ANALYZING AND DESIGNING FOR SUBSTRUCTURE MOVEMENT IN HIGHWAY BRIDGES: AN LRFD APPROACH

by

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ABSTRACT

Movement of bridge substructures can adversely affect the strength and serviceability of bridge superstructures. Research concluded in 1985 utilized field and analytical studies to create tolerable bridge movement limits. Because the limits were determined based on the design and loading provisions of the American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications for Highway Bridges, there was a need for the previous research to be reproduced and updated based on the provisions of the AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications. Simple two-dimensional analytical models and models of actual in-service highway bridges were analyzed in order to study the effects of differential vertical movement. The results obtained from the analyses suggested that, in general, the reserve moment capacities of bridge girders are sufficient to resist the additional stresses caused by differential settlement of bridge substructures.

Despite the ability of most bridges to tolerate small magnitudes of differential vertical substructure movement, expected movements should be accounted for during the design process to ensure that safety and reliability are maintained. A method of calculating the effects of anticipated differential settlements was determined. It is recommended that force effects associated with anticipated differential vertical movements of substructures be included in the design of bridge girders.

Chapter 1

INTRODUCTION

Bridge substructure movements introduce additional stresses in the superstructure and can raise a variety of strength and serviceability issues. Previous studies have shown that differential settlement can cause overstress of bridge girders, reduced rider comfort, concrete cracking, drainage issues, and problems with associated facilities. Horizontal movements also cause issues, which are often more problematic than the issues caused by vertical settlement. Jamming of expansion joints, bearing issues, and damage to abutment walls can be caused by horizontal substructure movements. The problems caused by these movements can limit the life of a bridge or create the need for costly repairs (Samtani and Nowatzki, 2006).

The effects of substructure movement are not typically considered during the bridge design process. Typically, engineers are required to design foundations that will prevent damaging deformations of the bridge superstructure. Limiting foundation movements in an attempt to mitigate the negative effects of substructure movement on the structural members of a bridge can be costly and is not guaranteed to limit movement to a tolerable level. Without knowledge of the tolerance of a given bridge to substructure movement, proper designs cannot be created.

A study concluded in 1985 sought to determine the amount of substructure movement that certain bridges could withstand. The research included analyzing survey data from states and provinces throughout the United States of America and Canada. In the end, tolerable bridge movement limits were suggested based on the empirical data. Angular distortions (a measure of differential vertical settlement) were to be limited to 0.005 for simply supported bridges and 0.004 for continuously supported bridges. Horizontal movements were to be limited to 1.5 inches. Several computer models were also analyzed in an attempt to better understand tolerable bridge movements. The simple analytical models were used to investigate the behavior of bridges subjected to differential vertical movement and propose a method of designing more tolerable bridges (Moulton, GangaRao, & Halvorsen, 1985).

All of the bridge analyses were conducted using provisions provided in the American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications for Highway Bridges (AASHTO, 2002). Because the Standard Specifications were replaced by the Load and Resistance Factor Design (LRFD) Bridge Design Specifications (AASHTO, 2010), the previous research on tolerable bridge movements needs to be updated. Analyzing bridges in accordance with the LRFD Specifications will provide insight on the tolerances of newly designed bridges to differential vertical substructure movement. In addition to analyzing simplistic bridge models, typical in-service bridges also need to be examined in order to obtain more realistic and applicable results.

Rather than simply investigating the tolerable settlement limits for various bridges, methods of accounting for differential settlement stresses in bridge designs also need to be examined. The updated analytical data, along with the recommendations of past researchers, can be used to create design criteria that can easily be incorporated into the AASHTO LRFD Bridge Design Specifications. Recognizing the potential effects of differential settlement and accounting for expected stress increases during the design process will help guarantee that engineers produce safe, serviceable, and long-lasting bridge designs.

Chapter 2

BACKGROUND AND REVIEW OF RELEVANT LITERATURE

Determining whether or not a substructure movement is considered tolerable can be complicated. Intolerable movements affect the safety and function of a structure (Wahls, 1981). In general, movements are considered intolerable if the damage caused by the movement "requires costly maintenance or repairs and a more expensive construction to avoid this would be preferable" (Walkinshaw, 1978, p. 7). The economic aspect of the definition fueled much of the past tolerable movement research. Practical, cost-effective solutions are needed in order to determine the best method of calculating the amount of movement a structure can safely tolerate.

Much research has been done in the area of tolerable substructure movements. Initially, the research pertained solely to buildings. The concepts developed during the tolerable building movement research were later applied to bridges. Prior to the research, most structures were designed without consideration of the additional stresses caused by structure movement. Conservative approaches, such as the use of pile foundations, were sometimes taken in an attempt to limit movement (Moulton et al., 1985). Crude rules of thumb were often used to reduce the serviceability issues introduced by substructure movement (Burland & Wroth, 1974). Many early structures, however, were designed without considering substructure movement.

2.1: Components of Substructure Movement

When investigating bridge substructure movements, three different movements are of concern. Vertical movement, horizontal movement, and a combination of vertical and horizontal movements each have the potential to create structural and serviceability issues.

2.1.1: Vertical Movement

Vertical substructure movement, often referred to as settlement, is the most common and recognized type of movement. Vertical settlement is composed of three components: uniform settlement, tilt or rotation, and non-uniform settlement. No superstructure deformation occurs during uniform settlement; however, no settlement is ever completely uniform. Tilt or rotation involves a bridge settling linearly along the length of the bridge. This type of settlement is only truly possible for single span bridges. Finally, non-uniform, or differential, settlement involves deformation of the superstructure (Duncan & Tan, 1991).

2.1.1.1: Measuring Vertical Movement

Vertical settlements are quantified either by a measurement of the total settlement or differential settlement. Differential settlements are more damaging than uniform settlements. Total settlements, therefore, are often of less interest than differential settlements. Differential settlements can be measured in multiple ways, but the most commonly used measurement is angular distortion.

Skempton and MacDonald (1956) determined through their research on building settlements that the structural parameter that caused damage was actually the radius of curvature. Because the radius of curvature is somewhat difficult to calculate, however, angular distortion was determined to be the best characteristic for evaluating allowable settlements. Angular distortion (δ/ℓ) is defined as the differential settlement between two supports divided by the span length between the supports (Skempton & MacDonald, 1956).

The validity of using angular distortion as a method of relating differential settlement to the damage caused by settlement was questioned. Grant, Christian, and Vanmarcke (1974) addressed the concerns and concluded that the curvature parameter was no better suited for use as a damage limit than angular distortion. Additionally, angular distortion is much easier to calculate, which is preferable (Grant et al., 1974).

The use of angular distortion as the main method of measuring vertical differential settlements was extended to bridges with the research of Moulton et al. (1985). Use of angular distortion measurements is favorable with bridge research because angular distortion limits can be applied to bridges of any span length. General limits can, therefore, be applied to a large number of bridges.

2.1.2: Horizontal Movement

Horizontal substructure movement includes any lateral movement. In general, however, only lateral movement of bridge substructures parallel with the direction of the bridge are of interest. Because lateral movements are usually small in magnitude, any substructure movement perpendicular to the direction of the bridge will likely have an insignificant effect on the superstructure. Small lateral movements can, however, have considerable consequences when the movement occurs in the direction of the bridge.

2.1.3: Combined Movement

Realistically, no substructure movement is purely vertical or horizontal. Movement in one direction may be small and, therefore, neglected. Often, however, the movement of a substructure will have both vertical and horizontal components. In those cases, neglecting one of the movement components would cause an inaccurate assessment of the effects of movement on the structure.

2.2: Problems Caused by Substructure Movement

Every substructure movement has the potential to damage the structure or create a variety of other serviceability issues. The problems that can arise are dependent on the type of movement that is experienced by the structure. Research on the movement of buildings has mainly dealt with vertical settlement and the superficial cracking it can cause. Structural damage is also considered, but superficial cracking is the main issue related to building settlements (Skempton & MacDonald, 1956). Additionally, horizontal movements are hardly discussed. For those reasons, the issues that affect bridges needed to be investigated separately. Any potential structural damage or functional distress caused by bridge substructure movements must be identified in order to create tolerable movement limits that will properly protect a bridge superstructure.

2.2.1: Vertical Movement

Vertical settlement can cause a variety of issues in bridges. Uniform settlement can reduce clearance below the bridge, cause drainage problems, create "the bump at the end of the bridge," and have harmful effects on any connected utilities, but do not generally cause structural distress. Differential settlement, however, can induce structural distress in addition to the issues associated with uniform settlement. Structural distress caused by differential settlement may include increased internal stresses, which can reduce the load carrying capacity of the bridge, and increased support reactions. If the structural distress becomes very large, cracking may occur in the bridge deck or concrete girders. Riding quality over the length of the bridge can also be impacted by differential settlement. As the magnitude of the settlement increases, the effect of the settlement on a bridge also increases (Duncan & Tan, 1991, Samtani & Nowatzki, 2006).

2.2.2: Horizontal Movement

The damaging effects of horizontal movements have been recognized since the first research in bridge substructure movement. Though horizontal substructure movements are generally smaller in magnitude than vertical movements, they can be very damaging. Horizontal movements have been found to cause more severe problems than vertical movements of equal magnitude. Issues caused by horizontal movements include shearing of anchor bolts; excessive opening or complete closing of expansion joints; decks and girders jamming into abutments or adjacent spans; shifting of abutments; damage to abutment walls, approach slabs, or decks; distortion or damage to bearings; tilting of rockers; and damage to railings, curbs, sidewalks, parapets, and attached utilities (Samtani & Nowatzki, 2006).

2.2.3: Combined Movement

When both vertical and horizontal substructure movements occur, any of the issues previously discussed could affect the bridge. Additionally, problems are more likely to occur when combined movements occur. Research has shown that when combined vertical and horizontal movements occur, the effects are more detrimental than the effects of a unidirectional movement of the same magnitude (Walkinshaw, 1978).

2.3: Methods of Determining Tolerable Substructure Movements

Bjerrum (1963) and Wahls note that a design engineer is responsible for two general tasks when designing a foundation. First, the movement that can be expected due to conditions at the construction site must be estimated. Then, the tolerance of the structure to the predicted movement should be determined. Foundation movements can be approximated using various geotechnical methods. Calculating the allowable movement of a structure, however, can be a complex, indeterminate problem (Wahls, 1981). Several research projects have attempted to create general movement limits in an effort to eliminate the complexity of determining allowable movements.

