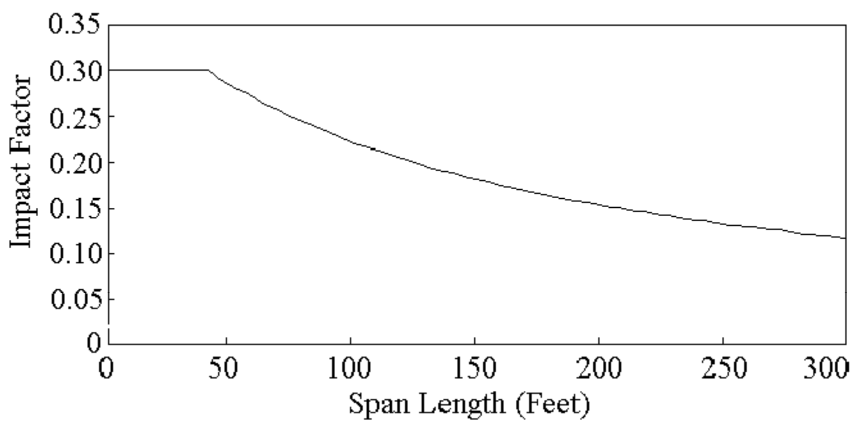

Figure 4.7. Effect of Impact Factor

4.4. Consequences of These Results

The prior discussion has shown that the application of deflection checks vary widely in practice. The deflection limits themselves vary between $\frac{L}{800}$ and $\frac{L}{1600}$. This results in a 100% increase in the minimum required moment of inertia, I, if identical bridges are checked for the same applied loads, impact factor, and lane load distribution factor. AASHTO truck loads require larger minimum I for short span bridges, but uniform lane loads will require larger minimum I for longer span bridges. States that employ combined truck and lane loads are requiring an I value that is nearly twice that needed for either of the individual load cases. The use of impact factors has a relatively modest effect on the deflection calculation as shown in Fig. 4.7. Some states use an equal load distribution model for their deflection check while other states employ the AASHTO lane load distribution factors. The use of AASHTO lane load distribution factors invariably increase the minimum required I by approximately 50% over that required with equal distribution model, and these factors may increase the minimum required moment of inertia by as much as 350% for some bridge geometry's. The combination of these effects indicate extreme variation in the application of these deflection limits.

An example of the possible variation in the total deflection limit criteria is useful. For this example, the same deflection vs. span length limit is used in both cases. The bridge is a 200 ft (61 m) simply supported bridge. Case A employs HS20-44 truck load only is used with equal distribution and no dynamic impact factor. For this case the minimum I_{rel} is 147 in^4 (.000061 m^4). For Case B an HS25-44 truck plus lane load is used with AASHTO lane load distribution and the dynamic impact factor. For Case B, the minimum I_{rel} is 1393 in^4(.000579 m^4). Case B requires a minimum I, which approximately 950% that required by Case A. This is a huge variation in the deflection limit application. Normally the $\frac{\delta}{L}$ limit would be included in the calculation, but because it was assumed that both checks would use the same limit, it was unnecessary to include it in this comparison. Thus, the above I values are relative values rather than absolute requirements. Larger differences are possible when the variation of the deflection limit are included in the evaluation. A 200 ft (61 m) bridge is a moderately long span but not unheard of. These two checks are on extreme opposites of the possible deflection limit application checks but they are still both possible checks based on survey data obtained from state bridge offices. They show that there is large possible variation in the application of deflection limits in various states, and this may have a greater impact upon steel bridge design in some states than in others.

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Chapter 5

Evaluation of Bridges Damaged by Deflection

5.1. Introduction

The survey of Chapter 3 identified a number of bridges, which had structural damage that engineers attributed to excessive bridge deflection and deformation. Photos, inspection reports, and design drawings were obtained for these bridges. A more detailed analysis of some of these bridges was completed, and this chapter summarizes that work.

The damaged bridges identified in the initial study were too numerous for detailed analysis of each individual bridge within the limited time and funding of this study. However, careful examination of the candidate bridges showed common attributes among both the bridge type and the damage characteristics. Bridges with similar design and construction and similar damage characteristics were grouped. A modest number of groups were identified, and the detailed analyses of the bridges were greatly simplified, because only selected candidate bridges from each of these groups were analyzed. The analyses established whether these selected bridges passed or failed the relevant state specific deflection criteria and standard deflection criteria, which is proposed in this chapter. The analyses established whether the damage can rationally be attributed to bridge deflection, and they examined whether alternate deflection criteria could control or prevent this damage

This chapter begins with a general description of the modeling and analysis procedures used in the analyses. The separate bridge type and damage mechanisms are then discussed, because of the common groups noted earlier. Cumulative results of the analysis and a discussion of the consequences to this project are then provided.

5.2. Analysis Methods

The initial analyses established whether the damaged bridge passed or failed existing deflection limits. Chapters 3 and 4 show wide variation in the application of the AASHTO deflection limit, and two separate deflection limit checks were employed. The proposed "standard" deflection limit evaluation was based upon an HS25-44 truck loading with impact. Equal distribution of the bridge load deflections between all bridge girders was employed, and the truck load was applied at the critical location in each bridge lane. The dynamic impact factor was determined based on the span length of the span in which the deflection was computed. The $\frac{L}{800}$ limit was used and was based upon the equivalent span length as discussed in Chapter 4 and illustrated for a continuous girder in Fig. 4.1. Upon completion of this standard load analysis, the bridge deflections were checked by the state specific procedure provided by the state in which the bridge was built. There sometimes was room for variation in the interpretation of the state specific deflection limits, because of ambiguity in the survey results. The range of this ambiguity was also analyzed. The general results of both global deflection checks are provided in Table 5.1 for these selected bridges. For most groups, other similar candidate bridges are known to exist, but they are not discussed here.

Plane frame, line girder models were established in the SAP 2000 (Wilson and Habibullah, 2001) computer program for each selected bridge. Composite action was assumed only where shear connectors were present on bridge plans, and the effective concrete flange width for composite sections was determined as recommended in the AASHTO LRFD Specification. In the calculation of composite transformed sections, steel reinforcement in the deck was ignored, and the concrete flange was modeled as a solid concrete section. The full variation of in-plane flexural properties over the member length were considered. Support conditions were modeled as pin supports or rollers in all cases.

Modeling began by constructing a MSExcel file that contained the various girder cross sections provided on the bridge plans. The analysis section properties were established and a relatively course initial finite element discretization were established in this spreadsheet to incorporate all section changes encountered in each structure. Connectivity of members and nodal locations were specified at this point. Haunched girders were modeled by step function changes to the bridge cross section at 2 ft (610 mm) increments or smaller. The MSExcel file was then loaded into SAP 2000, and the SAP graphical user interface was used for developing the remainder of the model. Once in SAP, all elements that were not already in 2 ft (610 mm) or smaller elements were automatically refined to this mesh. Symmetry was employed to simplify the model where possible. Support conditions were specified, and the joints and elements were re-numbered to aid in the interpretation of results.

Loading was applied in two steps. First, the standard load case was applied to the bridge using the SAP 2000 built in HS25-44 truck load. A separate load case was used for each span of continuous bridges, because separate AASHTO dynamic impact factors were defined for each span. The points of maximum deflection in each span were found, and influence lines for vertical deflection at those points were used to determine the critical position of truck loading. Once the points of maximum influence were determined, the centroid of the HS25-44 truck was placed at the point of maximum influence in each span, and the maximum deflection and $\frac{\delta}{L}$ ratio were determined. This second step was necessary because SAP 2000 returns only deflection and moment envelopes, when the automatic truck loading is used. Envelopes are useful for design but they do not accurately determine the $\frac{\delta}{L}$ ratio values for continuous spans. For continuous spans, the deflected shape and bending moment diagrams for the critical deflection case are required to correctly determine the L used to establish the deflection limit (see Fig. 4.1). In simply supported spans, this second step was not necessary, because the maximum overall deflection is given for the envelope, and L is the distance between supports. The maximum deflections for the automatically applied trucks and the manually applied trucks were compared and were always within 1 percent of each other.

Further analyses were completed for some bridges after the initial results were established. These further analyses attempted to determine if the damage can truly be attributed to bridge deflection and if a modified deflection check would prevent this bridge damage. These additional analyses typically evaluated local or system behavior, which is often a dominant consideration. These individual analyses are very specific to the individual groups, and they are briefly discussed in the sections that follow.

5.3. Discussion of Damaged Bridge Results

The damaged bridges were divided in 5 basic groups or categories as illustrated in Table 5.1. These individual categories of bridges are discussed separately here.

Table 5.1. Summary of Damaged Candidate Bridges Analyzed in this Study

Bridge	State	Standard Evaluation	State Specific	Comments
Plate Girders with Damaged Webs at Diaphragm Connections				
I-5 Sacramento Bridge	California	Pass	Pass	Two bridges. 5 simple spans. 25^{o} skew. Staggered diaphragms. Damage at cross-frame connections. Prevalent near supports.
SR-99 East Merced Overhead	California	Pass	Pass	6 simple spans. Staggered diaphragms with 58.38^{o} skew. Cracking at toe of diaphragm cope on bracing near supports.
SR-99 West Merced Overhead	California	Pass	Pass	Two bridges. 5 simple spans, 61^{o} skew. Staggered diaphragms. Most cracking at interior diaphragms near supports.
I-70 Great Tonoloaway Creek	Maryland	Pass	Fail	Two bridges. 3-span continuous girders, 15^{o} skew. Diaphragms aligned with no stagger. Cracking in negative moment regions.
I-75 Lake Allatoona	Georgia	Pass	Pass	Two bridges. 6-span continuous haunched girders. Right bridge. Cracking of web in gap between flange and stiffener.
US-50 By-Pass	Ohio	Fail	Fail	Two bridges. 3-Span continuous girders, 11.63^{o} skew. Bracing welded directly to web, full depth web cracking over piers.
Damaged Stringer to Floorbeam Connections				
Lake Lanier Bridge	Georgia	Pass	Pass	4-Span continuous truss. Right bridge. Double-angle stringer-floorbeam and floorbeam-truss connections. Floorbeam web cracking.

I-5 Cowlitz River Bridge	Washington	Pass	Pass	Simple span truss. Right bridge. Double-angle web stringer-floorbeam connections. Stringer web cracking from cope.
I-5 Skagit River Bridge	Washington	Pass	Pass	Simple span truss. Right bridge. Double-angle web stringer-floorbeam connections. Stringer web cracking.
Deck Cracking Damage				
Bridge Over Bear River and UP Railroad	Wyoming	Fail	Fail	4-Span continuous plate girder. 47^{o} skew. Deck cracking and spalling in regions of negative bending or small positive moment.
North Platte River Bridge	Wyoming	Pass	Fail	Two 5-Span continuous plate girder bridges. 20^{o} skew. Deck cracking in regions of negative bending or small positive moment.
Steel Box Girder Damage				
Glendale Ave Over Truckee River	Nevada	Fail	Fail	3-span continuous box girder. 32^{o} skew. Cracking of box webs at the diaphragm connections near piers and abutments.
Truss Superstructure Damage				
Davis Creek Bridge	California	Pass	Pass	Single span truss bridge. Right bridge. Cracking of truss pins and other damage due to differential truss deflections.

5.3.1. Plate Girders with Damaged Webs at Diaphragm Connections

The first category of bridge damage consisted of cracking of plate girder webs adjacent to diaphragm connections, and this was the most common damage mechanism obtained in the survey. Cracking occurs in the girder webs in the gap between the web stiffeners and the girder flanges as illustrated in Fig. 5.1. The damage can occur at any cross-frame or diaphragm connection, but damage was more common on interior girders, at diaphragm connections near the interior supports for continuous spans, and near mid-span for simple spans. Sharply skewed bridges appear to be more susceptible to this damage, and the orientation and stagger or misalignment of the diaphragms all play a role in the damage.

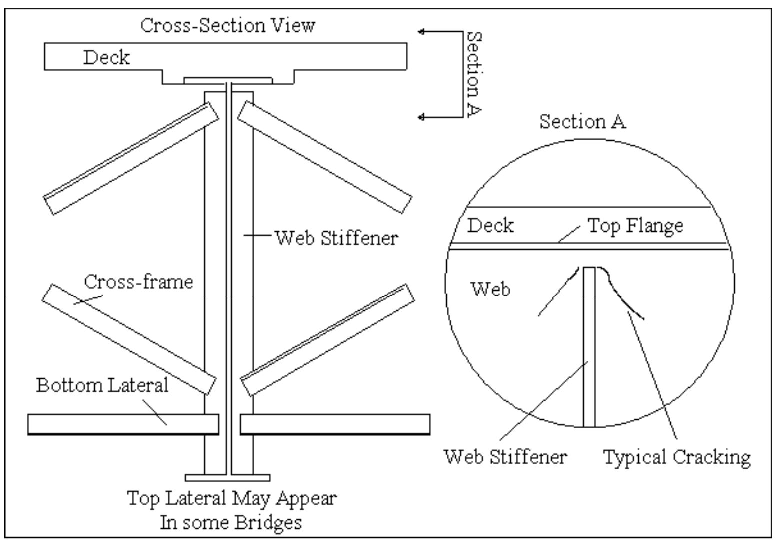

Figure 5.1. Typical Web Cracking at Diaphragm Connections

Analysis suggests that this damage is due to the out-of-plane deformation and connection rotation caused by differential girder deflections. When loading is applied to one lane of traffic or to one bridge girder, while other lanes and girders are unloaded, the bracing diaphragms and the deck combine to transfer load from the loaded girder to adjacent girders. The load transfer induces local stresses and strains or deformation at the diaphragm-to-girder connections. If the girder web is flexible to out-of-plane bending and if the diaphragm connection does not stiffen the web excessively, these stresses are minimal, and little damage can occur. The presence of a gap and the size of the gap between the diaphragm stiffener and the beam flange as shown in Fig. 5.1 affect the stress and strain levels. Stiff webs, stiff diaphragm connections, and short deformation lengths (gap between stiffener and girder flange) for the girder web increase the local stiffness, and large local stresses and strains develop in the webs of bridge girders. Stiffness and restraint is added at internal bridge piers. Misaligned diaphragms also add large local stiffness, which increases the local restraint, and misalignment may also increase the local deformation demand through opposing deformations in close proximity. This cracking is regarded as out-of-plane distortional fatigue by most researchers [Fisher 1990], but the distortion is caused by differential deflection. This cracking has been noted with a number of different diaphragm connection details, but analysis shows that this damage is clearly related to the local stiffness. Outside girders are relatively more free to undergo free body rotation, and they are less likely to incur this damage.

Table 5.1 shows that the deflection limit is ineffective in controlling or preventing this damage. Six bridges are included in this evaluation, and all but one of these bridges pass the standard deflection check described above. Two of the six bridges fail their state specific deflection limit. A live-load deflection limit would need to be very restrictive to prevent these bridge designs, and it is unclear that damage would prevented even if a more prohibitive deflection limit were employed. The damage is caused by differential deflection between adjacent girders, and the deflection limit does not address these deflections. Much of the damage would be limited or controlled by detailing measures to avoid local stress and strain concentrations at this diaphragm connection. Therefore, these bridges are clearly at the point of concern with regard to bridge deflections, and damage is noted regardless of whether the present AASHTO deflection limit is satisfied or not. Somewhat more detailed descriptions of these bridges and the resulting damage are provided.

5.3.1.1. I-5 Sacramento River Bridge

The Sacramento River Bridge consists of two identical, five span, simply supported, welded plate girder bridges with a 125 ft (38.1 m) span length. Each bridge has a 25 degree skew angle and consists of four girders spaced at 9 ft (2.75 m). The total bridge width is 34 ft (10.36 m) with a roadway width of 28 ft (8.53 m). The cross-framing is oriented perpendicular to the girder axis, and the diaphragm connections are staggered. The bridges were built in 1965 and are incurring the typical damage described above. The locations of damaged cross-frame connections were not specifically mentioned in most inspection reports, but it appears that most cross-frame connections in this bridge were damaged at some point. Particular damage is noted at diaphragm connections that are one cross frame away from the supports.

The bridges pass the proposed standard deflection check with a $\frac{\delta}{L}$ ratio of $\frac{1}{1534}$. The California deflection limit evaluation uses the HS20-44 truck plus lane plus impact load combination, and the bridge satisfies this deflection limit with a $\frac{\delta}{L}$ ratio of $\frac{1}{890}$ if no lane load distribution factor is employed. The bridge fails this check with a $\frac{\delta}{L}$ ratio of $\frac{1}{339}$ if AASHTO lane load distribution factors are used with HS20-44 loading.

5.3.1.2 SR-99 East Merced Overhead Right

The SR-99 East Merced Overhead Right Bridge has six simple spans and a skew angle of 58.4 degrees, and it consists of six welded plate girders spaced at 8 ft (2.44 m). The six spans are 60.62, 74.90, 86.32, 86.31, 100, and 97.19 ft (18.48, 22.83, 26.31, 26.3, 30.42, and 29.62 m), and the total bridge width is 45.33 ft (13.82 m) with a roadway width of 41 ft (12.5 m). The cross-bracing diaphragms are oriented perpendicular to the girders and have staggered connections. The bridge was built in 1962, and cracking is occurring in the girder web adjacent to the toe of a stiffener cope around the flange-web weld. The stiffeners butt up against the girder flanges and are seal welded to the flanges. They do not have the gap as illustrated in Fig. 5.1. Damage was most prevalent at diaphragms near supports.

All spans were analyzed as simple beams, and the largest deflection noted with the proposed standard deflection check had an $\frac{\delta}{L}$ ratio of $\frac{1}{2806}$. California's reported deflection limit application case uses the truck plus lane plus impact load case. The bridge also passes the state specific deflection limit check if equal distribution between girders is employed, but it fails the limit with an $\frac{\delta}{L}$ ratio of $\frac{1}{629}$ if the AASHTO lane load distribution factors with HS20-44 loading is employed.

5.3.1.3. SR-99 West Merced Overhead

The SR-99 West Merced Overhead consists of two identical bridges with five simple spans with lengths between 97.1 and 108 ft (29.6 and 32.91 m). Each bridge has a skew angle of 61 degrees and consists of five girders spaced at 8.5 ft (2.59 m). The total bridge width is 39.67 ft (12.09 m) with a roadway width of 37 ft (11.28 m). The bridge has staggered cross-framing oriented perpendicular to the flow of traffic.

This bridge was built in 1962, and cracking is noted in girder webs at the diaphragm connections. Most reported damage was on the interior girders near supports. All spans were analyzed as composite girders with the standard deflection limit evaluation, and the largest $\frac{\delta}{L}$ value was $\frac{1}{2068}$. The state specific deflection limit is again satisfied if the AASHTO lane load distribution factors are not employed. With AASHTO lane load distribution factors and HS20-44 loads, the most critical span clearly fails the $\frac{1}{800}$ deflection check with a $\frac{\delta}{L}$ ratio of $\frac{1}{410}$.

5.3.1.4. I-70 Over Great Tonoloaway Creek

The I-70 over Great Tonoloaway Creek Bridge consists of two identical 3-span continuous welded plate girder bridges. The bridge has a 15 degree skew angle, and consists of five girders spaced at 8.08 ft (2.46 m). The three spans are 124, 155, and 124 ft (37.8, 47.24, and 37.8 m), and the total bridge width is 36.17 ft (11.02 m). The cross-framing on the bridge is oriented perpendicular to the flow of traffic, but the diaphragms are aligned. The diaphragm connections in had a 1 in (25 mm) clearance between the end of the stiffener and the tension flanges in the negative moment regions.

The bridge was built in 1963, and cracking occurred only in the negative moment regions in the stiffener gap noted above. Specifically the damage is localized to the first or second line of cross-framing from each side of the two piers. The analysis suggests that these diaphragms transfer more load than may be normally expected because they are attempting to transfer load directly to interior piers from adjacent bridge girders.

Two single girder models were developed to represent the various bridge girders. One model simulated the outside girders, and the other model represented interior girders, which had slightly different dimensional properties. For the standard load check with the HS25-44 loading, the bridges satisfied the deflection limit with a $\frac{\delta}{L}$ ratio of $\frac{1}{1411}$. Maryland's reported deflection limit application case uses the worst of an HS25-44 truck or HS25-44 lane load and AASHTO distribution factors. No load factors are used, and the respondent of the phase one survey was unsure of the use of the dynamic impact factor. As a result, the deflections were checked with and without the AASHTO impact factor. These bridges clearly failed the state specific deflection limits with deflection ratios in the order of $\frac{1}{400}$.

5.3.1.5. I-75 over Lake Allatoona

The I-75 over Lake Allatoona Bridge has a pair of six span, continuous, haunched, welded plate girder bridges with no skew. They have 7 identical girders spaced at 8.75 ft (2.67 m), and the individual span lengths are 133.82, 182, 190, 190, 182, and 133.82 ft (40.79, 55.47, 57.91, 57.91, 55.47, and 40.79 m), respectively. The total bridge width is 62.5 ft (19.05 m). The bridge was built after 1975 and is experiencing web cracking in the gap between the diaphragm connection stiffener and the girder flange in regions of both positive and negative moment.

The bridge was analyzed as a composite girder. The proposed standard deflection limit evaluation was applied, and the critical $\frac{\delta}{L}$ ratio was $\frac{1}{808}$ in the center span. Georgia's reported deflection limit application case uses the worst case of a lane load plus impact, a truck load plus impact, or a military load plus impact. The military load plus impact was not defined in the survey, but it is likely heavier than the HS25 truck load. The deflection limit barely satisfied the standard check, and so the deflections are unlikely to satisfy the deflection limit with the military vehicle load if multiple lane loads are applied. In addition, the distributed lane load was also investigated. The second span has the critical deflection under the uniform applied load, and the deflection is $\frac{1}{714}$ of the span length with the HS20-44 uniform lane load applied to alternate spans of the girder. However, the survey indicated that Georgia employs only a single lane loading with their state specific deflection check, and with the single lane loading the bridge passes the $\frac{L}{800}$ deflection limit.

5.3.1.6. US-50 By-Pass

The US-50 By-Pass Bridge is one of two similar bridges that differ only in the horizontal slope of the bridge deck. Because the relative vertical locations of the girders is not a factor in the analysis only one of the bridges was analyzed. The bridge is a 3-span continuous wide flange girder bridge with an 11.6 degree skew angle. There are 6 identical wide flange girders spaced at 7.92 ft (2.41 m) and the span lengths are 56, 70, and 56 ft (17.06, 21.34, and 17.06 m), respectively. The total bridge width is 44.33 ft (13.52 m). Diaphragms are perpendicular to the axis of the girders, and their connections are aligned. This bridge differs from the previous examples in that the cross framing is welded directly to the girder webs. Full depth girder web cracking has occurred in two girders directly over interior piers. The cracking occurs at a diaphragm connection, which are also girder splices. The cracks originate from the weld access hole where the girder was field spliced.

The standard deflection check was applied. The critical deflection occurred in the center span, and it was $\frac{1}{385}$ of the span length. Ohio's reported deflection limit uses a lane load plus impact loading, and they use AASHTO lane load distribution with multiple lane loading. This state specific loading was applied based upon the HS20-44 loading. The critical deflection was in the center span and was $\frac{1}{264}$ of the span length.

5.3.2 Bridges with Damage in Stringer Floorbeam Connections

Damage to stringer-floorbeam connections as illustrated in Fig. 5.2 was also quite common. This damage is noted in truss bridges, tied arch bridges, and bridges with two heavy plate girders, since these bridge types may contain a stringer-floorbeam system. The damage occurs in either the stringer-to-floorbeam connection or the connection between the floorbeam and the large superstructure element. Typically, the primary superstructure elements are very stiff and do not have deflection related damage. Three typical bridges of this type are included in Table 5.1, but a much larger number of similar bridges were identified in the survey.

Analysis shows that this particular damage mechanism is related to the relative stiffness of the stringers, floorbeams, the primary superstructure and their connections. As loading passes over the stringers, they deflect. Then:

- If the stringer-floorbeam connection is stiff, the stringers twist the floorbeam as the stringers deflect. The connection rotation of the stringer provides the floorbeam rotation, and this induces cracking in the floorbeam web.
- If the floorbeam is adequately restrained against twisting, cracking as shown in Fig. 5.2 may occur in the stringer web at the stringer-floorbeam connection, because of the negative bending moment induced by the connection stiffness.
- If the floorbeam is unrestrained against twisting, cracking may occur at the floorbeam-superstructure connection as illustrated in Fig. 5.3. This later damage is caused by the differential twist rotation of the floorbeam relative to the small rotation and deformation expected in the bridge superstructure.

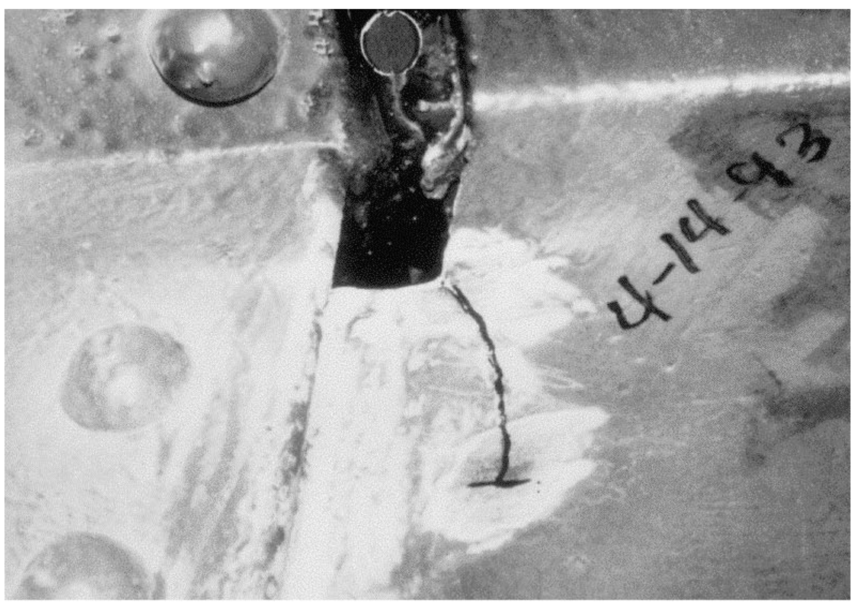

Figure 5.2. Stringer Cracking Due to Connection Restraint

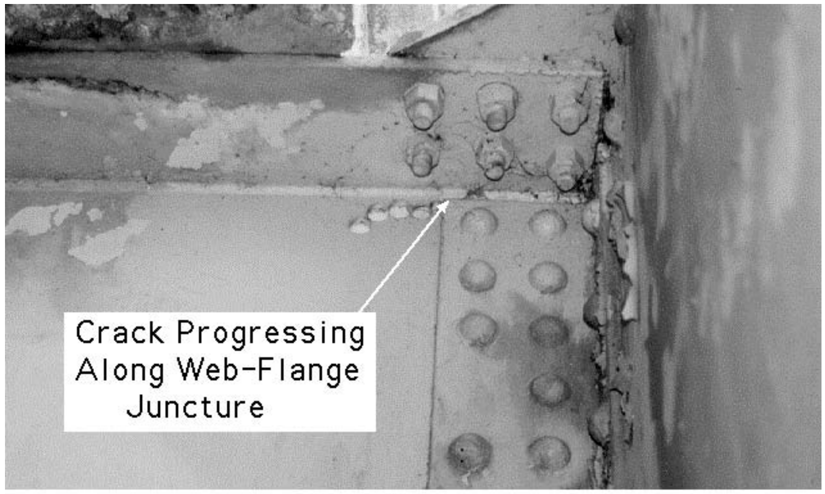

Figure 5.3. Floorbeam Cracking Due to Relative Twist Between Floorbeam and Superstructure

The AASHTO deflection limit is normally applied to the main bridge structure, and this deflection limit is evaluated in Table 5.1. In all cases, the global bridge deflections satisfied both the standard deflection check and the state specific deflection check. The above comments show that the connection deformations are caused by local deflections of stringers and floorbeams. The individual deflections of these elements were always closer to the $\frac{L}{800}$ deflection limit than the global checks, but they usually satisfied the deflection limit. Therefore, the existing AASHTO deflection limits clearly have no benefit in controlling this damage type. Nevertheless, this damage is caused by connection rotations (both torsional and flexural) induced by bridge deflection and deformation. Design engineers commonly treat these connections as pinned connections. They seldom consider the consequences of member end rotations on the connection or the adjoining members, and they typically don't consider the effect of the true connection stiffness on the performance. The relative stiffnesses of these different elements cause this local deformation, but there is no clear method for controlling this stiffness differential. A more detailed description of several individual bridges follows.

5.3.2.1 Lake Lanier Bridge

The Lake Lanier Bridge is a 4-span, continuous, truss bridge. The span lengths are 200, 260, 260, and 200 ft (60.96, 79.25, 79.25, and 60.96 m). The two main trusses have floorbeams spanning between the top chords of the trusses and stringers spanning between floorbeams. It is a right bridge, and the total bridge width is 30.5 ft (9.30 m) with a roadway width of 26 ft (7.92 m). The stringers are substantially smaller than the floorbeams, and the stringer-floorbeam and floorbeam-truss connections were riveted double-web-angle connections.

This bridge was designed by the Army Corps of Engineers in 1955. Cracking occurred in the floorbeam webs just above the connection angles due to localized twisting of the floorbeams. This bridge has since been retrofitted by replacing the floorbeams, and installing new stringers on top of the floorbeams. There has been no reported damage since this retrofit, because the stringers are now unable to twist the floorbeams and thus induce stress and rotation into the floorbeam-truss connections.

The proposed standard deflection check was evaluated. The span to deflection was found to be $\frac{1}{1781}$. Georgia's reported deflection limit application case uses the maximum deflection obtained by applying; a lane load plus impact, a truck load plus impact or a military load plus impact. The calculations show that the bridge also satisfies the state specific deflection limit.

The deflection limit is normally applied to the global bridge deflections, but the analysis indicates that the damage is caused by relative twisting deformation between the floorbeam and the truss. The floorbeam twist is largely driven by the stringer end rotations. Therefore a local application of the deflection limits was applied to the stringers. The stringers failed the standard deflection check with a deflection that was $\frac{1}{734}$ of the stringer span length. The stringers satisfied the state specific deflection limit, because the HS20-44 load was used for this check.

5.3.2.2. I-5 Cowlitz River Bridge

The Cowlitz River Bridge has 2 simple span trusses with a stringer-floorbeam deck system. The truss span lengths are 240 ft (73.15 m) with a roadway width of 28 ft (8.53 m). The stringer-floorbeam connection is a riveted double-web-angle connection. The stringer top flange is either above or level with the floorbeam top flange, and the stringer flange is coped to accommodate the floorbeam top flange. Stringer cracking initiates from the stringer cope and progresses into the stringer web at numerous stringer-floorbeam connections. The bridge was built in approximately 1962.