2.3.1: Tolerable Building Movements

Early research in tolerable building foundation movements included analysis of collected data and simplistic building models. Accurate analytical models, however, were found to be too complex. Therefore, the main focus of the tolerable building movement studies was empirical analyses. The empirical research of tolerable building foundation movements produced ideas that would later be incorporated into the research of tolerable bridge substructure movements.

2.3.1.1: Empirical Methods

The first extensive research into tolerable structure movement was performed by Skempton and MacDonald. They stated that, in order to obtain a rational foundation design, an understanding of allowable structural settlements was necessary. Any settlement data obtained by the engineer was of no use if the amount of settlement the structure could tolerate was not known. Calculating the allowable settlement of a building analytically was found to be very difficult. Measured stresses were found to be only 50 to 75 percent of those calculated. In light of this difficulty, Skempton and MacDonald collected foundation movement records and utilized empirical analyses in their work to create building settlement limits (Skempton & MacDonald, 1956).

Bjerrum supported the empirical analyses used by Skempton and MacDonald. He noted that allowable settlements could not be theoretically calculated because assuming static behavior does not account for many actual behaviors, which are inelastic. Interaction between structural and secondary elements, time factors, and load redistribution all influence the amount of settlement that can be tolerated by a structure, but are not accounted for in typical theoretical calculations. Because theoretical calculations do not represent realistic building behavior, Bjerrum concluded that observations and previous experience must be used to estimate the allowable settlements of buildings (Bjerrum, 1963).

While the research by Skempton and MacDonald was generally accepted to be the basis by which building settlements were judged, the empirical method used by Skempton and MacDonald was questioned by some. Golder (1971) suggested that more observations, along with careful building classification, were needed in order to produce more agreeable limits. Research by Grant et al. (1974) addressed the criticisms by updating, extending, and critically examining the research performed by Skempton and MacDonald. The methods used by Skempton and MacDonald were found to be valid and the results that were produced were confirmed by additional data (Grant et al., 1974).

Wahls stated that the available studies on allowable settlement are biased toward situations where damage occurred. Only records of buildings experiencing damage were used in the previous building settlement research. Having said that, Wahls confirmed the work done by Skempton and MacDonald. Additionally, Wahls reiterated the concern of Golder regarding building classification. The need for different settlement limits specific to different types of building structures was emphasized (Wahls, 1981).

In 1965, Feld called attention to the issue of horizontal movements. Until this point, little had been said about horizontal movements and how they affect structures. Feld stated that since movements are generally not solely vertical or horizontal, the tolerance of a structure to all movements should be investigated (Feld, 1965). No horizontal movement limits or methods for determining such limits were discussed. Based on the methods used in previous research, however, empirical analyses would have been the preferred method of determining allowable horizontal foundation movement.

2.3.1.2: Analytical Methods

Despite the difficulties recognized by Skempton and MacDonald and Bjerrum, Golder insisted that computer models be used in an attempt to verify observational results. Golder viewed the verification of the empirical limits a necessary step in the creation of tolerable substructure movement limits. The complicated models required for verification, however, were not created as part of his research.

Burland and Wroth deemed it necessary to have a clear understanding of how movement and damage are related. As part of their research, they created some simplistic models to represent the effects of movement on a building. The analyses included modeling a building as a beam in an effort to relate settlements to the damage they cause. Some cracking issues were able to be investigated, but, overall, these simplistic models were unable to represent the behavior of entire buildings (Burland & Wroth, 1974).

Burland and Wroth reiterated the idea that modeling the response of a structure to movement is very complex. Factors that must be taken into consideration include non-linear interactions, immediate versus long-term settlement, changes in structure stiffness throughout construction, and load redistribution. A large, complicated finite element model would be needed in order to analyze the true effects of foundation movements on a structure (Burland & Wroth, 1974).

2.3.2: Tolerable Bridge Movements

Research investigating tolerable movements of bridges was initiated in a manner similar to the building movement research. The initial approach to investigating tolerable bridge movements was empirical. Early empirical analyses allowed for crude movement limits to be created and general conclusions to be drawn. The general limits and conclusions then facilitated further research.

Analytical studies were also used to investigate tolerable bridge movement criteria despite the difficulties previously encountered by Skempton and MacDonald, Bjerrum, and Burland and Wroth. While many of the same issues that complicated the formulation of accurate building models also apply to bridges, the differences between the structures allows for bridge models to be created more easily. Although not completely accurate, the analysis of bridge models provided a good supplement to the empirically based movement limits.

2.3.2.1: Empirical Methods

The first major effort to gain knowledge of the effects of substructure movement on bridges was initiated by the Transportation Research Board (TRB). In 1975, TRB Committee A2K03 sent out surveys to states and provinces throughout the United States of America and Canada. The 1975 survey was actually a second survey, following a smaller survey conducted by TRB Committee SGF-B3 in 1968 (Moulton et al., 1985). The surveys gathered information regarding horizontal and vertical movements of piers and abutments, structure types, construction sequences, effects of movements on structural elements, and whether the observed movements were considered to be tolerable (Walkinshaw, 1978). In total, 35 states and provinces responded with information. The survey data was utilized in four different research projects and the results were published in 1978 (Keene, 1978).

Upon reviewing the survey data, Keene found that movements were often considered harmful, yet tolerable. When considering the economic aspect of limiting substructure movement, some damage may be considered an acceptable alternative to spending additional money to prevent the damage. Keene stated that the major factors to consider when determining whether movements are tolerable are the amount of movement, type of structure, member effects, cost of alternative choices, effect on travelers, subjective reasons, and apprehension during the design process. Considering all of the factors and the economic feasibility of the design could lead to a design that allows for harmful, yet tolerable, movement (Keene, 1978).

Walkinshaw used the previously established definition of intolerable movements to categorize bridge movements into three categories: 1) tolerable movements, 2) intolerable movements with respect to riding quality only, and 3) intolerable movements that result in structural damage (Walkinshaw, 1978). The categorization of the survey data allowed for investigation of how different magnitudes of settlement affect bridges.

In addition to the results of the TRB surveys, Grover (1978) utilized 1961 survey data of highway bridges in Ohio (Grover, 1978). An empirical analysis of all of the Ohio bridge data obtained from all of the surveys was performed. Based on the results of the survey, Grover was able to suggest some tolerable bridge movement limits. Both vertical and horizontal limits were discussed.

Bozozuk (1978) found the performance of the bridges investigated related directly to the type of movement experienced. Various large movements were determined to be tolerable because the movements were uniform. On the other hand, some small differential and rotational movements were found to be intolerable. Horizontal movements were found to have greater effects than equivalent vertical movements (Bozozuk, 1978). The paper by Bozozuk was the first to touch on the issue of uniform versus differential settlement for bridges. Also, he was the first to recognize the criticality of horizontal substructure movements.

Stermac (1978) questioned the methods used by Bozozuk to analyze the survey data. He stated that only the designer can safely estimate the tolerable movements of a given bridge. The type and amount of movement a bridge can withstand will vary by bridge type, length, span length, and width (Stermac, 1978). The arguments by Stermac were valid and were addressed, to some extent, through the use of angular distortion as a measure of differential settlement. Additionally, future researchers would note that general limits may not apply to all bridges and engineering judgment must always be used.

An in-depth Federal Highway Administration (FHWA) study started in 1978 and completed in 1985 by Moulton, et al. investigated tolerable bridge foundation movements further. Tolerable bridge movement criteria were created based on survey results and supported by analytical studies. Data from a total of 314 bridges throughout the United States of America and Canada, including the data from the previous TRB surveys, was collected and analyzed (Moulton et al., 1985).

Moulton et al. analyzed the survey data in three different ways. First, the influence of substructure variables on foundation movements was investigated. Next, the influence of the foundation movements on the bridge structure was examined. Finally, the tolerance of bridges to various movements was studied (Moulton et al., 1985). Using the three different analyses, Moulton et al. were able to create some general movement limits.

Yokel (1990) claimed that Moulton et al. did not obtain and analyze data samples representative of the entire bridge population. The data gathered by Moulton et al. was only for bridges that had experienced foundation movement. While the data did represent a fairly large number of bridges, Yokel argued that the records were not selected at random (Yokel, 1990). While this may be true, the limits created by Moulton et al. cannot be completely disregarded. The tolerable substructure movement limits can simply be considered somewhat conservative. Similar arguments were made toward the tolerable limits created by Skempton and MacDonald; however, the limits were later confirmed. Additionally, the work by Moulton et al. is widely recognized as a good, comprehensive study on tolerable bridge movements.

2.3.2.2: Analytical Methods

In addition to the survey results, Moulton et al. performed analytical studies in order to produce additional data to assist in investigation of tolerable bridge movements. These studies involved the use of various line-girder analyses. Both steel and concrete girders were investigated. Two-span continuous and four-span continuous bridges with a variety of span lengths were modeled. The analytical studies were theoretically based and, therefore, assumed complete elastic behavior (Moulton et al., 1985).

The bridge models were designed using the 12th Edition of the AASHTO Standard Specification for Highway Bridges. The analyses utilized the AASHTO HS20-44 wheel loading or the equivalent lane loading in addition to the dead load. Differential vertical settlements between 0 and 3 inches were applied to the bridges. Settlement of each support was investigated separately in order to maximize the effect of the vertical movement (Moulton et al, 1985).

Steel girder bridges with spans of 30 to 60 feet were designed with rolled beams. Rolled beams with cover plates were used for girder bridges with span lengths of 100 and 150 feet. Plate girders were used in bridges with span lengths of 200 and 250 feet. An 8-inch composite concrete deck was used in all cases (Moulton et al., 1985).

Analyzing concrete bridges was found to be more complex than analyzing steel bridges. Material properties, structural configuration, construction sequence, and general assumptions complicated the process. Concrete creep can help to relieve some of the additional stress introduced by differential settlement and, therefore, creates difficulty when studying stress increases. Additionally, different section choices and the construction timeline can significantly affect the behavior of a girder (Moulton et al., 1985).

Because of these complicated issues, a sophisticated model is necessary when analyzing concrete bridges. The models need to account for the various time-dependent issues involved in the analysis (such as creep, shrinkage, and strand relaxation). Large, complex models, however, lead to time-consuming computations. Less sophisticated models, which are more approximate in nature, are also available. Moulton et al. utilized both the relaxation method and the step-by-step method to calculate the time-dependent properties of prestressed concrete (Moulton, GangaRao, & Halvorsen, 1982).

After selecting an appropriate modeling method, Moulton et al. analyzed two-span continuous concrete I-girders with non-composite decks, girders made continuous with a field joint and non-composite decks, and girders with composite decks. Concrete bridges with spans of 75, 100, and 125 feet designed with AASHTO type II, IV, and VI girders, respectively, were analyzed (Moulton et al., 1985).

Moulton et al. showed that tolerable bridge movements can be investigated using analytical models. When reviewing the research of Moulton et al., Duncan and Tan reiterated that traditional structural analysis methods produce overly conservative results when investigating substructure movements. Simple models and traditional analysis methods do not account for many complex issues, which were first recognized by Skempton and MacDonald, Bjerrum, and Burland and Wroth.

Yokel addressed the overly conservative results of traditional structural analyses. Steel beams that meet compactness requirements are known to have rotational capacities larger than calculated using elastic analyses. If the inelastic rotational capacities of steel beams are recognized, a more accurate bridge system analysis is possible (Yokel, 1990).

Hearn and Nordheim (1998) expanded on the idea of inelastic rotation in compact steel girder bridges. The researchers attempted to produce a model capable of analyzing the response of steel beams to differential settlements. As has been stated previously, structural analysis tends to be overly conservative. Traditional structural analysis assumes elastic behavior. Hearn and Nordheim proposed a model for the tolerable inelastic rotation capacity of steel beams. Use of the inelastic model produced results comparable to the empirical limits that had been previously established (Hearn & Nordheim, 1998).

Inelastic analyses were performed in accordance with the 1994 AASHTO LRFD Bridge Design Specifications (and National Cooperative Highway Research Program Report 352). Braced, compact steel beams were utilized. Inelastic design allows the negative moments at the piers to be reduced and redistributed to the positive moment regions in the span. Theoretically, the moments caused by differential settlement could be completely relieved by moment redistribution (Hearn & Nordheim, 1998).

Moment redistribution is not applicable in all situations, however. Only steel sections meeting the compactness criteria are eligible for inelastic analysis. Inelastic analysis of plate girders is only allowed in a limited number of cases based on the proportions of the girder. Also, moment redistribution can only be applied to sections experiencing negative moments. Positive moments are not allowed to be redistributed (Yokel, 1990).

Yokel also found that, if detailed properly, concrete beams can also have enough rotational capacity to counteract the effects of differential settlement. AASHTO does not allow moment redistribution in concrete bridges. The American Concrete Institute (ACI), however, does allow some moment redistribution under certain circumstances. Yokel suggested that rotations caused by differential settlements can be accommodated by concrete bridges without any loss in strength as long as the angular distortion is limited (Yokel, 1990).

2.4: Results of Tolerable Substructure Movement Analyses

Based on the substructure movement studies performed, tolerable limits were proposed. While these limits are mainly based on actual data, they should not be taken as strict rules. Instead, the proposed tolerable movement limits should be used as general guidelines to aid in the design process. Every structure is different and good engineering judgment should be used in determining the tolerances of individual structures to substructure movements (Skempton & MacDonald, 1956, Duncan & Tan, 1991).

2.4.1: Vertical Movement Limits

Much of the focus on substructure movement during the design process is placed on vertical settlements. As previously stated, settlements can negatively impact the performance of a structure. Differential settlements were found to be the most damaging and are, therefore, the focus of the most widely accepted tolerable movement limits. While early tolerable settlement limits were found primarily for buildings, they can be used as a means of comparison to the tolerable bridge movement limits.

2.4.1.1: Buildings

Skempton and MacDonald concluded that, for buildings, an angular distortion greater than 1/300 (0.0033) would cause panel cracking to begin and an angular distortion greater than 1/150 (0.0067) would cause structural damage. In cases where settlements occur very slowly, larger settlements may be tolerable (Skempton & MacDonald, 1956).

A similar study by Polshin and Tokar (1957) generally agreed with the limits determined by Skempton and MacDonald. A number of angular distortion limits for specific types of buildings and components were presented. The limits found by Polshin and Tokar were based on the Union of Socialist Soviet Republics (USSR) building code requirements and, therefore, differed slightly from those presented by Skempton and MacDonald. The angular distortion limits proposed by Polshin and Tokar ranged from 0.002 to 0.007, depending on the type of building (Polshin & Tokar, 1957).

After updating the work by Skempton and MacDonald, Grant et al. confirmed the original tolerable settlement limits. Angular distortions greater than 1/300 (0.0033) were likely to cause damage. When settlements were very slow, however, the maximum allowable angular distortion tended to be between 1/250 (0.004) and 1/200 (0.005). Grant et al. emphasized that the rate of settlement should only be considered in extreme cases – very slow or very fast settlement (Grant et al, 1974).

2.4.1.2: Bridges

Upon categorizing the TRB survey data, Walkinshaw found that tolerable vertical movements of 0.5 to 18 inches were reported. Vertical movements that were considered intolerable due to rider discomfort ranged from 2.5 to 12 inches. Finally, vertical movements that were structurally intolerable varied from 0.5 to 24 inches.

Based on the vertical movement ranges, Walkinshaw concluded that large vertical movements were often found to be tolerable from a structural standpoint, but any vertical movement larger than 2.5 inches was found to cause rider discomfort. Therefore, Walkinshaw suggested that the vertical movement limit for bridges be set at 2.5 inches (Walkinshaw, 1978). Moulton et al. later contradicted Walkinshaw by concluding that settlements would become intolerable for some other reason before reaching a magnitude that would cause unacceptable rider discomfort (Moulton et al., 1985).

After analysis of all of the data collected from the state of Ohio, Grover concluded that settlements of less than 1 inch should be classified as tolerable and would rarely be noticed by travelers, settlements of 2 to 3 inches would cause minor damage at most, but would be noticeable to travelers, and settlements of more than 4 inches would likely cause physical damage to the bridge and would be very noticeable to travelers (Grover, 1978).

Bozozuk concluded that tolerable vertical settlements were less than approximately 2 inches. Tolerable, yet harmful vertical movements were between 2 and 4 inches. Finally, intolerable vertical movements were found to be greater than 4 inches (Bozozuk, 1978).

Analysis of the survey data collected by Moulton et al. showed that nearly 98 percent of differential settlements less than 2 inches were considered tolerable. Approximately 91 percent of differential settlements less than 4 inches were considered tolerable. Only 24 percent of differential settlements between 4 and 8 inches and only

18 percent of differential settlements over 8 inches were considered tolerable (Moulton et al, 1985).

Moulton et al. then considered the issue first recognized by Stermac. The survey data was modified to account for varying span lengths. The differential settlements were divided by the associated span lengths in order to obtain angular distortion values. Analysis of the corrected data produced tolerable angular distortion values of 0.005 for simply supported bridges and 0.004 for continuously supported bridges (Moulton et al., 1985). The angular distortion limits produced by Moulton et al. for bridges are larger than the limits created by Skempton and MacDonald for buildings, which should be expected. Buildings are stiffer, more rigid structures and are, therefore, more affected by differential settlements.

Based on his research, Yokel proposed more restrictive vertical movement limits. He stated that total settlements of 1 inch or 1/600th (0.0017) the span length, whichever is smaller, should be unconditionally allowed. Settlements of 2 inches or 1/300th (0.0033) the span length, whichever is smaller, should be allowed only if the bridge is designed to accommodate such movements (Yokel, 1990).

Upon examination of the results and limits provided by Moulton et al., Duncan and Tan found it necessary to make an alteration. The authors argued that one of the simple span bridges included in the Moulton et al. analysis was a clear aberration and should not have been included. By removing the aberrant bridge and re-analyzing the data, Duncan and Tan found that the allowable angular distortion for simply supported bridges should be 0.008, not 0.005 as suggested by Moulton et al. Duncan and Tan suggested that the tolerable limit for the angular distortion of a continuously supported bridge remain at 0.004 (Duncan & Tan, 1991).

The inelastic modeling performed by Hearn and Nordheim did not produce specific angular distortion limits. Instead, a method for finding the inelastic rotation capacity of compact steel beams based on flange local buckling is proposed. Rotational capacities found using the proposed inelastic method were found to be consistent with the angular distortion limits previously created from empirical data by Moulton et al. (Hearn & Nordheim, 1998).

2.4.2: Horizontal Movement Limits

Past research has repeatedly determined that horizontal substructure movements are more critical than vertical movements. Small horizontal movements can cause great amounts of damage and distress to bridge structures. Tolerable horizontal substructure movements have been researched strictly on an empirical basis.

2.4.2.1: Buildings

Initially, research in the area of horizontal building movement was lacking. Feld first recognized the importance of investigating lateral movements, but did not create any tolerable limits. No tolerable horizontal movement limits were created for buildings prior to research of tolerable bridge movements.

2.4.2.2: Bridges

Walkinshaw categorized the original TRB survey data in order to perform meaningful analyses. Horizontal movements up to 2 inches were found to be tolerable. Horizontal movements were found not to cause rider discomfort, thus no movements were considered to be intolerable due to rider discomfort. Finally, movements between 1 and 8 inches were found to be intolerable due to the structural damage caused. Walkinshaw concluded that the tolerable limit for horizontal bridge movement should be 2 inches (Walkinshaw, 1978).

Bozozuk concluded that tolerable horizontal movements were less than 1 inch. Tolerable, yet harmful horizontal movements were between 1 and 2 inches. Finally, intolerable horizontal movements were found to be greater than 2 inches (Bozozuk, 1978).

The data collected by Moulton, et al. showed that horizontal substructure movements of 1 inch or less were nearly always considered tolerable. Horizontal movements less than 2 inches were found to be tolerable 89 percent of the time. Conversely, 82 percent of horizontal movements of 2 inches or more were considered to be intolerable. Mouton et al. suggested that horizontal abutment movements should be less than 1.5 inches. This value is the average between a value that was often considered acceptable (2 inches) and a value that was nearly always acceptable (1 inch) (Moulton et al, 1985).

Yokel again suggested stricter limits. Horizontal foundation movements up to 3/8 (0.375) inch should be considered unconditionally acceptable. If a bridge is designed to accommodate the additional movement, horizontal movements up to 3/4 (0.75) inch can be allowed (Yokel, 1990).

2.4.3: Combined Movement Limits

Moulton et al. were the only researchers to investigate the effects of both vertical and horizontal substructure movement. They found that only 60 percent of horizontal movements less than 2 inches were considered tolerable when paired with any amount of differential vertical settlement. When accompanied with differential settlement, only horizontal movements of 1 inch or less were found to be acceptable (Moulton et al., 1985).