The deflections were evaluated for the standard deflection limit evaluation, and the maximum deflections were $\frac{1}{4787}$ of the span length. This deflection is significantly smaller than the $\frac{1}{1000}$ limit used for bridges with pedestrian access. Washington's reported deflection limit application case uses the larger deflection caused by an HS25-44 truck plus impact or the HS25-44 lane plus impact. They do not use load factors and assume multiple lanes loaded with equal distribution. The bridge passed the $\frac{1}{1000}$ limit with a maximum deflection that was $\frac{1}{2678}$ of the span length.

The deflection limits were applied locally to the stringers and floorbeams. The maximum $\frac{\delta}{L}$ ratio was $\frac{1}{1165}$ for both the standard and state specific checks, because the uniform lane loading will not provide the controlling deflection with the short spans. This bridge passes all relevant deflection limits but is sustaining significant damage.

5.3.2.3. I-5 Skagit River Bridge

The Skagit River Bridge has 4 simply supported truss spans with 160 ft (46.77 m) lengths and roadway widths of 56 ft (17.07 m). The bridge was built after 1957, and it is experiencing similar damage to the Cowlitz River Bridge. The stringer-floorbeam connections are identical, but there are slight differences in the performance of the two bridges. The majority of the cracking in the Skagit River Bridge originates and propagates from the stringer flange-web intersection as opposed to the corner of the cope. It is unclear why this difference occurs, but it may be affected by differences in the shape and size of the copes.

The standard evaluation procedure was applied and the maximum $\frac{\delta}{L}$ ratio was $\frac{1}{2596}$. The state specific deflection limit was also applied to check the global deflections of the bridge, and the maximum deflection was $\frac{1}{1987}$ of the span length. As with prior examples, the deflection limit was applied to the local deflections for the stringers and floorbeams. The maximum $\frac{\delta}{L}$ ratio for this check was $\frac{1}{1020}$ for both the standard and state specific evaluation.

5.3.3. Bridges with Deck Damage

Deck cracking is often regarded as a potential source of damage caused by bridge deflection, but the survey identified only 2 bridges where deflections contributed to deck damage. Considerable deck cracking is noted on existing bridges, but this cracking is often attributable to other material and environmental phenomenon as noted in Chapter 2.

These bridges exhibited transverse deck cracking located in regions of negative bending over interior supports and at the ends of outside spans. The cracking appears to be driven by bridge deflection. It is occurring in locations of negative bending and locations with relatively small positive bending moments. Therefore, the AASHTO deflection limit is a very indirect measure of this damage potential. Table 5.1 summarizes the deflection check on these bridges, and the results are clearly mixed. Both bridges fail the state specific deflection check, but one passes the standard check. This damage category is the one possible category where the existing deflection limit may provide a beneficial effect, because limiting the overall deflection would also limit the negative bending moments observed over interior supports and at inadvertent joint and bearing restraint. However, the existing deflection limit would clearly be an indirect check, and observation of this damage provides no evidence as to what the deflection limit should be. Further, the cracking is not occurring at the locations of maximum deflection, strain or curvature in the bridge girders. This cracking may at least be partially caused by restraint provided by joints and bearings. A limit on the tensile strain in the concrete deck as a result of the expected or inadvertent restraint may be effective in preventing this damage.

5.3.3.1 Bridge Over Bear River and Union Pacific Railroad

The Bridge over Bear River and Union Pacific Railroad is a 4-span, haunched, continuous, welded plate girder bridge with a 47 degree skew. The four spans are 84, 120, 120, and 84 ft (25.6, 36.58, 36.58, and 25.6 m), respectively. The bridge has 4 girders spaced at 9 ft (2.74 m) with a roadway width of 32 ft (9.75 m). Diaphragm bracing is aligned and is oriented perpendicular to axis of the bridge. The stiffener used to achieve the diaphragm-girder connections was welded the full height of the web. The stiffener had a close fit to the tension flange but was not welded to the flange. The bridge was designed in 1965 and is experiencing transverse deck cracking. Transverse cracks and spalling are noted over 5 percent of the wearing surface.

The bridge was analyzed without composite action, and the standard deflection limit check resulted in maximum deflection of $\frac{1}{669}$ of the span length. This clearly fails the $\frac{1}{800}$ deflection limit. Wyoming reports that they use a truck plus lane plus impact load case with factored loads. These loads be will significantly heavier than the standard load case, and so this bridge also fails the state specific deflection limit.

5.3.3.2. Bridge over North Platte River

The Bridge over North Platte River consists of 2 identical, 5-span, continuous, welded plate girder bridges. Each bridge has a skew angle of 20 degrees and consists of five girders spaced at 9.25 ft (2.82 m). The span lengths are 110, 137, 137, 137, and 110 ft (33.53, 41.76, 41.76, 41.76, and 33.53 m), respectively, and the total bridge width is 44.67 ft (13.62 m) with a roadway width of 42 ft (12.8 m). The diaphragms were aligned and placed at a skew with respect to the bridge axis. The bridge was designed in 1969 and it is experiencing transverse deck cracking, but the cracking is less severe than noted in the prior example.

The bridge passed the standard deflection limit check with a maximum $\frac{\delta}{L}$ ratio of $\frac{1}{865}$ in the center span. The state specific deflection limit employs a truck plus uniform lane load plus impact load case with factored loads. This load combination is significantly larger, and the bridge failed the state specific check with a critical $\frac{\delta}{L}$ ratio of $\frac{1}{483}$.

5.3.4. Steel Box Girder Damage

The survey produced only one steel box girder bridge with damage.

5.3.4.1. Glendale Avenue over Truckee River

The Glendale Ave over the Truckee River Bridge is a 3 span, continuous, box girder bridge with a 32 degree skew. The span lengths are 112.5, 160, and 112.5 ft (34.29, 48,77, and 34.29 m). The bridge has 5 girders spaced at 20 ft (6.10 m) with a total bridge width of 101 ft (30.78 m) and a roadway width of 88 ft (26.82 m). The internal cross-framing is aligned and is oriented parallel to the skew angle. The bridge was designed in 1977, and cracking at the diaphragm connections is scattered throughout the bridge with no detectable pattern. The cracking occurs in the toe of the cross-frame connector plate where it is welded to the webs within the box girder. The bridge is very wide relative to the span length.

A single girder model was used to check basic girder stiffness. The beam elements included composite action. The concrete flange for the girder was taken as 20 ft (6.1 m) and included the concrete used to embed the girder flanges. Longitudinal WT sections stiffened the bottom flange over the supports, and these were also included in the calculation of the moment of inertia of the girder. The standard deflection check was applied. The bridge failed the $\frac{1}{1000}$ limit for bridges with pedestrian access with a maximum $\frac{\delta}{L}$ ratio of $\frac{1}{829}$. Nevada reports that they use an HS 20-44 truck plus impact load case for non-NHS roads and an HS 25-44 truck plus impact load case for NHS roads with no load factors, multiple lanes loaded, and AASHTO distribution factors. The bridge also fails this state specific deflection check. The bridge is quite flexible, and this flexibility causes the bridge damage. However, more detailed analysis shows that the system behavior of the combined girders in the wide, skew bridge directly causes the damage.

There was not adequate time or funding to complete a system analysis of this bridge, but a somewhat more detailed analysis suggests that the damage is caused by differential deflections and box girder rotations that are caused by the skewed geometry of the bridge and the wide bridge deck. Skew bridges deform so that some girders are lightly loaded under these conditions, and the uplift or unloading causes rotation and twist of some box girders. The box girder cross-section undergoes slight cross-sectional warping when subject to this twist, but the bracing diaphragms restrain part this warping, because they are not normal to the girder axis. The large box girder forces caused by the rotation induce the local stress and strain that cause the web cracking. It is possible that the omission of the cross-frames would eliminate this damage but this would make construction of the box girders nearly impossible.

5.3.5. Truss Superstructure Damage

One truss bridge, which is experiencing damaged pins in the top chord connections, was identified.

5.3.5.1. Davis Creek Bridge

The Davis Creek Bridge is a one span, simply supported, truss bridge with no skew. The span length is 129.5 ft (39.47 m), the total bridge width is 21.33 ft (6.5 m), and the roadway width is 18 ft (5.49 m). The bridge consists of 2 trusses with a stringer-floorbeam system. The bridge was constructed in 1925, and the damage is occurring near mid-span where the bridge is less restrained to cross-sectional distortion. The bottom chords of the truss are pinned eye bars while the top chords, verticals, and diagonals are all built up double channel sections. Damage is occurring in the form of cracked and fractured truss pins as well as other damage types.

The bridge passed the standard deflection limit with a maximum $\frac{\delta}{L}$ ratio of $\frac{1}{1756}$. The state specific deflection limit was employed, and the maximum deflection was $\frac{1}{819}$ of the span length. Analysis suggests that this damage is occurring due to differential deflection of the two trusses. The damage occurs when one truss deflects relative to the other, because this causes twisting of the bridge cross-section. The rotation and distortion are resisted by the top laterals and top chord connections, but these are very light. The torsional deformation places great demands on the pins in the top chord connections, and the pins and connections ultimately fracture or sustain other damage.

A deflection check that compares the deflection of individual trusses compared the spacing or distance between trusses may be a relevant method of controlling this bridge damage. The bridge is relatively narrow compared to its span length, and so even a modest vertical truss deflection may cause significant torsional distortion.

5.4. Summary and Discussion

Welded plate girders with damaged webs at cross-frame connections were evaluated, and these bridges usually satisfied both the standard and state specific deflection limits. The damage noted in this group could be reduced by better detailing practices. The use of staggered diaphragms clearly can place significant demands on plate girder webs. Gaps and connectivity detailing between the diaphragm stiffener and the girder flange also affect the local stress and strain. Connection details that employ larger gaps could reduce these stresses and strains, although the larger gap may also reduce the lateral support provided by the bracing. Diaphragm connection details that prevent the local deformation could also have a beneficial effect. Many of the problems with these bridges are associated with skew. The distribution of load between girders is different in skew bridges and curved bridges than in straight right bridges. Greater forces are transferred through the diaphragms in these more complex structural systems, and the diaphragm places greater demands upon the diaphragm connection. Deflection limits are at best an extremely indirect way of controlling this damage. The best technique for controlling this damage is better detailing and a better understanding of the bridge system behavior.

Damage due to rotation and twist in stringer-floorbeam and floorbeam-superstructure connections was also frequently noted. Global deflection limits do not control this damage, because local deflection and member end rotation in the stringers and floorbeams are the driving effect. Local deflection checks based upon stringer and floorbeam deflections are more relevant, but the AASHTO deflection limit does not prevent this damage even on this local level. Instead, the engineer must recognize the local rotations and deformations that occur within the structural system and examine their consequence on adjacent members and connections if this damage is to be avoided.

Deck cracking caused by bridge deflection was identified in a relatively small number of bridges. Transverse deck cracking occurs in regions of negative bending and regions with small positive bending moment. The AASHTO deflection limit is at best an indirect control of this damage, because the deck cracking is not occurring anywhere near the location of maximum deflection.

Other damage mechanisms were noted, and these were caused by local deformations and system behavior rather than global bridge deflections. The AASHTO deflection limit is applied as a line element check, and it is not effective in controlling this behavior. Of the thirteen damaged bridges analyzed in this chapter, 77% passed a standardize application of the AASHTO deflection check. The state specific deflection checks are much more variable, but 61% of the bridges were found to pass the state specific check. This again suggests that existing deflection limits are not effective in preventing this damage.

This chapter has described a number of bridges that have sustained damage due to local deformations and differential deflection. The evidence shows that these bridges are damaged by deflection, but the evidence also clearly shows that:

- Existing deflection limits provide no clear benefit in controlling this damage,

- The bridge designs for these damaged bridges frequently had ill-conceived details that contributed to or caused the problems, and
- Many of these ill-conceived designs are today prohibited because of later changes to the AASHTO Specifications.

Nevertheless, serviceability and durability of bridges are continual concerns. Engineers knowledge and understanding of bridge behavior is continually expanding, and economic pressures upon bridge engineers cause continual change in design practice. AASHTO Specifications can never be so detailed as to avoid all ill-conceived designs in the future, and it is unlikely that all deformation and differential deflection problems are prevented in existing bridges.

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Chapter 6

Evaluation of Existing Plate Girder Bridges

6.1 Introduction

From the survey of Chapter 3 and from meetings with state bridge engineers affiliated with the AASHTO T-14 Steel Bridge Committee, the investigators obtained design drawings for 12 typical plate girder bridges, which are summarized in Table 6.1. These bridges were recently (approximately last 10 years) constructed by 6 different state transportation departments. The bridges include simply supported and continuous spans, and they include structures fabricated from HPS 70W and more conventional steels. Hence, they are a representative cross-section of I shaped steel plate girder bridge designs typically employed in present practice. Bridges with haunched girders, box-girders, and very wide deck widths were obtained but were not considered in the present effort.

This chapter evaluates the live-load deflection performance of these representative bridges against current AASHTO Specifications and examines the impact of two alternative serviceability criteria on there performance and design. The alternate serviceablility criteria included the Walker and Wright [Walker and Wright, 1972] procedures and the Ontario Highway Bridge Design Codes[Ministry of Transportation, 1991] as discussed in Chapter 2.

6.2 Analysis Methods

Two sets of analyses are conducted for each bridge. The first was a line girder analysis incorporating the effective width, load distribution factors, and loadings as implied by the AASHTO Standard Specifications (AASHTO,, 1996). The AASHTO Specification deflection check was computed based upon the larger deflection developed through application of the AASHTO standard truck load or distributed lane load with impact. The deflections assumed uniform deflection of all bridge girders and incorporated multiple presence lane loads where applicable. The second analysis was based on the requirements specified in the Ontario Highway Bridge Code (Ministry of Transportation, 1991). The commercial design package SIMON (SIMON SYSTEMS, 1996) was used for the Load Factor Design Analyses and CONSYS 2000 by Leap Software (CONSYS 2000) was used to conduct the moving load analyses based on the Ontario specifications for each of the bridge. For each analysis, both dead loads and section properties were calculated based on cross section information provided on the plans. Analyses were conducted assuming composite action throughout. The analyses accounted for all flange thickness transitions. The maximum deflection for a given span from the software output was then recorded and compared to respective limits. The natural frequency for both the Walker and Wright recommendations and the Ontario Highway Bridge Design Code are computed using Equation 2.5.

6.3 Description of Bridges

Design drawings, inspection reports and other detailed information were obtained for these candidate bridges, and this section provides a brief description for each bridge. Table 6.1 provides summary information for each of the bridges described below. These bridges were selected because they represent typical bridges constructed from HPW70W steel, other conventional grades of steel, or a hybrid application of the materials.