2.4.4: General Trends Observed in Bridges

The analytical studies performed by Moulton et al. were not used to produce tolerable movement limits directly. Instead, the results of the analyses were used to assemble tables and graphs and draw general conclusions regarding differential vertical settlements. The effects of settlement magnitude, span length, section choice, number of spans, cross-sectional layout, construction material, time of settlement, and bridge type were investigated.

2.4.4.1: Critical Settlement Conditions

Examination of the resulting data suggested that two settlement conditions were critical. For two-span continuous bridges, settlement of an exterior support caused the maximum negative stress over the center support. The maximum positive stress occurred near the center of the first span when the center support settled. For four-span continuous bridges, settlement of the first interior support caused the maximum negative stress over the center support. The maximum positive stress occurred near the center of the second span when the center support settled (Moulton, et al., 1982).

2.4.4.2: Span Length

As would be expected, when the magnitude of differential settlement increases, the stresses induced by that settlement also increase. As span length decreases, the stress increases caused by differential settlement increase rapidly. Therefore, shorter bridges are more affected by differential settlements than longer bridges. The effects of increasing settlement and differing span lengths are shown in Figure 2.4.1, which displays the theoretical percent increases in negative stress at the center support of multiple two-span continuous W36 rolled steel girder bridges due to settlement of an exterior support. Figure 2.4.2 shows the theoretical percent increase in positive stress at the middle of the first span of the same two-span continuous W36 rolled steel girder bridges due to settlement of the center support (Moulton et al., 1982).

2.4.4.3: Stiffness

Stiffness was found to be another important factor in the response of a bridge superstructure to settlement of the substructure. Three different wide-flange sections were analyzed in multiple bridges. Figure 2.4.3 shows the theoretical negative stress

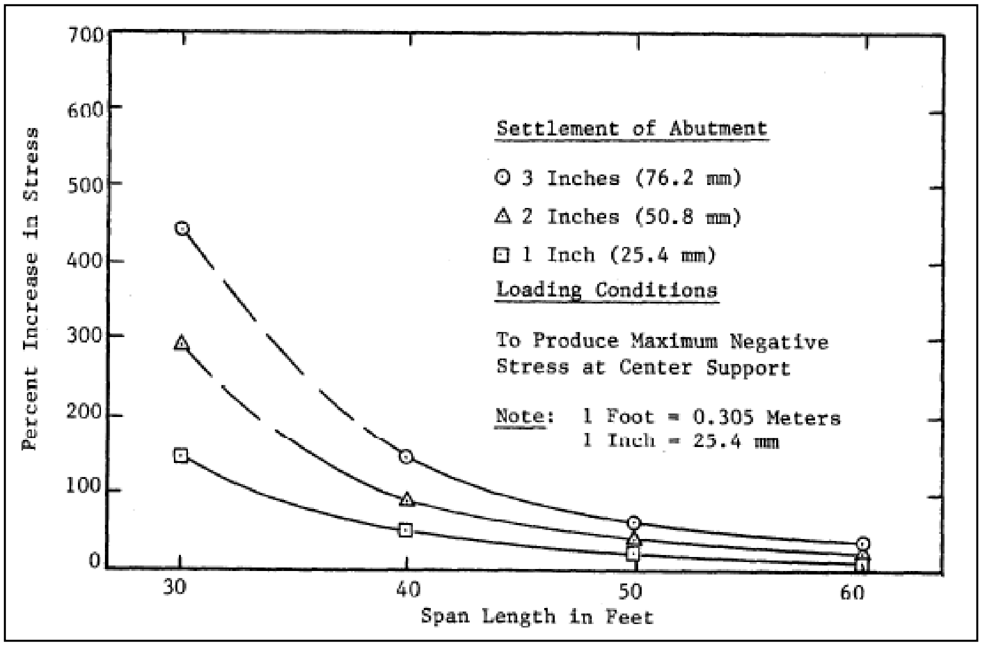


Figure 2.4.1: Increases in Negative Stress in Two-Span Continuous Steel Bridges (Moulton et al., 1982)

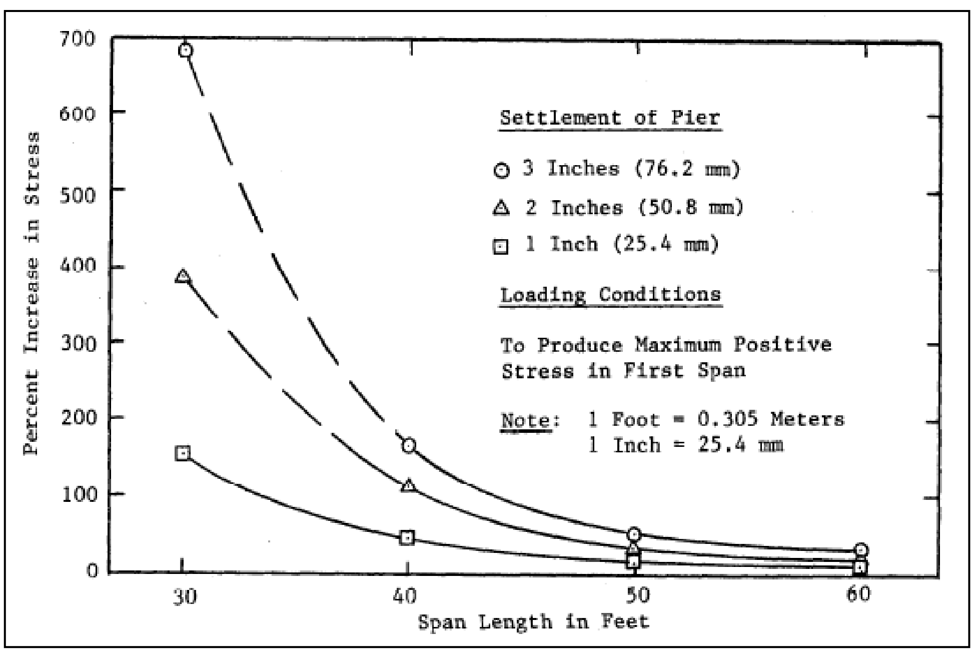


Figure 2.4.2: Increases in Positive Stress in Two-Span Continuous Steel Bridges (Moulton et al., 1982)

increases for bridges designed with W30, W33, and W36 rolled steel girders subjected to a 3-inch exterior support settlement. Theoretical increases in the positive stress in the same bridges due to a 3-inch center support settlement are displayed in Figure 2.4.4. As the figures clearly show, the stiffer (W36) girders are more affected by differential settlement than the more flexible girders (Moulton et al., 1982). The deeper sections are stiffer and, therefore, experience larger stress increases due to settlement.

Another method of investigating the stiffness-to-stress increase relationship is by calculating stiffness values. The stiffness for a given section is defined as the moment of inertia (I) of the section divided by the span length (ℓ) of the member.

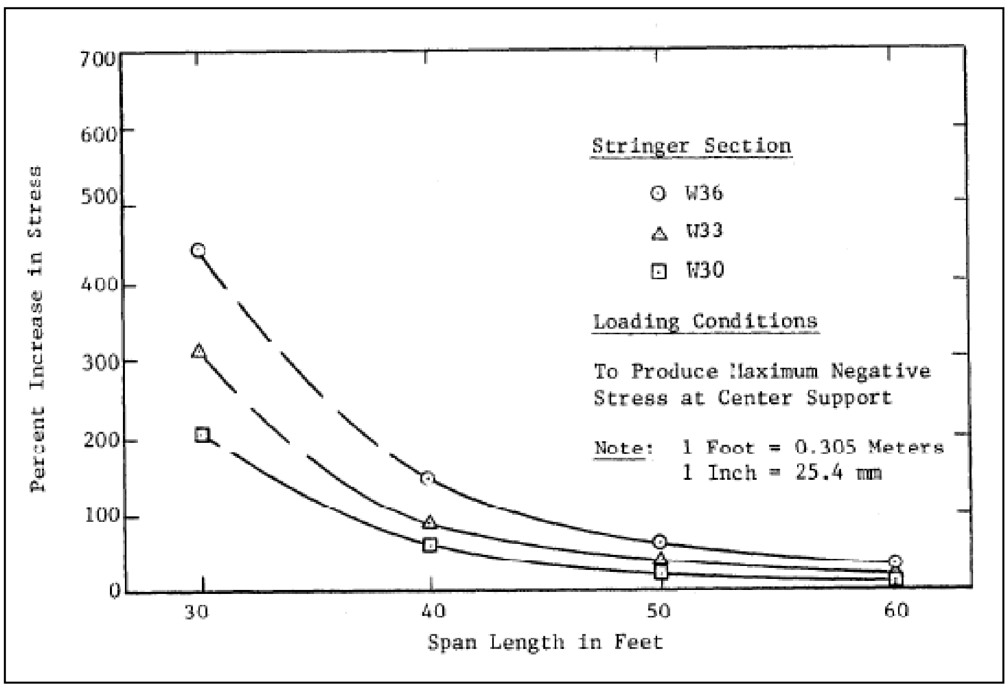


Figure 2.4.3: Negative Stress Increases for W30, W33, and W36 Steel Girders (Moulton et al., 1982)

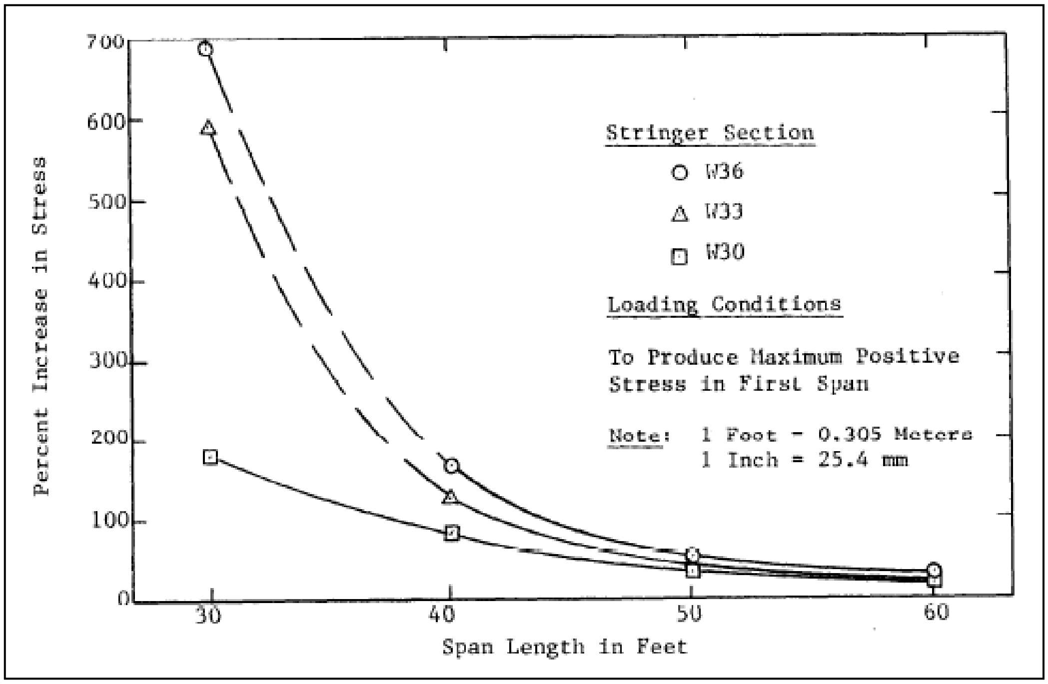


Figure 2.4.4: Positive Stress Increases for W30, W33, and W36 Steel Girders (Moulton et al., 1982)

Figures 2.4.5 and 2.4.6 show the increases in negative and positive stresses, respectively, for varying stiffness values. Again, the figures show that increasing stiffness will cause larger stress increases for the same magnitude of differential settlement. Based on the results of the stiffness analyses, Moulton et al. concluded that members with stiffness values less than 20 cubic inches were generally able to tolerate the applied differential settlements (Moulton, et al., 1985).

2.4.4.4: Girder Spacing

The effect of altering cross-sectional layouts was also investigated as part of the analytical studies. The number of girders and the girder spacing were varied. Although

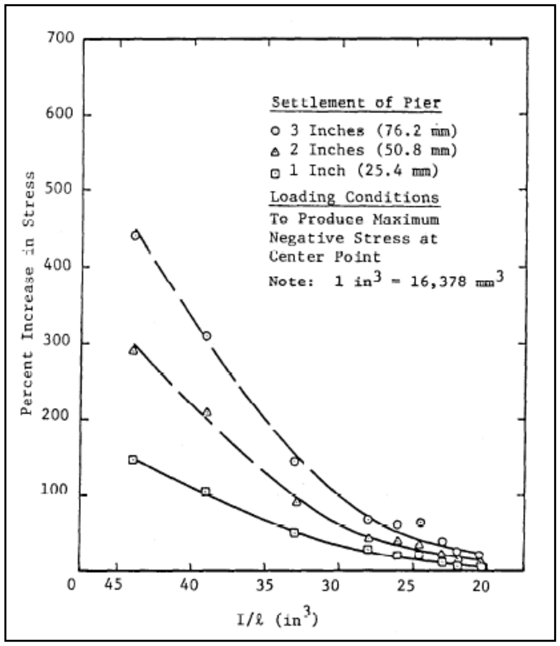


Figure 2.4.5: Affect of Varying Stiffness on Negative Stress Increases (Moulton et al., 1985)

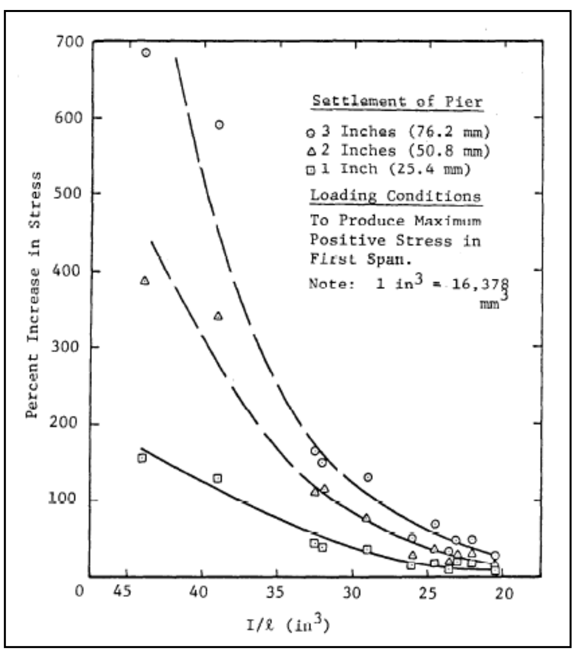


Figure 2.4.6: Affect of Varying Stiffness on Positive Stress Increases (Moulton et al., 1985)

adding or removing girders changed the moments carried by the bridge girders, the response of the girders to differential settlement was not significantly impacted. The effect of altering girder spacing was determined to be negligible when investigating vertical substructure movement (Moulton et al., 1982).

2.4.4.5: Construction Material

Though steel girder bridges were used to show the previous trends, similar results would be expected for concrete bridges since material properties were not the focus of the analyses. Material properties, however, do affect the response of a structure to differential settlement. Concrete creep can help to relieve some of the additional stress introduced by differential settlement. Creep is one factor that makes concrete bridges slightly more tolerant to differential settlement than steel bridges (Moulton et al., 1982).

Concrete bridges are also more tolerant to differential settlements because concrete is less rigid (defined as the product of the modulus of elasticity (E) and moment of inertia (I) of a section) than steel. More rigid structures were found to have smaller allowable angular distortions, which suggests that more rigid structures are more susceptible to damage from differential settlements (Skempton & MacDonald, 1956).

2.4.4.6: Time of Settlement

Typical structural analyses assume that any applied settlement is immediate. Realistically, that is not always the case. Any settlement that occurs within a small amount of time after construction can be considered immediate. Settlements that occur over greater amounts of time, however, are not immediate. The amount of time over which the differential settlement occurs was discussed by various researchers.

Structures subjected to slower settlements were generally able to withstand larger amounts of distortion without damage. The longer the period of time over which the settlement occurs, the more time the structure has to react and relieve some of the additional stresses (Bjerrum, 1963). These findings were confirmed by the angular distortion limits proposed for buildings by Grant et al.

Sudden settlements can cause concrete cracking, especially in the bridge deck above an interior support. The sudden increase in negative moment causes tension in the concrete, which, if high enough in magnitude, will cause cracking. Despite the potential negative effects of immediate settlement, the stress increases in concrete girders due to the sudden settlement was found to be reduced by creep (Moulton et al, 1985).

Moulton et al. found gradual settlement in concrete bridges to have little effect on girder stresses initially. Over time, however, stresses due to gradual settlement exceeded those due to sudden settlement. At first, moment redistribution caused by creep would offset the settlement-induced stresses, but, over time, creep relief would no longer be present to offset the additional stresses (Moulton et al., 1985).

2.4.4.7: Bridge Type

Various types of continuous bridges were investigates as part of the study by Moulton et al. The majority of the analytical studies were performed using steel bridges due to the complexity of analyzing concrete members. Despite the difficulty presented by prestressed concrete, some concrete bridges were also utilized in the studies. Simply supported bridges were not included in the analytical studies because differential settlements should not create additional stresses in simply supported bridge girders.

2.4.4.7.1: Steel Bridges

Two- and four-span continuous steel bridges with spans up to 60 feet in length were designed using rolled beams. Rolled beams with cover plates were used in bridges with spans up to 150 feet in length. For spans up to 250 feet in length, plate girders were designed. Two-span continuous parallel and non-parallel chord trusses were also designed for spans up to 680 and 880 feet, respectively (Moulton et al., 1982).

Four-span continuous bridges were found to be more affected by differential settlement than two-span bridges. The number of continuous bridge spans affects the stiffness of a structure. Four-span continuous bridges are stiffer than two-span continuous bridges and, therefore, are more impacted by differential settlements (Moulton et al., 1982).

Based on the results of the span length analyses, Moulton et al. concluded that bridges with spans of 100 feet or more were not greatly affected by differential settlements up to 3 inches (Moulton et al., 1982). That implies that bridge types used to span 100 feet or more should be able to tolerate some differential settlement. As long as the foundation design is adequate and settlements are limited to approximately 3 inches, longer span bridges should not experience settlement damage.

Properly designed medium- to long-span bridges should not be significantly impacted by differential settlements. Since many plate girder bridges fall into those categories, they should be able with withstand up to 3 inches of differential settlement. Plate girders are also used in short-span bridges, which are more susceptible to settlement damage. Trusses are generally only used for long-span bridges and are, therefore, not susceptible to settlement damage. Moulton et al. found that the affects of 3 inches of settlement to be negligible for all of the parallel and non-parallel chord trusses analyzed (Moulton et al., 1982).

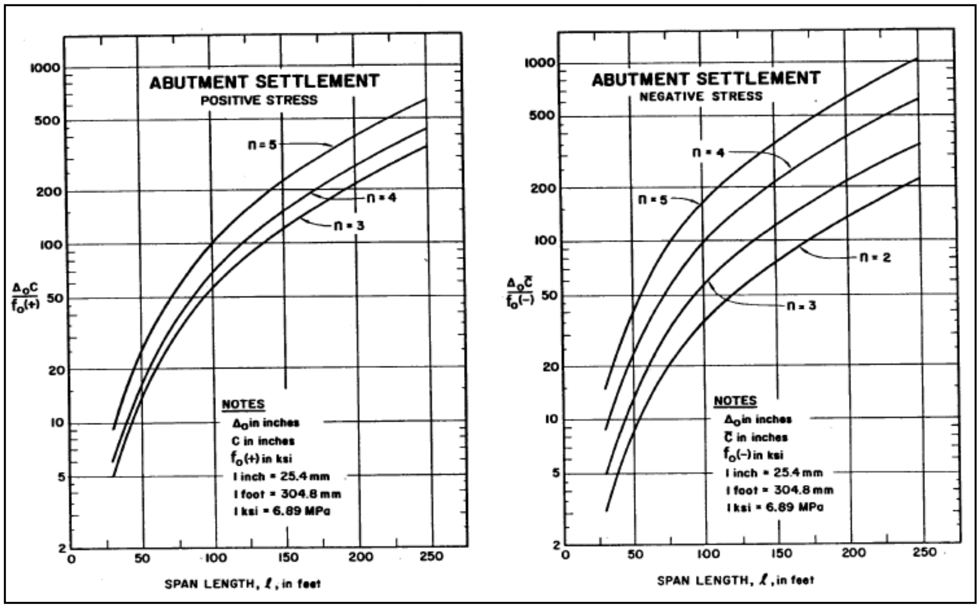
2.4.4.7.2: Concrete Bridges

Two-span continuous I-girder bridges with span lengths of 75, 100, and 125 feet were analyzed. Differential settlement stresses were found to be fairly significant for all three span lengths. As span length is increased, the effects of settlement would be expected to decrease. Because the cross-sectional properties of a prestressed concrete beam vary along with span length, however, the effects of settlement also vary. The effects of settlement depend on both span length and the cross-sectional shape of the concrete girder (Moulton et al., 1982). Because the selection of available concrete girder shapes is much smaller than the selection of steel girder shapes, concrete girders often must be designed with a less than desirable amount of flexibility.