Table 6.1 Summary of Typical Plate Girder Bridges Analyzed in this Study

Bridge Number	Bridge Identification	State	Standard Evaluation	Comments
1	Jackson County	Illinois	Pass	Simple span composite. 103.83 ft span. 75° skew. 5 girders at 7.42 ft spacing. Staggered diaphragms.
2	Randolph County	Illinois	Pass	4-span continuous. 81, 129.5, 129.5, and 81 ft spans. Right bridge. 5 girders at 5.17 ft spacing. Non-staggered diaphragms.
3	Dodge Street	Nebraska	Pass	2-span continuous. 236.5 ft spans. Right bridge. 8 girders at 9.5 ft spacing. Non-staggered diaphragms.
4	Snyder South	Nebraska	Pass	Simple span composite. 151 ft span. Right bridge. 5 girders at 8 ft spacing. Non-staggered diaphragms.
5	Seneca	New York	Pass	2-span continuous. 100 ft spans. Right bridge. 5 girders at 7.375 ft spacing. Non-staggered diaphragms.
6	US Route 20	New York	Pass	Simple span composite. 133 ft span. 120° skew. 6 girders at 9.5 ft spacing. Non-staggered.
7	Ushers Rd I-502-2-2	New York	Pass	2-span continuous. 183 ft spans. Right bridge. 6 girders at 9.33 ft spacing. Non-staggered diaphragms.

8	Berks County	Pennsylvania	Pass	Simple span composite. 211 ft span. 45° skew. 4 girders at 10.92 ft spacing. Non-staggered diaphragms.
9	Northampton County	Pennsylvania	Pass	Simple span composite. 123 ft span. Right bridge. 5 girders at 9 ft spacing. Non-staggered diaphragms.
10	Clear Fork	Tennessee	Fails	4-span continuous. 145, 220, 350, and 80 ft spans. Right bridge. 4 girders at 12 ft spacing. Non-staggered diaphragms.
11	Martin Creek	Tennessee	Fails	2-span continuous. 235.5 ft spans. Right bridge. 3 girders at 10.5 ft spacing. Non-staggered diaphragms.
12	Asay Creek	Utah	Pass	Simple span composite. 76,125 ft span. Right bridge. 6 girders at 8 ft spacing. Non-staggered diaphragms.

#1 - Illinois - Route I 27 over Cedar Creek in Jackson County

The Route I 27 Bridge is a simple-span composite steel plate girder bridge with a span length of 103.83 ft (31.67m). It has integral abutments. It consists of a 7.5 in (190.5mm) reinforced concrete deck supported by 5 girders spaced at 7.42 ft (2.26m) on center. The girders are fabricated from conventional Grade 50 (G345) steel. It was designed using the 1992 AASHTO 15th Edition LFD Design Specifications and the HS20-44 design loading.

#2 - Illinois –Route 860 over Old Mississippi River Channel in Randolph County

The Route 860 Bridge is a 4-span continuous steel plate girder with 82.25, 129.5 and 82.25 ft (25.07, 39.47, and 25.07 m) span lengths, respectively. It has a 7.5 in (190.5 mm) reinforced concrete deck and 5 Grade 50 (G345W) steel girders spaced at 5.17 ft (1.57m) on center. It was designed using the 1996 AASHTO LFD Design Specifications with the 1997 Interim and the design vehicle is HS20-44.

#3 - Nebraska - Dodge Street over I - 480 in Douglas County

The Dodge Street Bridge is a 2-span continuous steel plate girder bridge with equal spans of 236.5 ft (72.090m). It consists of an 8.5 in (216mm) reinforced concrete deck supported by 8 girders spaced at 9.5 ft (2.9m) on center. The hybrid girders are fabricated from HPS70W (485W) steel in the flanges of the negative bending region and conventional Grade 50W(G345W) steel is used in the web throughout the bridge and in the flanges in the positive bending region. It was designed using the 1997 AASHTO LRFD Design Specifications and the design vehicle is HL93.

#4 - Nebraska - Highway No. N-79 Snyder South

The Snyder South Bridge is a simple-span composite steel plate girder bridge with a span length of 151 ft (46m). It consists of a 7.5 in (190.5mm) reinforced concrete deck supported by 5 girders spaced at 8 ft (2.44 m) on center. The girders are fabricated from HPS70W (485W) steel. It was designed using the 1994 AASHTO LFD Design Specifications and the design vehicle is HS25 (MS22.5).

#5 - New York - Interstate 502-2-2 Ushers Road

The Interstate 502-2-2 Bridge is a two-span continuous steel plate girder bridge with equal spans of 183 ft (56.074m). It has a 9.5 in (240mm) reinforced concrete deck and 6 girders spaced at 9.33 ft (2.82m) on center. For live-load deflections the design vehicle is HS25 design load was applied according to AASHTO 16th Edition Act. 10.6.4.

#6 - NY State Thruway - Bridge No. TAS 98-8B Seneca 5 Bridges

The New York State Thruway authority used one typical plan set for 5 replacement bridges. The Seneca 5 Bridges are 2-span continuous composite steel plate girder bridges with equal spans of 100 ft (30.5m). They have an 8 in (200mm) reinforced concrete deck with a 1.5”(40mm) wearing course supported by 5 girders spaced at 7.375 ft (2.25m) on center. The girders are fabricated from HPS70W (485W) steel. It was designed using the 1996 AASHTO ASD Specifications and the design vehicle is HS25 (MS22.5).

#7 - New York –US Route 20 over Route 11 A in Onondaga County

The Route 20 Bridge is a simple-span composite steel plate girder bridge with a 133 ft (40.5m) span length. It has a 9.5 in (240mm) reinforced concrete deck and 6 conventional Grade 50 (G345W) steel girders spaced at 9.5 ft (2.89m) on center. It was designed using the AASHTO 16th Edition and the design vehicle is HS25 (MS22.5).

#8 - Pennsylvania –Berks County

The Berks County Bridge is a single-span composite steel plate girder bridge with a 211 ft (64.32m) span length. It consists of an 8.5 in (216 mm) reinforced concrete deck supported by 4 girders spaced at 10.92 ft (3.33 m) on center. The girders are fabricated from conventional Grade 50 (G345W) steel. It was designed using the 1992 AASHTO 15th Edition LFD Design Specification with the 1993 and 1994 interim and a HS25 design vehicle or 125 percent of the alternative military loading or the P-82 permit load.

#9 - Pennsylvania –Northampton County

The Northampton County Bridge is a single-span composite steel plate girder bridge with a 123 ft (37.5m) span length. It has a 8.5 in (216 mm) reinforced concrete deck supported by 5 girders spaced at 9 ft (2.75m) on center. The girders are fabricated from conventional Grade 50 (G345W) steel. It was designed using the 1992 AASHTO 15th Edition LFD Design Specification with the 1993 and the 1994 interim and a HS25 design vehicle or 125 percent of the alternative military loading or the P-82 permit load.

#10 - Tennessee - Bridge 25SR0520009 - SR 52 over Clear Fork River, Morgan Co

The Clear Fork River Bridge is a four-span continuous composite steel plate girder bridge with span lengths of 145, 220, 350, and 280 ft (44, 67, 106.5, and 85m). It has a 9.25 in (235mm) reinforced concrete deck and 4 hybrid girders spaced at 12 ft (3.66m) on center. The girders use HPS70W (485W) steel in the negative moment regions and in the tension flange in spans 3 and 4. Conventional Grade 50W steel is used in all other locations. It was designed using the 1996 AASHTO LFD Design Specifications and the design vehicle is HS20-44 plus alternate military loading.

#11 - Tennessee - Bridge No. 44SR0530001 SR 53 over Martin Creek

The Martin Creek Bridge is a 2-span continuous composite steel plate girder bridge with equal spans of 235. 5 ft (71.8m). It has a 9 in reinforced concrete deck (slab + wearing course) and 3 HPS70W (485W) steel girders spaced at 12 ft (3.66m) on center. It was designed using the 1994 AASHTO LRFD Design Specifications with the HL93 design loading. Live-load deflection limits were not imposed in the design of this bridge, and no reported structural or serviceability problems have been noted to date.

#12 - Utah –Asay Creek Bridge in Garfield County

The Asay Creek Bridge is a simple span composite steel plate girder bridge with a span length of 76.125 ft (14.266m). It has a 8 in (205mm) reinforced concrete deck and 6 Grade 250 steel (Fy=36 ksi) girders spaced at 7.83 ft (2.4m) on center. The 1996 AASHTO LFD Design Specifications and Interim and a HS20 (MS-18) design vehicle or alternative loading were used in the design.

6.4 Analysis Results

6.4.1. Relationship Between Deflection and $\frac{L}{D}$ Ratio

The bridges deflections were computed. Figure 6.1 shows the dependence of span length to deflection ratio, $\frac{L}{\delta}$ ratio, on the $\frac{L}{D}$ ratio selected by the designer It is clear that larger $\frac{L}{D}$ ratios will normally result in larger live-load deflections. Studies (Clingenpeel, 2001, Horton, R., 2000) have shown HPS 70W girders may be very economical where depth restrictions are mandated due to site restrictions or where it may be advantageous to use reduces superstructure depths to increase clearances and reduce require substructure requirements. Present AASHTO deflection limits reduce the economic potential of HPS may in these applications.

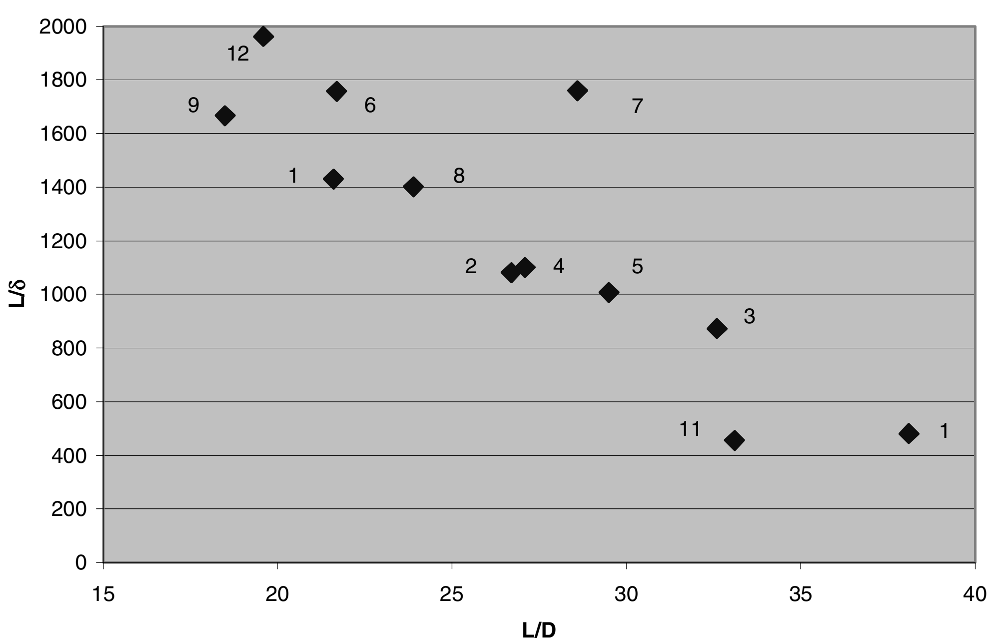

Figure 6.1 $\frac{L}{\delta}$ vs $\frac{L}{D}$ for Typical Highway Bridges

6.4.1. Comparisons with AASHTO Standard Specifications

Table 6.2 presents a summary of the maximum live-load deflections, the computed $\frac{L}{\delta}$ ratio for each of the 12 bridges, the $\frac{L}{D}$ ratio for each bridge, and the maximum allowable deflection at the $\frac{L}{800}$ deflection limit. The calculated $\frac{L}{D}$ ratios shown in Table 6.2 are based on the full span length of the span in which the maximum deflection was calculated divided by the total superstructure depth (i.e. bottom flange + web + haunch + deck thickness. Table 6.2 shows Bridges 10 and 11 (both the Tennessee structures) fail the AASHTO deflection limits with $\frac{L}{\delta}$ ratios of 481 and 456. These structure also had the highest $\frac{L}{D}$ values of all the bridges in the study, 38.1 and 33.1 respectively.

Table 6.2 Comparisons with AASHTO Standard Specifications

Bridge Identification	Actual $\frac{L}{D}$	δmax (in.)	$\frac{L}{\delta}$ max	$\frac{L}{800}$ deflection
1	21.6	0.872	1430	1.559
2	26.7	1.436	1082	1.943
3	32.6	3.232	873	3.525
4	27.1	1.640	1101	2.258
5	29.5	1.190	1008	1.500
6	21.7	0.915	1757	2.010
7	28.6	1.248	1760	2.745
8	23.9	1.806	1402	3.165
9	18.5	0.886	1666	1.845
10	38.1	8.729	481	5.250
11	33.1	6.180	456	3.525
12	19.6	0.465	1961	1.140

6.4.2. Comparison to Walker and Wright Recommendations

The Wright and Walker recommendations (Wright and Walker, 1971) determine an effective allowable peak acceleration based on the fundamental natural frequency along with a speed parameter and an impact factor. The value of this peak acceleration is then compared against tabulated limits that suggest a potential level of user comfort that may be expected. If the peak acceleration exceeds 100 in./sec^2 the member is to be redesigned such that this limit is not exceeded. This procedure is detailed in Chapter 2.

Table 6.3 shows a comparison of the computed peak accelerations for each of the twelve bridges; none of the bridges were found to be unacceptable. A comparison between the predicted accelerations and the $\frac{L}{\delta}$ values for each of the bridges indicates that there is no correlation between predicted $\frac{L}{\delta}$ values and vibration performance related the Walker and Wright procedure. For example, Bridge 7 has an $\frac{L}{\delta}$ of 1760 but is found to be categorized as ‘Perceptable to Most”based on Walker and Wright's procedures, while Bridge 10 with an $\frac{L}{\delta}$ of 481 (far below the allowable AASHTO limit) is found to be categorized as on ‘Perceptible.” Further, Bridge 3 with and $\frac{L}{\delta}$ of 873 is categorized as ‘Unpleasant to Few”, while Bridge 1 with an $\frac{L}{\delta}$ of 1430 (considerably above the require AASHTO limit) has the same vibration sensitivity. While it is not suggested by the authors that the Walker and Wright criteria is the most valid measure of superstructure vibration acceptability, these trends indicate that there is no direct relationship between superstructure deflections and vibration serviceability.

Table 6.3. Comparisons with Wright and Walker Alternative Serviceability Criteria

Bridge Identification	δmax (in.)	f (Hz.)	$\frac{L}{\delta}$ max	a in/sec^2	Wright and Walker Human Response
1	0.87	3.12	1430	80.68	Unpleasant to few
2	1.44	2.10	1082	38.38	Perceptable
3	3.23	1.11	873	64.82	Unpleasant to few
4	1.64	1.91	1101	52.80	Unpleasant to few
5	1.19	2.07	1008	36.14	Perceptable
6	0.92	2.39	1757	18.38	Perceptable to Most
7	1.25	1.66	1760	16.90	Perceptible to most
8	1.81	1.53	1402	29.12	Perceptible
9	0.89	2.93	1666	32.24	Perceptable
10	8.73	0.65	481	21.11	Perceptible
11	6.18	0.69	456	17.79	Perceptible to most
12	0.47	4.75	1961	63.09	Unpleasant to few

6.4.3. Comparison with the Ontario Highway Bridge Design Code

Table 6.4 presents the deflections calculated using the procedures specified in the Ontario Highway Bridge Code along with the natural frequency calculated using Eqn. 2.5. This table also shows the performance criteria that each of the respective structures would be classified in based on the Ontario specifications. Figure 6.2 provides a graphical presentation of the data from Table 6.4.

Bridges 1 and 12 come closest to failing the Ontario Highway Bridge Code procedures, but these bridges had lower $\frac{L}{D}$ ratios (21.6 and 19.6, respectively) and larger

$\frac{L}{\delta}$ ratios (1430 and 1961, respectively, see Table 6.2) than many of the typical bridges in this study.