Concrete box girders were also examined as part of the study. Both two- and four-span continuous box girder bridges were analyzed. The four-span girders were found to be more affected by settlement due to the added stiffness created by the additional spans. Overall, settlement stresses in 200-foot span bridges were found to be negligible. For 100-foot spans, however, significant stress increases are possible. The center supports can be expected to experience a sudden stress reversal due to the settlement. Tension cracking should be expected at mid-span and over the center support (Moulton et al., 1982).

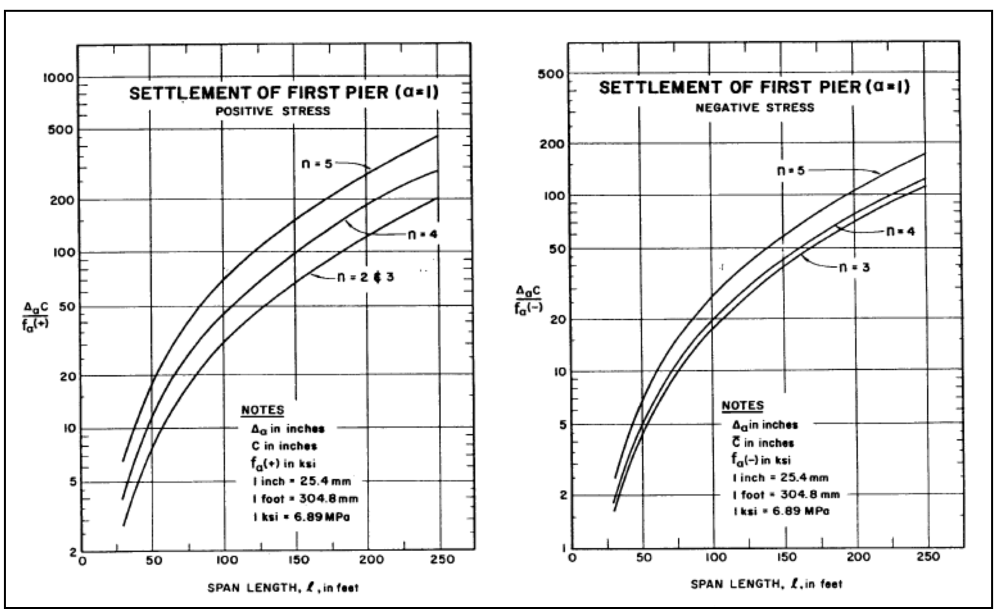
2.4.5: Design Aids

In addition to using the analytical results to observe different trends, Moulton et al. used the information to create a mathematical model, which was then used to create design aids. The design aids were intended to provide engineers with a simple method of estimating settlement-induced stresses based on the calculated settlements. The design aids were created for use with continuous steel girder bridges only (Moulton et al., 1985). Figures 2.4.7, 2.4.8, and 2.4.9 show the six design aids that were created. Positive and negative stress increases due to abutment settlement can be estimated from Figures 2.4.7(a) and 2.4.7(b), respectively. Figures 2.4.8(a) and 2.4.8(b) can be used to estimate positive and negative stress increases due to settlement of the first interior pier. Finally, the positive and negative stress increases due to settlement of the second interior pier can be estimated using Figures 2.4.9(a) and 2.4.9(b).



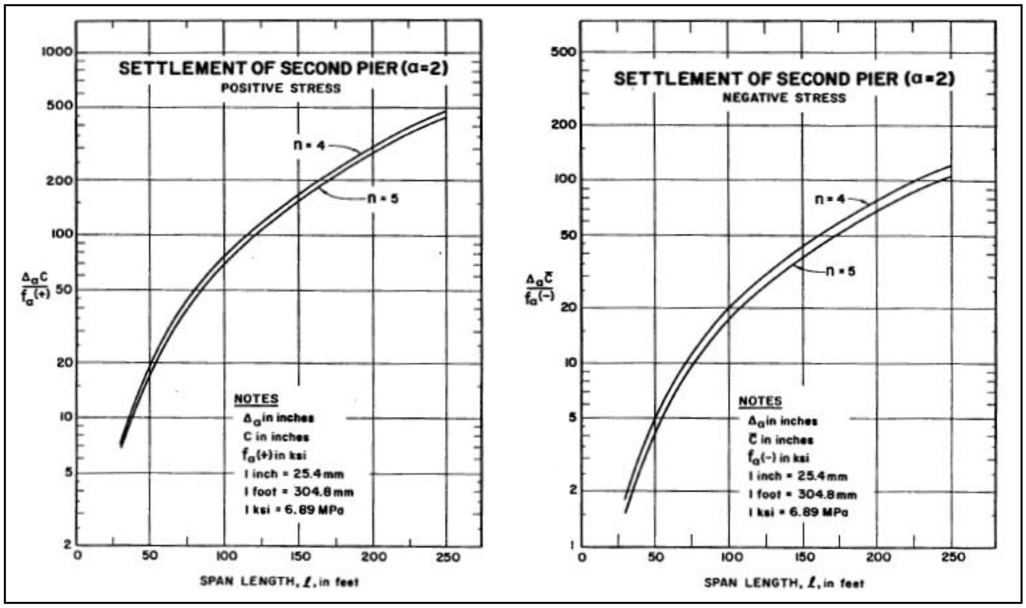
(a) Positive (b) Negative

Figure 2.4.7: Stress increases Due to Abutment Settlement
(Moulton et al., 1985)



(a) Positive (b) Negative

Figure 2.4.8: Stress increases Due to Settlement of First Interior Pier
(Moulton et al., 1985)



(a) Positive (b) Negative

Figure 2.4.9: Stress increases Due to Settlement of Second Interior Pier (Moulton et al., 1985)

In order to properly use the design aids, some basic information is needed. The span length of the bridge (ℓ) and number of spans (n) are required to locate the appropriate ordinate value (Δ*C/f). Once the ordinate value is obtained, the anticipated differential settlement (Δ_o for abutment settlement or Δ_α for pier settlement) and the distance from the neutral axis to the outer fiber (C for positive stress fiber or $\dot{C}$ for negative stress fiber) are used to calculate the maximum stress increase ($f_o^{(+)}$, $f_\alpha^{(+)}$ are positive and $f_o^{(-)}$, $f_\alpha^{(-)}$ are negative) caused by the differential settlement (Moulton et al., 1985).

2.5: Integral Abutments

Traditionally, bridges have been designed to accommodate expansion and contraction due to temperature changes and, to a smaller extent, creep and shrinkage. Various types of expansion joints were often used to accommodate the movement. Expansion joints, however, are associated with a variety of installation and maintenance problems. Studies have found that all phases of bridge service life are negatively impacted (economically) by expansion joints (Kunin & Alampalli, 2000).

In an effort to reduce costs and improve the performance of bridges, integral abutments have emerged. Integral abutment bridges do not contain joints on the bridge. Instead, movement is accommodated by superstructure and substructure moving together. Proper design of integral abutment bridges allows for the necessary expansion and contraction of the superstructure without the need for joints on the bridge or bearings (White, 2007).

Though integral abutment bridges are simple to construct and eliminate many problems, they are complicated structural systems. Designing such a system is a thorough and time-consuming process. Empirically based design processes are often used in an effort to avoid many of the complexities associated with analyzing integral abutment bridges. Such processes are typically conservative in nature and tend to produce reliable structures (White, 2007).

2.5.1: Vertical Movements

Integral abutment bridges have been found to perform well under induced stresses from settlement. Depending on the bridge and settlement conditions, considerable stress increases are possible. However, in bridges that are designed with appropriate foundations and with enough flexibility, the effects of settlement can be negligible (Thippeswamy, GangaRao, & Franco, 2002). Thippeswamy, GangaRao, and Franco go so far as to say that "the effects of settlement as a secondary load can be disregarded in the analysis and design of jointless bridges" (p. 286).

2.5.2: Horizontal Movements

Elimination of expansion joints and bearings significantly reduces the problems that can be caused by horizontal foundation movement. Nearly every issue listed in Section 2.2.2 would not be possible without expansion joints, bearings, and gaps between the girders and abutments. The use of properly designed integral abutment bridges should eliminate most, if not all, of the issues caused by horizontal substructure movement.

Integral abutment bridges are designed to withstand the calculated thermal movements (Kunin & Alampalli, 2000). Though lateral substructure movement of an integral abutment bridge may induce stresses in addition to thermal stresses, the effects would likely be negligible, particularly if conservative empirical methods are used to design the bridge. Additionally, any expected lateral substructure movement could be accounted for in the design in a manner similar to that used to account for thermal movements.

2.6: Construction Sequence

The sequence in which a bridge is constructed can be important because members are only subjected to the settlement that occurs after they have been placed. Much of the settlement that a structure will experience could occur prior to the placement of most structural members if the sequence is thought out properly (Bjerrum, 1963). Construction schedules should be organized such that the sequence of events minimizes post-construction movements. Since any movements that occur prior to superstructure construction do not affect the performance of the structure, fills should be placed and allowed to consolidate prior to construction of the structure. Small movements can always be expected, but measures should be taken to minimize potential movements such that they can be tolerated and do not adversely affect the bridge structure or approaches (Wahls, 1990).

2.7: Design Process

As previously noted, the design engineer is responsible for two general tasks when designing a foundation: determining the differential settlement that can be expected and differential settlement that the structure can tolerate (Bjerrum, 1963, Wahls, 1981). Often, the major damage caused by settlements can be attributed to the fact that the settlements were not accounted for during the design process. Simply incorporating some level of settlement into the structural design can significantly reduce the potential damage to the structure (Feld, 1965).

Golder expanded on the idea of anticipating and designing for settlements by stating that the allowable settlement of a structure should be determined through a joint effort of the building code, inspector, architect, structural engineer, foundation engineer, client, owner, and insurance company. Together, they can decide what the level of settlement they believe the structure should be able to withstand. From there, the structure should be designed in such a way that adjustments to settlement within the structure can be made. The foundation should be designed to limit the settlement to the allowable value decided upon (Golder, 1971).

The economic aspect of tolerable bridge substructure movement must also be examined. Multiple researchers suggested that restricting movement to a tolerable level may be more costly than allowing additional movement and taking measures to mitigate the effects of the movement later in the life of the bridge. For example, the increased costs associated with certain types of foundations would only be acceptable if the use of the more expensive foundations offset the potential maintenance, replacement, and/or failure costs of cheaper, less favorable foundations. In order to obtain an economical design, sound engineering judgment must be used throughout the design process (Yokel, 1990).

2.7.1: Methodology

Moulton et al. strived to create tolerable movement criteria and a design process that would lead to the creation of safe, yet economical bridge designs. The end result was a design methodology intended to aid a designer in creating a bridge that would tolerate any predicted movement. The design methodology "entails a systems approach to the design of highway bridges, whereby the bridge superstructure and its resulting substructure are not designed separately, but as a single integrated system offering the best combination of economy and long-term low-maintenance performance" (Moulton et al., 1985, p. 87).

Based on the goal of the design methodology, Moulton et al. suggested that an iterative approach be taken when designing a bridge. Figure 2.7.1 shows the proposed methodology. First, an appropriate bridge type should be selected based on the geometric constraints of the construction area. Preliminary subsurface conditions should also be taken into consideration when choosing a bridge type. Detailed subsurface conditions should then be obtained so that an appropriate foundation system can be designed. All foundation types should be considered. Geotechnical analyses of the designed foundations would then need to be performed in order to estimate strength and movement parameters. Based on the results of the geotechnical analyses, the preliminary bridge design should be evaluated (Moulton et al., 1985).

The tolerance of the superstructure to the possible foundation movements must be determined using an appropriate method. If the design is deemed appropriate by the tolerable bridge movement criteria, then the engineer can perform cost comparisons in

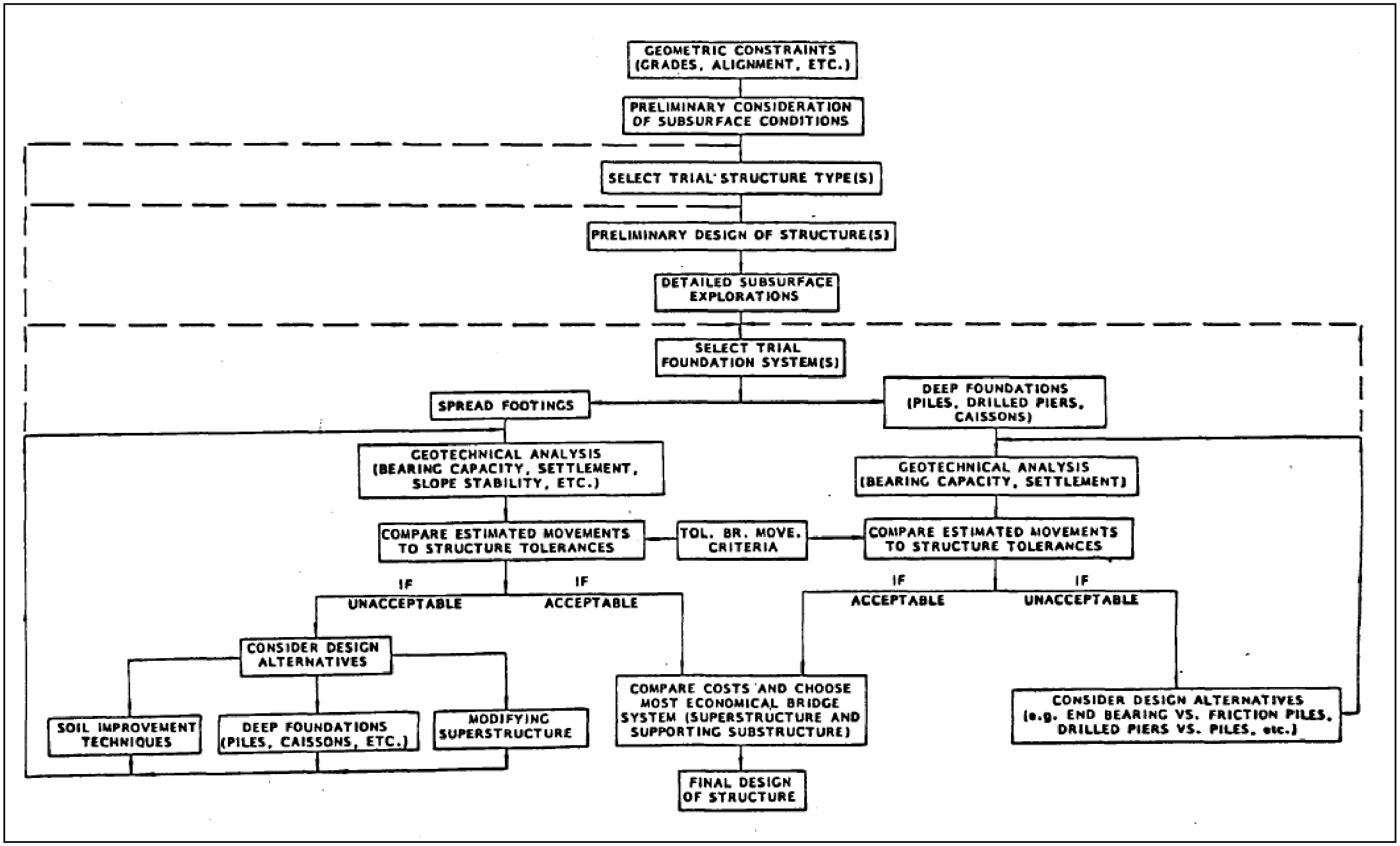


Figure 2.7.1: Design Methodology Proposed by Moulton et al.
(Moulton et al., 1985)

order to provide the owner with an economical bridge system. However, if the tolerable bridge movement criteria suggest that the design is not acceptable, the engineer must alter some or all of the design. The foundation type, foundation design, and superstructure designs are possible areas for modification. In any case, the design modifications will require that the engineer return to steps earlier in the design process (Moulton et al., 1985).

By following this process, an engineer should be able to create two or more appropriate designs. All designs would be expected to perform well over time. The design process should provide the owner with the opportunity to choose the most appropriate and/or economical bridge (Moulton et al., 1985).

Samtani and Nowatzki outlined a three-step process that would allow an engineer to determine the amount of movement a bridge and the associated facilities can tolerate. First, all possible facilities and the movement tolerance of those facilities need to be identified. Next, a conservative value for the expected differential settlement should be determined. The authors suggest using a method such as the one proposed by Duncan and Tan, which will be discussed in the Section 2.7.2. Finally, the differential settlement value should be compared with the tolerances of the facilities. The critical component can then be identified. Altering the critical component or adjusting the constructions sequence may allow for a larger amount of differential settlement. In any case, close coordination between the geotechnical and structural engineers will be crucial (Samtani & Nowatzki, 2006).

2.7.2: Estimating Settlements

One of the biggest issues encountered when designing a bridge is accurate settlement prediction. Nearly all methods for predicting settlement are based on experimental results. These empirical methods generally predict settlements within 50 percent of the measured values (Moulton et al., 1985). In order to reduce the magnitude of any potential differential settlement, Wahls suggested that it is very important to obtain sufficient and accurate information regarding subsurface soil conditions (Wahls, 1990). Even with good soil information, settlement calculations can vary greatly.

Since there is a large margin of inaccuracy associated with movement prediction, Duncan and Tan suggested that a simple differential settlement estimation method be used. Any computed differential settlement could easily be exceeded. Instead, the largest settlement should be assumed to occur at one end of a span and no settlement should be assumed at the other end. This method will provide a large differential settlement and, therefore, a conservative result. In the end, as always, the authors suggest that the designer should use good judgment in selecting a reasonable settlement value and determining whether that value is tolerable (Duncan & Tan, 1991).

Yokel suggests that the differential settlement value used during the design process should be the larger of 75 percent of the maximum total settlement and the difference between the maximum and minimum calculated settlements. If settlements are computed deterministically, the estimates can be considered an upper bound. However, if a more sophisticated method is used to compute settlements, the settlement values used for the design should be at least the mean settlement calculated plus 1.3 standard deviations (Yokel, 1990).

2.7.3: Calculating Tolerable Movements

The tolerable movement limits discussed in Section 2.4 provide a good starting point for engineers. The amount of horizontal movement that can be tolerated by a bridge is largely dependent on the design. Joints and bearings are critical in limiting horizontal movement. In general, limiting the movement to 1.5 inches, as suggested by Moulton et al., should result in tolerable lateral movement. Tolerable differential settlements should be estimated using the limits proposed by Moulton et al. and modified by Duncan and Tan. Angular distortions should be limited to 0.004 for continuous bridges and 0.008 for simply supported bridges.

Samtani and Nowatzki suggested that the angular distortion limits created by Moulton et al. are troubling to some structural engineers. Often engineers limit the angular distortions of a bridge to one-half or one-quarter of the proposed values. Samtani and Nowatzki claim that more than the bridge structure should be considered when investigating tolerable movements. In fact, all of the facilities associated with the bridge (utilities, rails, parapets, sidewalks, etc.) should be analyzed (Samtani & Nowatzki, 2006).

2.7.4: Accounting for Additional Stresses

If an expected movement is determined to be intolerable or the engineer wishes to conservatively design a bridge superstructure, the increase in superstructure stress due to differential settlement should be accounted for by altering the bridge design method. Internal stresses should not develop in simply supported bridges due to vertical movement. Differential settlement will, however, cause additional internal stresses to develop in continuous bridges (Moulton et al., 1985). Three methods of altering the design process in order to account for the additional capacity needed were presented by Moulton et al.

The allowable overstress method would permit stresses to exceed the design stress levels. Since the highest stress that a section experiences occurs infrequently, it is likely that the full load carrying capacity of the member is not being utilized at any given time. Also, bridges behave in a more complex manner than is assumed during the design process and are generally stronger than anticipated when constructed. This method is used in various areas, but generally applies to temporary overstresses (Moulton, et al., 1985).

Working stress design for overloads is a more conservative method. Once the anticipated settlement is calculated, the design aids created by Moulton et al. (presented in Section 2.4.5) or other acceptable methods can be used to determine an approximate stress increase (for continuous steel girder bridges only). Those stresses can be added to the calculated load stresses and the bridge can be designed as it would have been without settlement considerations. Alternatively, the allowable stress can be decreased (Moulton et al., 1985).