Table 6.4. Comparisons with Ontario Highway Bridge Design Code

Bridge Indentification	δ max (in.) [1]	f (Hz.) [2]	Criterion Satisfied
1	1.169	3.12	Without Sidewalks
2	2.091	2.10	Without Sidewalks
3	2.909	1.11	With Sidewalks,Little Ped. Use
4	2.085	1.91	Without Sidewalks
5	1.691	2.07	Without Sidewalks
6	0.959	2.39	With Sidewalks,Little Ped. Use
7	1.198	1.66	With Sidewalks,Little Ped. Use
8	0.837	1.53	With Sidewalks,Sig. Ped. Use
9	0.913	2.93	Without Sidewalks
10	3.396	0.65	With Sidewalks,Sig. Ped. Use
11	4.169	0.69	With Sidewalks,Little Ped. Use
12	0.576	4.75	Without Sidewalks

It may also be noted that bridges 10 and 11, which were specifically designed with disregard for the deflection limit (i.e., in both cases the lane load deflections exceeded $\frac{L}{800}$, but all other strength and serviceability criteria were met), were found to almost meet the highest level of bridge vibration criteria. Figure 6.2 suggests that there is not a clear relationship between the $\frac{L}{\delta}$ and implied user comfort ratings. For example, Bridges 6 and 7 have the largest $\frac{L}{\delta}$ ratios, but they do not provide the 'best" performance as suggested by the Ontario recommendations. There is no dependent trend seen in this figure between $\frac{L}{\delta}$ and performance rating.

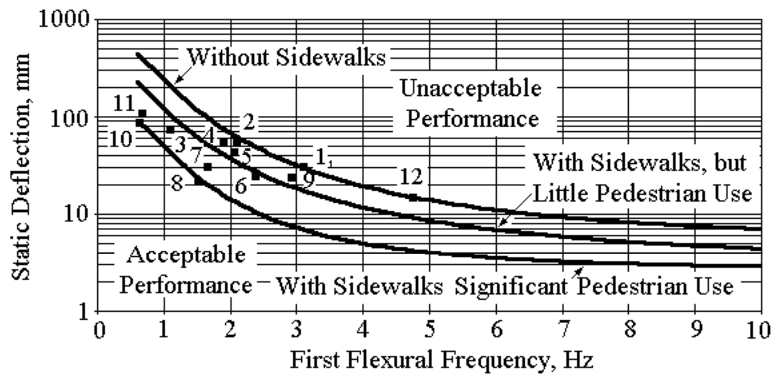

Figure 6.2. Deflection and vibration characteristics of existing bridges comparing to Ontario Highway Bridge Design Code

6.5 Concluding Remarks

Two of the twelve structures in this section failed to meet respective AASHTO criteria, both bridges were designed disregarding the criteria. The discrepancies observed between live-load deflections and vibration performance are an indicator that, as has been reported by others, that the AASHTO deflection limits as they are posed, are not a practical design limit to control bridge vibration.

Both the Walker and Wright and Ontario Highway Bridge Design code depend on the accuracy of the prediction of the fundamental natural frequency. In both cases, they use the standard equation for the natural frequency of a simply supported beam. However, this expression is not specifically applicable for continuous spans. Closed form solutions are not readily available for typical design configurations because of the variation in bridge cross section as a function of length. While empirical expressions exist for the calculation of natural frequencies in continuous spans (Billing, 1979), little documentation is available to relate this to the actual vibration periods of typical bridge superstructures.

Bridges 10 and 11 exceed the AASHTO deflection limit requirements, but there have been no reports of rider discomfort or of structural damage. Results of this study suggest that there is little relationship between a direct limit state check on live-load deflection and the suitability of a given structure to provide acceptable levels of user comfort.

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Chapter 7

Parametric Design Study

7.1 Introduction

A design optimization study to evaluate the impact of bridge deflection limits on the economy and performance of resulting bridge designs was completed. A matrix of bridges representing a wide range of steel bridge designs and considering key design parameters such as span length, girder spacing, and cross-section geometry was developed. Bridges were designed for combinations of these variables based on a least weight approach using various commercial bridge design software. Initial designs disregarded AASHTO live-load deflection limits, but met all other relevant AASHTO strength and serviceability requirements. Initial designs that failed the deflection criteria were then redesigned such that the live-load deflections were less than $\frac{L}{800}$. Comparisons were made between the initial girder weight and that of the redesigned girder to determine additional steel requirements needed for girders to meet the AASHTO limits. While it is recognized that the least weight design is not always the most economical or practical design, this comparison provides evidence of the effect of the deflection limit on bridge economy.

This parameter study also provided information regarding interaction between various combinations of design variables and current deflection limits. Additionally, girder designs generated in this parametric study are compared to two alternative serviceability criteria provided by Wright and Walker (Wright and Walker, 1972) and the Ontario

Highway Bridge Design Code (Ministry of Transportation, 1991). These criteria are presented in Chapter 2.

7.2. Methodolgy

The majority of the design studies used the AASHTO LFD Specifications (AASHTO, 1996), but a subset used the AASHTO LRFD Specifications (AASHTO, 1998). The LFD bridge designs were completed using a steel bridge design optimization program, SIMON (SIMON SYSTEMS, 1996), and the LRFD designs were performed using MDX (MDX software). These are commercially available bridge design packages that perform complete analysis and design for given input parameters. Extensive hand calculations were performed to verify program output including shear and moment envelopes as well as respective strength and serviceability limit state calculations. Several iterations were typically conducted for a given set of design variables for the initial designs generated by the software in order to develop a more practical design. For example, sometimes it was necessary to reduce the number of plate thickness transitions or to make minor changes to plate widths to produce cleaner designs.

To begin a design, a preliminary superstructure depth based on the targeted $\frac{L}{D}$ was calculated. Once the preliminary superstructure depth, *D*, was calculated, the structural thickness of the deck, the haunch, and the bottom flange was subtracted to achieve the web depth, h. From this web depth, an initial flange width was selected such that web depth to compression flange width, $\frac{h}{b_f}$, ratio fell in the range of 3.00 to 4.5. This target range for the $\frac{h}{b_f}$ ratios resulted from previous research (Barth and White 2000). It was not possible to remain with this range for all designs, and the maximum permitted variation was between 2 and 5. After a preliminary girder was chosen, the appropriate noncomposite and composite dead loads were calculated. The preliminary information was input into the respective design package to obtain an optimized section.

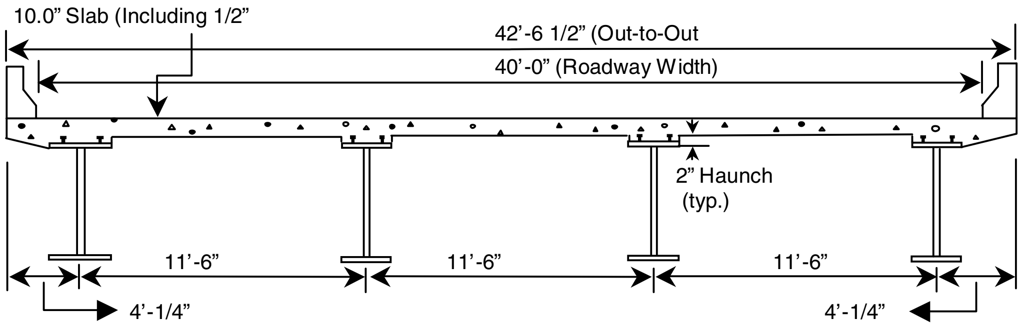
For the simple span designs, a flange thickness transition was included 20% away from each abutment if a weight savings of 900 lbs or more was achieved. In the negative moment region of the two-span continuous bridges, a flange thickness transition was included 15-ft away from the pier if a weight savings of more than 900 lbs was achieved.

The web thickness, t_w, was initially based on the thickness required such that no transverse stiffeners are needed. This initial thickness was then reduced by 1/16-in. to 1/8-in., depending upon the resulting stiffener layout and weight savings. The resulting web thickness was held constant for full length of a given girder.

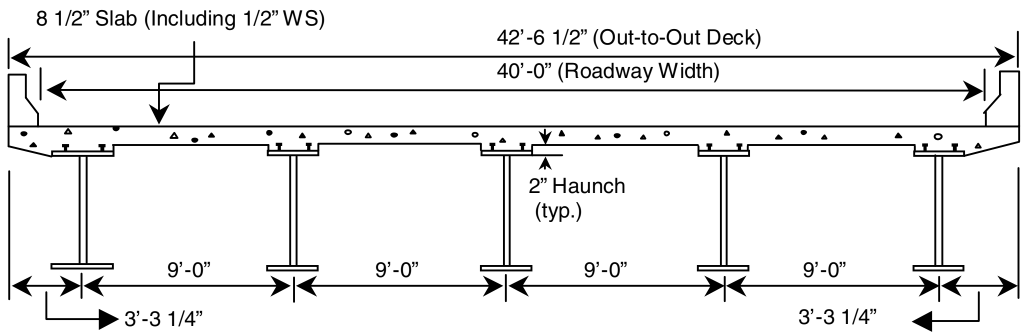
The haunch (which includes the top flange thickness) was assumed to be 2 in. unless section requirements mandated that the top flange thickness be greater than 2 in. In these cases, the haunch was increased to the thickness of the top flange.

7. 3. Design Parameters

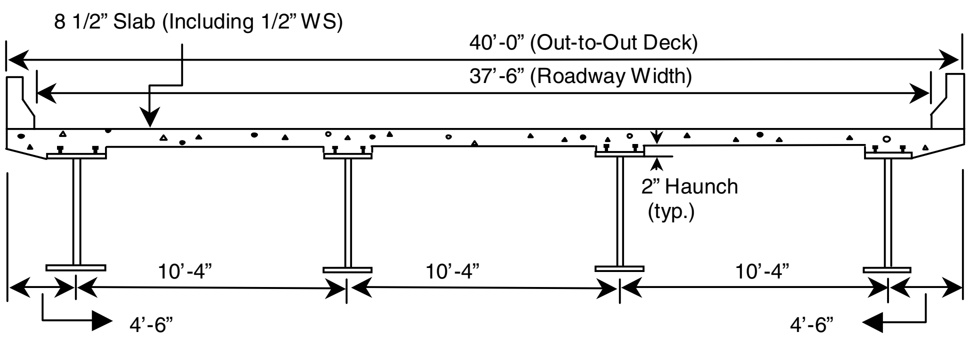
Table 7.1 shows a matrix of design variables that were selected for four representative bridge cross sections. Figure 7.1 shows each of the four cross sections selected to investigate the influence of both the number of lanes and the number of girders.



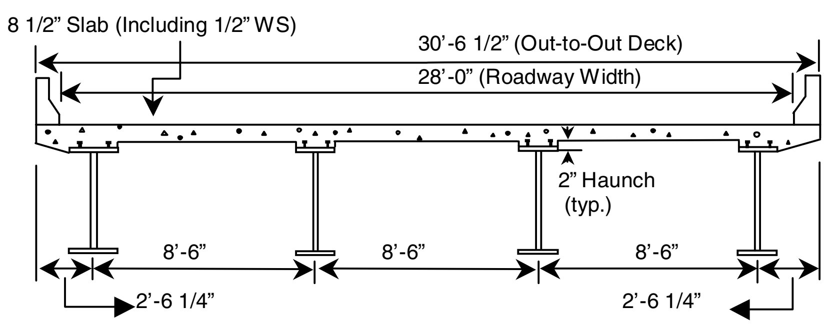
a. Cross-Section # 1



b. Cross-Section # 2



c. Cross-Section # 3



d. Cross-Section # 4

Figure 7.1 Cross-sectional Geometry for 4 Bridge Arrangements

Table 7.1 Matrix of Initial Parameters

Cross Section	Span Length, L (ft.)	Steel Strength, Fy (ksi.)	$\frac{L}{D}$ Ratio	Girder Spacing, S	Span Configuration
1	100, 200, 300	50, 70	15, 20, 25, 30	9'-0"	Simple, 2-span
2	100, 200, 300	50, 70	15, 20, 25, 30	11'-6"	Simple, 2-span
3	100, 150, 200, 250	50, 70	15[1], 20[1], 25[1], 30[1]	10'-4"	Simple, 2-span
4	100, 200, 300	50, 70	15, 20, 25, 30	8'-6"	Simple, 2-span

A number of parameters were held constant throughout the study. These include:

- HS25 live loading for LFD or HL93 live loading for LRFD,
- Stay in place metal forms = 15 psf,
- Future wearing surface = 25 psf,
- Parapet weight = 505 lb./ft.,
- Cross frame spacing = 25 ft.,
- 5% increase in dead weight for miscellaneous steel,
- Interior girder design,
- Class I roadway, and
- Constant flange widths.

Parameters that describe the cross-section of the bridge and the members and material parameters were varied throughout the study as illustrated in Fig. 7.1 and Table 7.1. The study considered simple and two-span continuous bridges with span lengths ranging from 100 to 300 ft (30.5 to 91.4 m). Four $\frac{L}{D}$ ratios between 15 and 30 were investigated. L was defined as the total span length for simple spans and the length between dead load contraflexure points for continuous bridges in determination of the $\frac{L}{D}$ ratios. Bridges were designed with HPS70W and conventional Grade 50W (G345W) steels.

7.4. Results

The combinations of material and geometric parameters described above and summarized in Table 7.1 and Fig. 7.1 yield an initial set of 272 girder designs. Twenty nine of these initial 272 LFD designs did not satisfy the AASHTO live-load deflection limit.

Tables 7.2 through 7.5 present design summary information for initial girder designs that failed to meet the AASHTO deflection limit. For the LFD designs, two HS25 trucks were placed on the bridge and impact was I=50/(L+125). For the LRFD examples, an HS20 trucks and IM=1.33 was used. The full width of the deck slab was used to compute the girder section properties with n=8, and all girders were assumed to carry the live load equally for analysis in this chapter. For each initial girder design shown in these tables, the following line (shown in italics) presents information for the redesigned performed to meet the deflection limit. This table also presents the Walker and Wright vibration classification for both the initial designs as well as the girder redesigns.

7.4.1. Effect of Variations in Geometric and Material Properties

All bridges having a target $\frac{L}{D}$ of either 15 or 20 satisfied the AASHTO deflection limit. However, as the $\frac{L}{D}$ ratio is increased to 25 and 30, an increasing number of structures fail to meet the AASHTO limits. This can be illustrated by noting:

Table 7.2 Comparison of Initial Girder Designs with Girders Not Meeting the Deflection Criteria for Cross-section # 1

Span (ft)	Fy (ksi)	$\frac{L}{D}$	$\frac{L}{\delta}$	Weight[1] (tons)	f_b[2] (Hz)	a[3] (in/sec^2)	Classification[3]
simple spans							
100	70	30.4	629	15.0	2.07	44.680	Perceptible
100	*70*	*30.0*	*815*	*26.5*	*2.22*	*38.076*	*Perceptible*
200	70	30.1	711	50.0	1.22	26.502	Perceptible
200	*70*	*30.0*	*808*	*58.9*	*1.27*	*24.714*	*Perceptible*
300	70	29.6	774	126.0	0.91	19.764	Perceptible to Most
300	*70*	*25.3*	*806*	*144.7*	*0.90*	*18.658*	*Perceptible to Most*

Notes:
[1] weight is for one steel girder
[2] natural frequency computed using Eqn. 6.1
[3] parametric based on Wright and Walker (Wright and Walker, 1971)

Table 7.3 Comparison of Initial Girder Designs with Girders Not Meeting the Deflection Criteria for Cross-section # 2

Span (ft)	Fy (ksi)	$\frac{L}{D}$	$\frac{L}{\delta}$	Weight[1] (tons)	f_b[2] (Hz)	a[3] in/sec^2	Classification[3]
simple span							
100	70	30.1	615	11.0	2.22	63.116	Unpleasant to Few
100	*70*	*25.1*	*806*	*19.7*	*2.39*	*53.542*	*Unpleasant to Few*
200	70	30.1	671	38.0	1.27	37.229	Perceptible
200	*70*	*25.0*	*802*	*48.9*	*1.34*	*33.711*	*Perceptible*
300	70	29.9	716	102.0	0.92	27.143	Perceptible
300	*70*	*25.6*	*815*	*130.6*	*0.93*	*56.838*	*Perceptible*
100	50	30.3	657	12.0	2.28	61.337	Unpleasant to Few
100	*50*	*30.1*	*821*	*19.5*	*2.41*	*53.210*	*Unpleasant to Few*
200	50	30.0	768	44.0	1.33	34.821	Perceptible
200	*50*	*30.0*	*802*	*46.2*	*1.35*	*34.072*	*Perceptible*
2 span continuous							
300	70	29.6	774	184.6	0.67	15.863	Perceptible to Most
300	*70*	*29.7*	*801*	*188.6*	*0.68*	*15.658*	*Perceptible to Most*

Notes:
[1] weight is for one steel girder
[2] natural frequency computed using Eqn. 6.1
[3] parametric based on Wright and Walker (Wright and Walker, 1971)

Table 7.4 Comparison of Initial Girder Designs with Girders Not Meeting the Deflection Criteria for Cross-section # 3

Span (ft)	Fy (ksi)	Design method	$\frac{L}{D}$	$\frac{L}{\delta}$	Weight[1] (tons)	f_b[2] (Hz)	a[3] in/sec2	Classification[3]
simple spans								
100	50	LFD	25.3	726	11.42	2.54	51.991	Unpleasant to Few
100	*50*	*LFD*	*25.1*	*811*	*14.06*	*2.64*	*49.237*	*Perceptible*
100	50	LFD	30.0	628	14.93	2.27	51.036	Unpleasant to Few
100	*50*	*LFD*	*29.7*	*808*	*26.25*	*2.44*	*44.051*	*Perceptible*
100	50	LRFD	30.5	638	11.90	2.10	44.954	Perceptible
100	*50*	*LRFD*	*30.0*	*802*	*15.40*	*2.28*	*40.206*	*Perceptible*
100	70	LFD	25.1	734	10.86	2.57	52.254	Unpleasant to Few
100	*70*	*LFD*	*25.1*	*800*	*12.19*	*2.66*	*50.460*	*Unpleasant to Few*
100	70	LRFD	25.3	752	8.00	2.36	45.052	Perceptible
100	*70*	*LRFD*	*25.1*	*864*	*9.00*	*2.51*	*42.909*	*Perceptible*
100	70	LFD	30.0	548	12.72	2.19	55.584	Unpleasant to Few
100	*70*	*LFD*	*29.7*	*806*	*25.45*	*2.45*	*44.374*	*Perceptible*
100	70	LRFD	30.5	582	9.50	2.05	47.629	Perceptible
100	*70*	*LRFD*	*30.0*	*824*	*15.0*	*2.35*	*40.902*	*Perceptible*
150	50	LFD	30.2	711	26.27	1.72	37.430	Perceptible
150	*50*	*LFD*	*29.5*	*817*	*37.52*	*1.73*	*32.830*	*Perceptible*
150	70	LFD	24.9	723	20.17	1.77	38.410	Perceptible
150	*70*	*LFD*	*24.9*	*810*	*23.83*	*1.82*	*35.687*	*Perceptible*
150	70	LFD	29.8	567	21.10	1.57	41.068	Perceptible
150	*70*	*LFD*	*29.8*	*840*	*41.14*	*1.70*	*32.333*	*Perceptible*
150	70	LRFD	29.7	731	16.9	1.55	31.307	Perceptible
150	*70*	*LRFD*	*29.5*	*819*	*19.2*	*1.65*	*30.563*	*Perceptible*
200	50	LFD	29.9	716	48.95	1.38	31.519	Perceptible
200	*50*	*LFD*	*29.9*	*801*	*65.66*	*1.36*	*27.585*	*Perceptible*
200	70	LFD	25.0	729	36.82	1.43	32.669	Perceptible
200	*70*	*LFD*	*25.0*	*803*	*44.51*	*1.44*	*30.064*	*Perceptible*
200	70	LFD	30.0	571	37.48	1.27	35.003	Perceptible
200	*70*	*LFD*	*29.9*	*801*	*65.66*	*1.36*	*26.783*	*Perceptible*
250	70	LFD	24.9	777	69.24	1.21	27.164	Perceptible
250	*70*	*LFD*	*25.1*	*804*	*74.59*	*1.19*	*28.831*	*Perceptible*
250	70	LFD	30.0	578	63.34	1.07	30.371	Perceptible
250	*70*	*LFD*	*29.9*	*802*	*101.85*	*1.13*	*26.752*	*Perceptible*
2 span continuous								
150	50	LFD	24.9	765	56.88	1.27	22.539	Perceptible
150	*50*	*LFD*	*24.9*	*900*	*75.39*	*1.29*	*19.580*	*Perceptible to Most*
150	50	LFD	30.0	623	76.88	1.07	21.870	Perceptible
150	*50*	*LFD*	*30.0*	*845*	*111.65*	*1.15*	*17.788*	*Perceptible to Most*
150	50	LRFD	30.1	710	62.3	1.01	17.755	Perceptible to Most
150	*50*	*LRFD*	*30.0*	*818*	*67.4*	*1.07*	*16.662*	*Perceptible to Most*

Table 7.4 Continued

Span (ft)	Fy (ksi)	Design method	$\frac{L}{D}$	$\frac{L}{\delta}$	Weight[1] (tons)	f_b[2] (Hz)	a [3] in/sec2	Classification [3]
2 spans Continuous (Cont')								
150	70	LFD	24.8	739	43.78	1.34	25.155	Perceptible
150	*70*	*LFD*	*24.9*	*812*	*54.38*	*1.35*	*23.144*	*Perceptible*
150	70	LFD	30.0	575	55.68	1.12	25.235	Perceptible
150	*70*	*LFD*	*30.0*	*845*	*111.65*	*1.16*	*18.820*	*Perceptible to Most*
150	70	LRFD	30.0	781	53.6	1.09	17.893	Perceptible to Most
150	*70*	*LRFD*	*30.0*	*816*	*55.4*	*1.15*	*18.423*	*Perceptible to Most*
200	50	LFD	24.9	728	100.27	0.96	18.395	Perceptible to Most
200	*50*	*LFD*	*24.9*	*805*	*109.20*	*0.99*	*17.376*	*Perceptible to Most*
200	50	LFD	29.5	669	132.12	0.86	17.185	Perceptible to Most
200	*50*	*LFD*	*29.5*	*905*	*179.77*	*0.92*	*13.946*	*Perceptible to Most*
200	70	LFD	25.0	647	75.97	0.97	21.013	Perceptible
200	*50*	*LFD*	*25.7*	*822*	*107.72*	*1.02*	*17.738*	*Perceptible to Most*
200	70	LFD	29.7	522	90.45	0.84	21.337	Perceptible
200	*70*	*LFD*	*29.5*	*816*	*157.38*	*0.92*	*15.466*	*Perceptible to Most*
250	50	LFD	30.0	720	224.77	0.71	19.641	Perceptible to Most
250	*50*	*LFD*	*30.0*	*804*	*165.60*	*0.75*	*12.029*	*Perceptible to Most*
250	70	LFD	25.1	630	126.35	0.78	17.699	Perceptible to Most
250	*70*	*LFD*	*25.5*	*827*	*178.75*	*0.83*	*14.701*	*Perceptible to Most*
250	70	LFD	30.0	498	148.23	0.66	17.773	Perceptible to Most
250	*70*	*LFD*	*30.0*	*804*	*239.59*	*0.75*	*13.127*	*Perceptible to Most*

Notes:
[1] weight is for one steel girder
[2] natural frequency computed using Eqn. 6.1
[3] parametric based on Wright and Walker (Wright and Walker, 1971)

Table 7.5 Comparison of Initial Girder Designs with Girders Not Meeting the Deflection Criteria for Cross-section # 4

Span (ft)	Fy (ksi)	$\frac{L}{D}$	$\frac{L}{\delta}$	Weight[1] (tons)	f_b[2] (Hz)	a [3] (in/sec^2)	Classification [3]
simple spans							
100	70	29.2	743	12.3	2.37	68.844	Unpleasant to Few
100	*70*	*29.1*	*802*	*14.6*	*2.42*	*65.773*	*Unpleasant to Few*
200	70	29.5	732	35.5	1.33	43.803	Perceptible
200	*70*	*29.4*	*801*	*38.9*	*1.38*	*42.268*	*Perceptible*
300	70	29.3	781	105.1	0.95	31.306	Perceptible
300	*70*	*29.8*	*812*	*107.8*	*0.96*	*30.620*	*Perceptible*

Notes:
[1] weight is for one steel girder
[2] natural frequency computed using Eqn. 6.1
[3] parametric based on Wright and Walker (Wright and Walker, 1971)

- For 61 designs with $\frac{L}{D}$ of approximately 25 (i.e. $\frac{L}{D}$ between 24 and 26), 9.8% failed the AASHTO deflection limit.
- For 60 designs with $\frac{L}{D}$ of approximately 30, 45% failed the AASHTO deflection limit.
- Simple span girders were found to be more likely to fail the AASHTO deflection limit than 2-span continuous girders, since 82% of those failing the AASHTO deflection limit were simple span girders. This may be partially attributed to the procedure used to establish the $\frac{L}{D}$ ratio for continuous girders.

It is relevant to note that plots of girder weight versus $\frac{L}{D}$ ratio would show the optimum weight to be developed at an $\frac{L}{D}$ of approximately 25. This $\frac{L}{D}$ ratio is also the recommended value specified in AASHTO. Structures with larger $\frac{L}{D}$ ratios are the most severely affected by the deflection limits. However, these structures are routinely used in depth restricted applications.

Span length may also be an issue of concern. Simple span bridges with ratios in the range of 24 and above were evaluated separately, since these are the bridges more susceptible to failing the AASHTO deflection limit. This comparison shows that:

- 79% of bridges with a 100 ft (30.5 m) span length failed the AASHTO deflection limit,

- 40% of bridges with a 200 ft (71 m) span length failed the AASHTO deflection limit, and
- 25% of bridges with a 250 ft (76.2 m) or longer span length failed the AASHTO deflection limit.

Also, the yield strength of the steel was found to have a clear impact upon the deflection limit. This is illustrated by noting:

- 75% of the bridges that failed the AASHTO deflection limit were designed of HPS 70W steel, but
- continuous 2 span bridges with grade 50W steel failed the deflection limit with approximately the same frequencey as HPS 70W steel.

7.4.2. Comparison of Re-Designs

As noted earlier, initial designs with deflections exceeding the $\frac{L}{800}$ limit were re-designed to meet the deflection limit. Doing so naturally decreased the overall performance ratio of the girder and the demand/capacity ratio for all other design criteria. The performance ratios were larger than 0.985 (but less than 1.0) for all initial designs. However, these ratios fell as low as 0.887 for the redesigns.

Figure 7.2 shows a plot of the deflections for a 150 ft simple span bridge for cross section 3 for a range of $\frac{L}{D}$ limits for both 50 and 70 ksi designs. This figure shows values for the LFD studies. Again, this figure shows that no initial girder design with $\frac{L}{D}$ of 15 or 20 fails the AASHTO deflection criteria. However, at $\frac{L}{D}$ = 25, the 70 ksi design fails to meet the limit and at $\frac{L}{D}$ = 30 both the 50 and 70 ksi design fails to meet the limits. Figure 7.3 shows a plot of the total girder weight for both the initial and redesigns for the same example. While the increase in required steel weight at $\frac{L}{D}$ = 25 was negligible, at $\frac{L}{D}$ = 30 a substantial increase in steel weight was required for a given girder to meet the deflection limit.

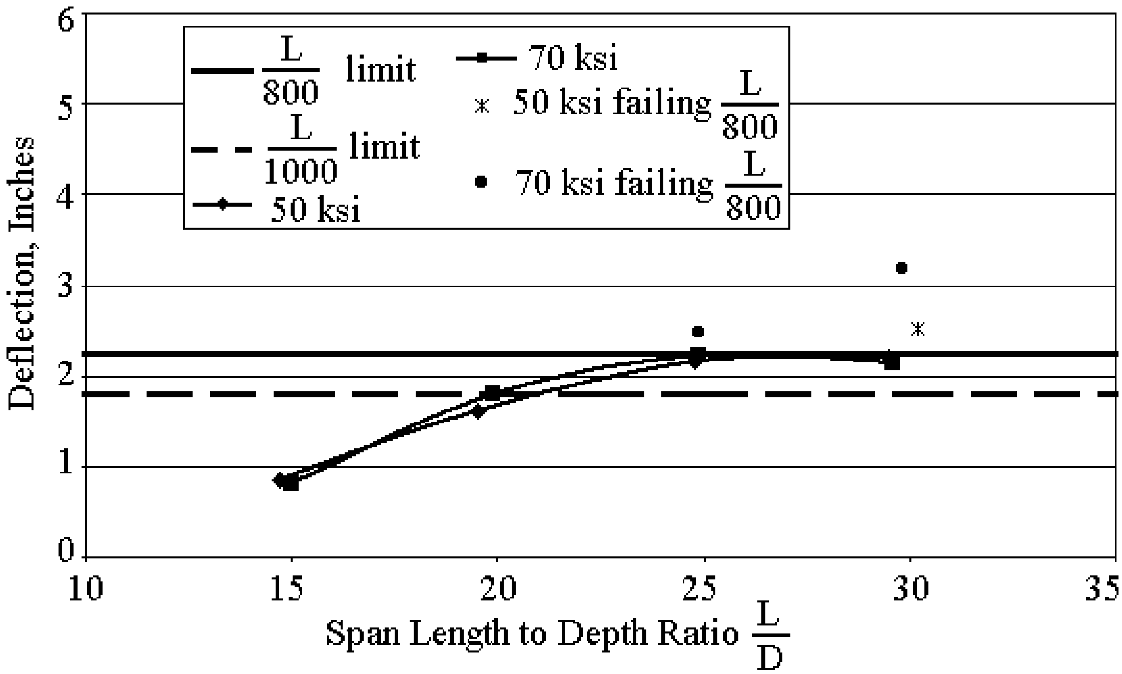


Figure 7.2. Deflection Versus $\frac{L}{D}$ for 150 ft/ Simple Span Bridge with Cross-section # 3

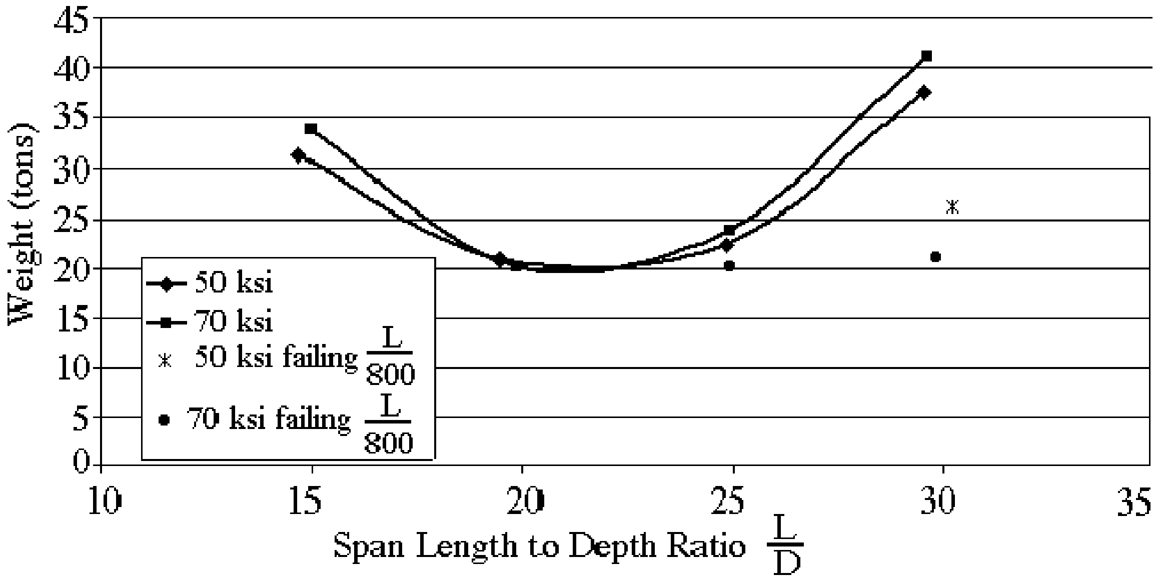


Figure 7.3. Weight Versus $\frac{L}{D}$ for 150 ft/ Simple Span Bridge with Cross-section # 3

Again, Tables 7.2 through 7.5 show design summary values for both the original design failing to meet the AASHTO criteria as well as the associate redesigns. On average, 36% more steel was required to meet the given deflection limits. This increase was the highest for the continuous span structures and lowest for the longer span simple-span bridges. Naturally, these numbers may vary based on design input, but it is clear that substantial cost savings may be possible with the incorporation of alternate serviceability criteria.

7.4.3. Comparison with Alternate Criteria

As noted earlier, a number of foreign specifications place limits on superstructure vibration characteristics rather than live-load deflection. Also, the Wright and Walker report cited earlier is referenced as a footnote in the LRFD specifications. None of the bridges from the initial set of studies was found to exceed the limits developed by Wright and Walker. Most frequently, the girders would be classified as *perceptible* (see Tables

7.2 through 7.5). In fact, in only a few designs were the structures classified as *unpleasant to few.*

Figures 7.4 and 7.5 show plots for the Ontario specifications (Ministry of Transporation, 1991) with data points plotted for those girders initially failing to meet the $\frac{L}{800}$ limit for simple and two span continuous bridges respectively. The Ontario Highway Bridge code limits the static deflection as a function of the first flexural frequency and the intended use. While the majority of bridges were found to fall within the limits for having sidewalks and little pedestrian use, all designs were found to fall within the acceptable range for bridges with no sidewalks.

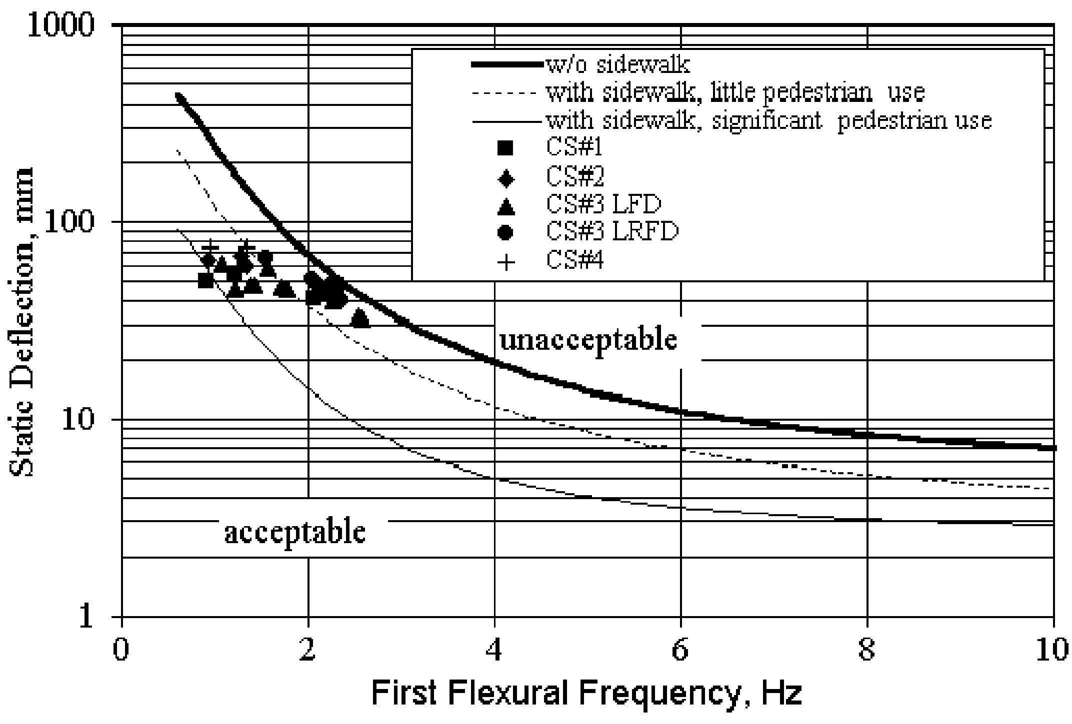


Figure 7.4. Comparison with OHBD Code for Simple Span Girders Failing the $\frac{L}{800}$ AASHTO Deflection Limit

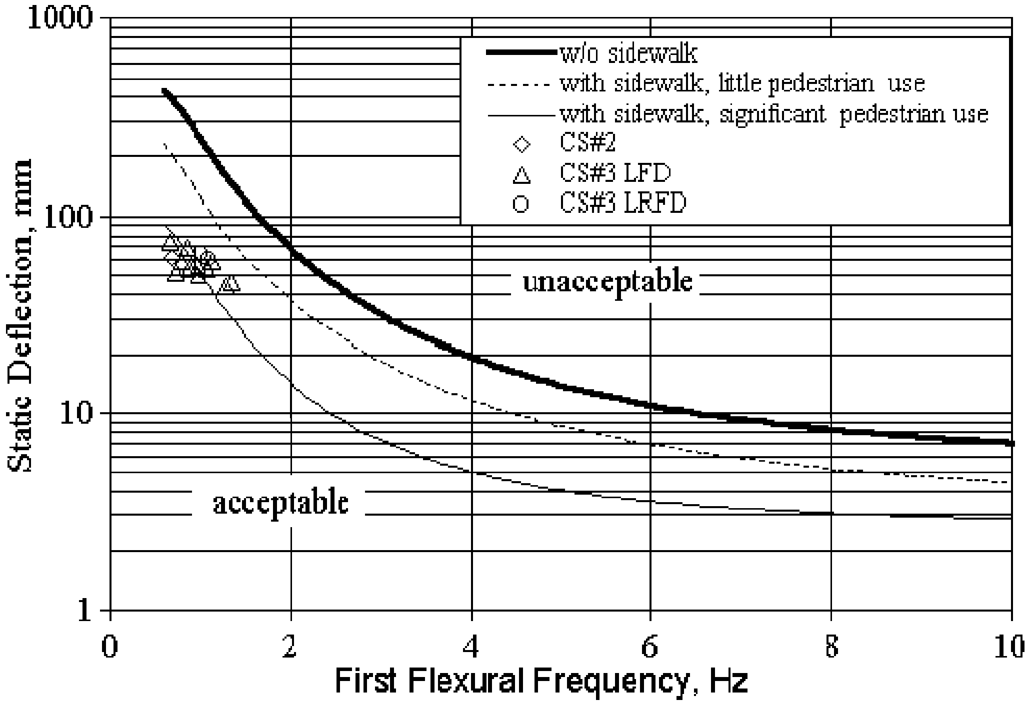


Figure 7.5. Comparison with OHBD Code for 2 Span Continuous Girders Failing the $\frac{L}{800}$ AASHTO Deflection Limit

7.4.4. Comparison of LFD with LRFD

Fifty seven bridges were designed by both LFD and LRFD design with the Cross-section # 3. These bridges covered a wide range of parameters as noted in Table 7.1. Twenty three of these bridges designed with LFD exceed the $\frac{L}{800}$ deflection limit, but only 6 of the LRFD designs exceeded this limit. This is partly due to the design vehicle used for evaluation of the limits. In LFD, it is specified that the vehicle used to evaluate strength must also be used to evaluate serviceability; hence, the HS25 truck loading was used in this evaluation. In LRFD, it is specified that the deflection criteria are to be evaluated using the design truck only, which is the HS20-44. Also, differences in resistance equations, distribution factors, and design loadings produce different geometries for LFD and LRFD. Both methods incorporate the same live-load deflection distribution factor, which is determined by assuming that only any load placed on the structure after deck placement may be assumed to be carried equally by all girders.

7.5. Final Remarks

This discussion shows that the present AASHTO deflection limits may have a significant influence on the girder economy of some ranges of bridge superstructure geometries. Shallower girders with larger $\frac{L}{D}$ ratios, simple span bridges, and bridges designed with HPS70W steel appear to be more seriously influenced by the deflection limit. However, it should be noted that other superstructure geometries may not be as dramatically influenced by the existing criteria.

Chapter 8

Summary, Conclusions and Recommendations

8.1 Summary

This research has examined the AASHTO live-load deflection limits for steel bridges. The AASHTO Standard Specification limits live-load deflections to $\frac{L}{800}$ for ordinary bridges and $\frac{L}{1000}$ for bridges in urban areas that are subject to pedestrian use. While these limits are also given in the AASHTO LRFD specifications, they are posed in the form of an optional serviceability criteria in this document. This limit has not been a controlling factor in most past bridge designs, but it will play a greater role in the design of bridges built with new HPS 70W steel. This study documented the role of the AASHTO live-load deflection limit of steel bridge design, determined whether the limit has beneficial effects on serviceability and performance, and established whether the deflection limit was needed. Limited time and funding was provided for this study, but an ultimate goal was to establish recommendations for new design provisions that would assure serviceability, good structural performance and economy in design and construction.

A literature review was completed to establish the origin and justification for the deflection limits. This review examined numerous papers and reports, and a comprehensive reference list is provided. A survey of state bridge engineers was completed to examine how these deflection limits are actually applied in bridge design. The survey also identified bridges that were candidates for further study on this research issue. Candidate bridges either:

- failed to meet the existing deflection limit,
- exhibit structural damage that was attributable to excessive bridge deflection,
- were designed of HPS steel, or
- had pedestrian or vehicle occupant comfort concerns due to bridge vibration.

The survey showed wide variation in the application of the deflection limit in the various states, and so a parameter study was completed to establish the consequences of this variation on bridge design. The effect of different load patterns, load magnitudes, deflection limits, bridge span length, bridge continuity, and other factors were examined. The survey identified a number of bridges which were experiencing structural damage and reduced service life associated with bridge deflections. Design drawings, inspection reports, photographs, and other information was collected on these bridges. They were grouped and analyzed to:

- determine whether the damage was truly caused by bridge deflections,
- determine whether the AASHTO live-load deflection limit played a role in controlling or preventing this damage, and
- examine alternate methods of controlling or preventing this damage.

Other bridges were also analyzed.

The literature review showed that the existing AASHTO deflection limit was initially introduced as a method of vibration control, and that bridge stiffness and flexibility is not the most rational method for controlling bridge vibrations. Bridge span-to-depth ratios ($\frac{L}{D}$) also were shown to have a major impact on bridge deflection. As a result, additional analyses were performed to examine how the deflection limit interacts with bridge vibration and $\frac{L}{D}$ ratio. The study examined the effect these parameters on the economy and performance of bridge design. A parameter study was completed on 12 typical steel I-girder bridges designed by bridge engineers for 6 different states. In addition, a design optimization study was completed with the aid of standard steel bridge design computer program packages. These studies examined the bridge vibration and deflection issues and considered how deflection and vibration concerns affect the economy of design as well as the bridge performance.

8.2 Conclusions and Recommendations

A number of conclusions and recommendations can be drawn from this work. The conclusions and recommendations of this chapter are based upon $\frac{L}{D}$ ratios where D is the total superstructure depth (i.e. girder depth plus haunch and slab thickness). In addition, span length based deflection limits (i.e. $\frac{L}{800}$) are based upon the actual span length and maximum deflection for the specified load for simple span bridges. For continuous bridges, the span length is the length between inflection points and the deflection is the chord deflection as illustrated in Fig. 4.1. It is recognized that this span length and deflection require additional engineering calculation, but it provides a more consistent serviceability measure between simple span and continuous bridges. Engineers may choose to use an alternate procedure for continuous bridges where the total maximum deflection and the total span length are employed. The alternate procedure uses a larger span length and a larger deflection, but it is easier to compute.

However, it must be recognized that this alternate method will provide a significantly more restrictive serviceability criteria for many continuous bridges.

8.2.1. Conclusions

Several conclusions are worthy of particular note.

1) The existing AASHTO deflection limit was initially instituted to control bridge vibration. Deflection control is not a good method for controlling bridge vibration. Alternate design methods have been developed and are more rational, but there is variability in these methods. Several practical limitations reduce the full design effectiveness of the alternate procedures.
2) There is wide variation in the application of the existing deflection limit. This occurs because of the variation in the actual limits used in evaluation, the variation in the load magnitude and load pattern used to calculated the deflection, the application of load factors and lane load distribution factors, and other effects. The difference between the least restrictive and most restrictive deflection limit may exceed 1,000%. Live-load deflections do not affect many steel bridge designs, but the huge variation reported by the various states show that the effect will be much greater in some states than in others.
3) The load pattern and magnitude have a big impact on the variation noted above. Some states use truckloads, some use distributed lane loads, and some use combinations of the above. Truck loads provide the largest deflection for short span bridges. Distributed lane loads provide the largest deflections for long span bridges.

4) Application of the deflection limit with truck load only shows that the existing AASHTO deflection limits will have a significant economic impact on some steel I-girder bridges built from HPS 70W steel. Simple span bridges are more frequently affected by this limit than continuous bridges. However, continuous bridges are also likely to be more frequently affected by existing deflections if the span length, L, is taken as the true span length rather than the distance between inflection points in the application of the deflection limit.

5) The AASHTO live-load deflection limit is less likely to influence the design of bridges with small $\frac{L}{D}$ ratios and is more likely to control the superstructure member sizes as the $\frac{L}{D}$ ratio increases.

6) A substantial number of bridges are damaged by bridge deformation. This deformation is related to bridge deflection, but the $\frac{L}{800}$ live-load deflection limit is a poor means of controlling this deformation. The deformations that cause the damage are relative deflections between adjacent members, local rotations and deformations, deformation induced by bridge skew and curvature, and similar concerns. None of these deformations are checked in the existing live-load deflection evaluation. Bridge serviceability is an important design consideration, and other methods of assuring serviceability are needed.

7) Many bridges that satisfy the existing deflection limit are likely to provide poor vibration performance, and they may experience structural damage due to excessive deformation. Other bridges, which fail the existing deflection limit, will provide good comfort characteristics and good serviceability.

8.2.2. Recommended Changes to AASHTO Specifications

Two types of recommendations are appropriate here. The first type reflects recommended changes to the AASHTO Specifications. The second type reflects research or additional study that is required to bring these changes to their full fruition. This second type of recommendations are provided in Section 8.3.

The live-load deflection requirements of the AASHTO Standard Specifications require a relatively few words. The interpretation of these words is sometimes ambiguous as noted by the wide variation in practice. The following recommended changes to the AASHTO Specification are made in this context.

1) It is recommended that an immediate change be made to AASHTO Specifications to avoid the ambiguity in interpretation of existing provisions. It is recommended that the deflection check be made with the AASHTO design truck plus impact. The maximum deflection computed when this truck is place at any possible location on the bridge should be considered, but only a single truck should be employed, and the full stiffness of the bridge system should be considered. Distributed lane loads should not be included because they have a very detrimental effect on the design of long span bridges and cause the large variability observed in the application of the deflection limit. Load factors and lane load distribution factors should not be used, because the deflection check is a serviceability check. It is recommended that the span length for the $\frac{L}{800}$ deflection limit be the total span length for simple span bridges and the distance between inflection points (or points of contraflexure) for continuous bridges. It should be noted that this recommendation is essentially the

standard criteria used in Chapters 5, 6 and 7. It will significantly reduce the adverse effect of the deflection limit on the economy of steel bridge design. However, it will not completely eliminate this adverse effect. This interim change is desirable because the existing deflection limit is the primary serviceability criteria in AASHTO Specifications. Engineers hold a strong belief that serviceability criteria is needed, and this study shows that structural damage due to deformation does occur. The AASHTO live-load deflection limit is not a good serviceability criteria, but at present sufficient documentation is not available to warrant removal of this limit.

2) As another immediate change, the $\frac{L}{1000}$ deflection limit for bridges with pedestrian access should be removed from the specification. This report is recommending that the deflection limit be used as an interim serviceability criteria. The sole goal of the $\frac{L}{1000}$ deflection limit for bridges with pedestrian access is vibration control. There is no reason why pedestrian bridges should have more restrictive serviceability criteria than other bridges. Further, bridge deflections are a poor method of assuring vibration control. Other better methods of vibration control are presently available, and these methods can be approximately employed with tools presently available to the design engineer. Until improved vibration control procedures are developed, the method proposed by Walker and Wright [(1971)] and summarized in Chapter 2 is recommended. The $\frac{L}{1000}$ deflection limit is not warranted in view of these factors.

3) As a longer-term recommendation, it is recommended that the live-load deflection criteria be completely eliminated from the AASHTO Specifications. It is not a good method for assuring vibration control or for assuring serviceability and damage

control. There are some limitations that must be addressed to completely accomplish this goal that are discussed in Section 8.3.

4) As a longer-term recommendation, it is recommended that a direct vibration frequency and amplitude control be inserted into the AASHTO Specifications as a method of assuring pedestrian and vehicle occupant comfort and structural damage control. Several tools are needed to fully achieve this goal as discussed in Section 8.3.

8.3. Recommendations for Further Study

Several issues need to be addressed to fully accomplish the recommendations of Section 8.2.

1) Criteria for preventing damage due to bridge deformations are needed. These criteria may be very specific to various bridge types. They may for some cases need to be based upon strain levels or curvature within members. Further, for some cases they may need to be based upon local rotations and deformations, relative deflections between adjacent members, or system behavior.
2) Improved design equations for estimating the frequency of practical bridges are needed. Existing methods of vibration control such as Ontario method or Wright and Walker method depend upon accurate determination of the frequency (or period) of the bridge span and the deflection (or amplitude of vibration). Existing closed form equations are available, but they are for simple spans and consider uniform beam cross section for the entire span length. Simple equations are needed by design engineers to quickly and accurately estimate the frequency, but existing equations are

inaccurate for frequency estimates in bridges with continuous girders. It is also unclear how well existing equations approximate the actual frequency with the flange transitions and member size changes commonly used in bridge design.

3) The structural problems associated with bridge deformation are invariably local effects. Much of this damage occurred in skewed bridges, and the local effects are usually attributable to the system behavior resulting from bridge skew. Skew and curved bridge girders do not behave as the line elements commonly assumed by bridge engineers. This system behavior needs to be better understood if the damage observed on these bridges is to be avoided.

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Appendix A

Sample Survey

and

Summarized State by State Results

Part I. General Information

Date: ____

Time: ____

Agency / DOH: ______________________________________

Name: ____

Position / Title: ___________________________________

Address: ____

____

Phone: ____

E-mail: ____

Other Information: __________________________________

Part II. General Questions

1. What deflection limit do you use in the design of steel bridges?

$\frac{L}{1000}$? $\frac{L}{800}$? None? Other? (Explain) ____________________________

2. For steel girder-concrete deck bridges, what loads do you use for application of this deflection limit?

a) Load magnitude?

Lane Live Load Only? Truck Load Only? Lane Live Load plus Truck Load?

Including Impact? Without Impact?

Factored Loads? Unfactored Loads?

Other? Explain____

b) Lane application?

Single Lane Only? Multiple Lanes? Explain how many____________________

Some distribution factor to bridge girders such as $\frac{S}{5.5}$? Explain _______________

There is no standard practice? Explain____________________________

3. Do you use the deflection limits for components of other steel bridge types such as truss bridges, arch bridges, box girder bridges and etc?

Yes? No?

a) If so, what lane application do you employ?

Single Lane Only?

Multiple Lanes? Explain for major bridge types used in your state_____________

4. For steel girder-concrete deck bridges, what stiffness do you consider when considering the deflection limit?

Single steel girder only without composite slab?

Single steel girder with composite slab stiffness?

Stiffness of the entire multiple girder system without curbs, railings and so forth?

Stiffness of the entire multiple girder system including curbs, railings and etc?

There is no standard practice? Explain______________________________

Other? Explain ____

5. For steel girder-concrete deck bridges, does your state employ a span-depth ratio limit?

Yes? No?

If so, what limit do you employ?_____________________________

6. What analysis procedures do you use?

a) What computer software package?_________________________________

b) When would you resort to a more sophisticated analysis procedure and how would you do it?

Sharply skew bridges? Approximately what skew angle?_______________

Curved Bridges? Approximately what radius?________________________

Other bridge structural systems?____________________________________

6. Has your state design a bridge with HPS steel?

Yes? No?

If so, please identify the bridges?____________________________________

If so, please identify an engineer who can be contacted for more detailed information on that bridge?____

(*Contact this person and complete the more detailed information sheet for these bridges*)

7. Does your state have any steel bridge which have experienced structural damage (excessive deck cracking, cracking of steel or etc) due to excessive deflection or vibration?

Yes? No?

If so, please identify the bridges with the most severe damage and note the type of damage observed on these bridges?____

____

______

If so, please identify an engineer (and phone number if possible) who can be contacted for more detailed information on that bridge?______________________

____

(*Contact this person and complete the more detailed information sheet for specific bridges after the statements have been evaluated*)

8. Does your state have any steel bridge which have objectionable deflection or vibration? (deformations that do not cause structural damage but that are objectionable to drivers or pedestrians)

Yes? No?

If so, please identify the bridges with the most severe response and note the response observed on these bridges?_______________________________________

____

______

If so, please identify an engineer (and phone number if possible) who can be contacted for more detailed information on that bridge?______________________

____

(*Contact this person and complete the more detailed information sheet for specific bridges after the statements have been evaluated*)

9. Does your state have any steel bridge which did not satisfy your state deflection limit but that appear to provide satisfactory performance?

Yes? No?

If so, please identify the bridges?____

____

______

If so, please identify an engineer (and phone number if possible) who can be contacted for more detailed information on that bridge?____________________

____

(*Contact this person and complete the more detailed information sheet for specific bridges after the statements have been evaluated*)

Summary of Survey Response

Though to some degree responses may be qualitative rather than quantitative, Table A.1 attempts to provide a state by state summary of key issues noted from the survey. It must be recognized that this table is not a precise indicator of the answers provided by the interviewee, but the evaluation of the total response. For example, question 4 was to determine -

- whether deflections were computed by using a line girder approach or by analyzing the total bridge system,
- whether stiffness in the deflection calculations included composite action of the girders, or
- whether the stiffness of curbs, railings and sidewalks were included in the calculation.

Individual answers to these individual questions varied widely, but the total effects of the different state responses were often quite similar. This occurred because different states compensated for the various issues at different steps in their evaluation process. The last column of Table A.1 summarizes the consensus of the final effect regarding this issue rather than the individual answers to specific questions.

Table A.1. Summary of General Survey Results

State	Deflection Limits		Span / Depth Ratios	Load Magnitude		Lane Application
	Ped.	Non-Ped.		Load Used	Factored	
Alabama	L / 1000	L / 800	loose AASHTO	HS 20 44 Truck	No	Evaluated with AASHTO lane distribution factor but analyzed as a system if too large
Alaska	L / 1000	L / 800	loose AASHTO	HS 20 Truck + I	No	Evaluated as a system
Arizona	L / 1000	L / 800	AASHTO	HS 20 Truck + I or Lane; whichever governs	No	Evaluated as single girder with lane distribution factors
Arkansas	L / 1000	L / 800	AASHTO	Truck + Lane + I	Yes	Evaluated as system with lane distribution factors
California	L / 800	L / 800	non-composite beams or girders are D/S > 0.04 and composite girders are D/S > 0.045 for simple and 0.04 for continuous	Truck + Lane + I	No	Start with girder analysis but move to system analysis
Colorado	L / 1000	L / 800	strict AASHTO	Truck + Lane + I	No	Evaluated as single girder with lane load distribution factors
Connecticut	L / 1000	L / 800	No	Truck + Lane + I	No	Evaluated as single girder with lane load distribution factors
Delaware	L / 1000	L / 800	AASHTO	HS 25 Truck + I before, now HL 93 Truck + I;	No	Evaluated with AASHTO lane distribution factor but analyzed as a system if too large

Florida	L / 1000	L / 800	AASHTO but may occasionally ignore	Truck + I	No	Effectively system analysis with equal distribution
Georgia	L / 1000	L / 800	AASHTO	Lane + I or Truck + I or Military Load + I; whichever governs	No	Effectively system analysis with equal distribution
Hawaii	Have not designed a steel bridge in 30 + years					
Idaho	L / 800	L / 800	recommend AASHTO	Truck + I	No	Equal distribution with system analysis
Illinois	L / 1000	L / 800	No	Lane + I or Truck + I; whichever governs	No	Effectively system analysis with equal distribution
Indiana						
Iowa	L / 1000	L / 800	No	Truck + Lane + I	No	Evaluated as a single girder with lane load distribution
Kansas	L / 1000	L / 800	No	Truck + I	No	Effectively system analysis with equal distribution
Kentucky	L / 1000	L / 800	AASHTO	HS 20 Truck + Lane + I	No	Start with girder analysis but move to system analysis but use lane load distribution
Louisiana	L / 1000	L / 800	strict AASHTO	Truck + Lane + I	No	Evaluated as a single girder with lane load distribution
Maine	L / 1000	L / 800	strict AASHTO	HS 20 Truck + Lane + I	No	Evaluated as a single girder with lane load distribution

Maryland	L / 1000	L / 800	AASHTO	HS 25 Truck or Lane; whichever governs (respondent did not know if impact was included)	No	Evaluated as a single girder with lane load distribution
Massachusetts	L / 1000	L / 800 as an upper limit but L / 1000 is preferred	strict AASHTO	Truck + Lane + I	No	Evaluated as a single girder with lane load distribution
Michigan	L / 1000	L / 800	loose AASHTO used for preliminary design	HS 25 Truck + I	No	Evaluated as single girder with S/14 lane load distribution -- effectively system analysis with uniform distribution
Minnesota	L / 1200	L / 1000	AASHTO as a preliminary	Truck + I	No	Effectively system analysis with equal distribution
Mississippi	L / 1000	L / 800	AASHTO	Truck + I or Lane + I or Military + I; whichever governs	No Response	Start with single girder and advance to system analysis if needed but with lane load distribution
Missouri	L / 1000	L / 800	AASHTO	Truck + Lane + I	No	Evaluated as a single girder with lane load distribution
Montana	L / 1000	L / 1000	loose AASHTO	Truck + Lane	Yes	Evaluated as system with lane distribution factors
Nebraska	L / 1000	L / 800	AASHTO	Lane + I or HS 25 Truck + I; whichever governs	No	Effectively system analysis with equal distribution

Nevada	L / 1000	L / 800	AASHTO	HS 20 Truck + I for non-NHS roads and HS 25 Truck + I for NHS Roads	No	Evaluated as a single girder with lane load distribution
New Hampshire						
New Jersey	L / 1000	L / 1000	No	HL 93 Truck + I and a Permit Vehicle	No	Evaluated as system with lane distribution factors
New Mexico	L / 1000	L / 800	AASHTO	No set policy, up to design engineer	No set policy, up to design engineer	Evaluated as a single girder with lane load distribution
New York	L / 1000 recommended	L / 800 recommended	AASHTO as a guideline	Truck + I or Lane + I; whichever governs	No	Effectively system analysis with equal distribution
North Carolina	L / 1000	L / 800	AASHTO recommended	Truck + Lane + I	No	Evaluated as a single girder with lane load distribution
North Dakota	L / 1000	L / 800	AASHTO	Truck + Lane + I	No	Effectively system analysis with equal distribution
Ohio	L / 800	L / 800	ratio of 10 to 20	Lane + I	No Response	Evaluated as single girder with lane distribution factors
Oklahoma	L / 1000	L / 800	AASHTO	Truck + Lane + I	Yes	Evaluated as a single girder with lane load distribution
Oregon	L / 800	L / 800	AAHSTO	Truck + I	No	Evaluated as single girder with lane distribution factors

Pennsylvania	L / 1000	L / 800	strict AASHTO	Truck + I	No	Effectively system analysis with equal distribution
Rhode Island	L / 1100	L / 1100	30 to 1	Truck + Lane + I	Yes	Evaluated as a single girder with lane load distribution
South Carolina	L / 1000	L / 800	AASHTO	1.25 times H20 Truck + Lane + I	Yes	Evaluated as a single girder with lane load distribution
South Dakota	L / 1200	L / 1000	AASHTO as a guideline	Truck + I or Lane + I; whichever governs	No	Evaluated as a single girder with lane load distribution
Tennessee	L / 1000 recommended	L / 800 recommended	AASHTO	HS 20 44 Truck + I	No	Effectively system analysis with equal distribution
Texas	L / 1000	L / 800	AASHTO but may deviate some	Truck + I or Lane + I	No	Evaluated as a single girder with lane load distribution
Utah	L / 1000	L / 800	AASHTO	Truck or Lane	No Response	Evaluated as single girder with lane distribution factors
Vermont	L / 1000	L / 1000	AASHTO	HS 25 Truck + I	No	Evaluated as a single girder with lane load distribution
Virginia	L / 1000	L / 800	strict AASHTO	Truck + Lane + I	No	Evaluated as system with lane distribution factors
Washington	L / 1000	L / 800	L/20 for simple spans; L/25 continuous; preliminary guideline	HS 25 Truck + or Lane +I	No	Equal distribution

West Virginia	L / 1000	L / 800	No limit.	HS 25 Truck + or Lane +I	No	Equal distribution including all stiffness contributing elements such as curbs and railings
Wisconsin	L / 1600	L / 1600	no but with L / 1600 deflection will usually control anyway	HS 25 Truck + I	No	Evaluated as system with lane distribution factors
Wyoming	L / 1000	L / 800	No	Truck + Lane + I	Yes	Start with single girder and advance to system analysis if needed but with lane load distribution