Finally, the load factor approach requires that an additional load factor be incorporated in the calculation of member stresses. The load factor would account for the additional stresses caused by settlement. Use of this method would require statistics regarding the reliability of settlement predictions (Moulton et al., 1985). Such statistics, however, were not available and may be difficult to obtain.

2.7.5: LRFD Approach

Yokel discussed Load and Resistance Factor Design (LRFD) and how it can be applied to tolerable foundation settlements. Four failure categories were introduced. Category I was a catastrophic failure, which included major traffic disturbances and/or loss of life. Category II was a major structural failure, which would leave most of a bridge unusable until major repairs were made. Category III was structural distress, which reduced durability or aesthetics. Category IV was functional failure, which led to reduced riding quality, reduced clearance, closing of the joints, improper drainage, etc. Yokel claimed that the probabilities of failure for Categories III and IV could be determined. A cost-benefit analysis could be performed in order to select acceptable probabilities. An example of a cost-benefit analysis is provided in Figure 2.7.2. The probabilities should have a reliability index of 1.3 or greater for a 50- or 100-year design period (Yokel, 1990).

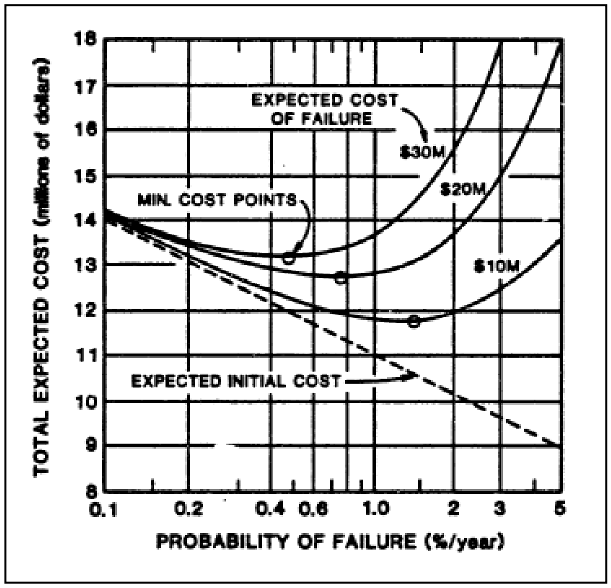


Figure 2.7.2: Example of a Cost-Benefit Analysis
(Yokel, 1990)

2.8: Current Design Code Provisions

The issue of tolerable settlement is briefly addressed in the AASHTO LRFD Bridge Design Specifications. Article 3.12.6 of the specifications states that "force effects due to extreme values of differential settlements among substructures and within individual substructure units shall be considered" (AASHTO, 2010, p. 3-104). The commentary for this section emphasizes that the load combinations (which can be found in Table 3.4.1-1 of the code) that include the settlement term be analyzed for every possible substructure settlement in order to ensure that the critical settlement case is accounted for. Additionally, load combinations should also be applied without consideration of settlement (AASHTO, 2010).

Table 3.4.1-1 of the code requires that settlement be considered with the load combinations for the Strength I, Strength II, Strength III, Strength V, Service I, Service II, and Service IV limit states. The effects of settlement are not to be considered with the Extreme Event or Fatigue limit states. A settlement load factor of 1.0 is given for use with the Service IV limit state. Settlement load factors for all other applicable limit states “should be considered on a project-specific basis” (p 3-12). If adequate information for determining an appropriate load factor is not available, AASHTO suggests using a load factor of 1.0 (AASHTO, 2010).

Although the specifications state that foundation movement shall be considered, an exact method of analysis is not provided. Instead, Article 10.5.2.2 addresses the issue by stating that “tolerable movement criteria shall be established by either empirical procedures or structural analyses, or by consideration of both” (AASHTO, 2010, p 10-28). Furthermore, the code informs the reader that “settlement shall be investigated using all applicable loads in the Service I Load Combination” (AASHTO, 2010, p 10-28). When evaluating horizontal movements and rotations, all applicable service limit state load combinations are to be investigated (AASHTO, 2010).

The choice of analysis method is left to the engineer. The code recognizes, however, that bridges are likely to accommodate more movement than calculated during the design process. Creep, relaxation, and force redistribution all allow for additional movements. AASHTO provides suggested angular distortion limits (0.008 for simple spans and 0.004 for continuous spans) produced by past research as potential guidelines for a bridge engineer. Bridge designers are urged to consider all aspects of the design including the cost of mitigation through larger foundations, realignment, or surcharge; rideability; aesthetics; and safety. When investigating horizontal movement tolerances, bridge seat/joint widths, bearing types, structure type, and load distribution effects are all key factors (AASHTO, 2010).

AASHTO, like many of the tolerable bridge movement researchers, encourages engineers to investigate the economic aspect of foundation movements. The commentary for Article 10.5.2.1 states that “the cost of limiting foundation movements should be compared with the cost of designing the superstructure so that it can tolerate larger movements or of correcting the consequences of movements through maintenance to determine minimum lifetime cost” (AASHTO, 2010, p 10-28). In the end, however, the harshness of the movement criteria is up to the bridge owner.

Chapter 3

RESEARCH APPROACH

Aside from the tolerable limit alteration by Duncan and Tan, the research performed by Moulton et al. has not been updated. The original analytical study utilized the AASHTO Standard Specifications for Highway Bridges to design and analyze various girders. Because the AASHTO LRFD Bridge Design Specifications replaced the Standard Specifications, the analytical research performed by Moulton et al. needs to be updated so that the results are more meaningful to current bridge designs.

Analyzing simplistic bridges provides a good starting point and a straightforward method of investigating bridge behaviors; however, evaluating more complex bridge systems would provide further insight into the response of various bridges to differential settlement. In addition to updating the simple girder models, detailed analyses of actual bridges should be performed. Simple models provide a good theoretical base, but models of actual bridges in service provide additional data that can be used to determine the accuracy of the simplistic models.

Because the effects of horizontal substructure movements generally do not include significant changes in member forces, investigating tolerable horizontal movements analytically would be difficult. In addition, the increased use of integral abutment bridges is likely to reduce the damaging effects of horizontal movement in new bridges. For these reasons, only vertical differential substructure movements were investigated.

No survey data was collected as a part of this study; hence, the accuracy of the empirical studies performed by past researchers was not investigated. The remainder of this thesis will focus primarily on analytical studies of differential vertical movement.

3.1: Bridge Types

In order to produce meaningful results, bridges representative of current design and construction practices need to be investigated. The most commonly designed and constructed highway bridges at the present time are continuous steel plate girder and prestressed concrete I-girder bridges. Analysis of both steel and concrete girders will allow for the response of bridges of both materials to be observed and compared. In addition, both two- and three-span bridges of varying span lengths need to be analyzed in order to study the influence of span length and number of spans on the response of a bridge to differential settlement.

Highway bridges employing rolled steel girders are less common than in the past; however, studying rolled girder bridges will allow for comparisons to be made between the present study and the study performed by Moulton et al. Since the effects of differential settlement are more clearly observed in short-span bridges and rolled steel girders are generally used in short-span bridges, rolled girder bridges fit well into the analytical studies.

Other bridge types, such as steel trusses and concrete box girders, will not be considered as part of the present study. Trusses are generally used for long-span bridges and are known to be tolerant of fairly large differential settlements. Concrete box girders used in long-span bridges will also be reasonably tolerant to differential settlements. Short-span concrete box beams will not be investigated.

Although simply supported bridges are somewhat common, continuous bridges are generally preferred when multiple spans are necessary. Since simply supported bridges are free to rotate at both ends, the girders should not be affected by differential vertical movement. Thus, settlement analysis of simply supported bridges is not relevant. Simply supported bridges will not be investigated as part of the analytical studies.

3.2: Reproduction of Past Results

In order to effectively proceed forward with updating the past analytical research, the original results obtained by Moulton et al. needed to be reproduced. Reproduction of the original research would allow for the past research to be confirmed and an effective analytical method to be determined. Once an effective approach is decided upon, the methodology could then be extended to analyses based on the current design code.

Difficulty was encountered when attempting to reproduce the original results due to the lack of information provided by Moulton et al. The structural members used in the analytical studies were vaguely described. For example, the depths of the wide-flange rolled steel sections designed were given, but not the unit weight. For plate girders, no dimensions were given. Prestressed concrete beam types were given, but no details of the strand patterns were presented. Several reports and journal articles authored by Moulton and his associates were examined in an attempt to obtain the needed information. All of the literature describing the work by Moulton et al. lacked the necessary information.

In addition to the lack of section information, the live load moments used by Moulton et al. during the girder analyses could not be reproduced. Also, impact loading was not included in the live load moment calculations and, therefore, was not considered during the stress increase calculations (Haslebacher, 1980). Since impact loadings are used in the design of flexural members, they should not be omitted when investigating moment or stress increases due to settlement.

For the sake of simplicity, only two-span continuous rolled steel girder bridges with span lengths ranging in length from 30 to 60 feet were investigated in an attempt to replicate the results obtained by Moulton et al. Reproduction of bridges longer than 60 feet was determined to be unnecessary and reproduction of the prestressed concrete bridges was determined to be impractical due to the numerous variables involved. Bridge girders were designed and analyzed in an attempt to reproduce the stress increase data given in Figures 2.4.1 and 2.4.2.

3.2.1: Girder Design

Due to the lack of details describing the previous analytical studies, the bridge girders designed by Moulton et al. could not be modeled. Instead, rolled steel girders were designed using the Allowable Stress Design (ASD) provisions presented in the AASHTO Standard Specifications for Highway Bridges. The work by Haslebacher (who worked in conjunction with Moulton et al.) suggests that the girders designed by Moulton et al. were assumed to be fabricated from Grade 36 steel. Though Grade 36 steel is no longer used to fabricate bridge girders, it was assumed in an attempt to accurately reproduce the results obtained by Moulton et al. Girders were also designed using Grade 50 steel in order to examine the effects of stronger steel. Additionally, Grade 50 steel girders designed using the Standard Specifications could then be compared to the girders designed using LRFD.

Figure 3.2.1 displays the bridge cross-section used in the study by Moulton et al. The same cross-section was used to design the girders used in the current study. An 8.5-inch composite concrete deck with a 0.5-inch sacrificial wearing surface was assumed. Design moments were obtained by utilizing the dead load of the steel girders, concrete deck, curbs, and railings and the HS-20 live load specified in the Standard Specifications. Dead loads were evenly distributed to the girders and the live load was distributed to the girders using the wheel load distribution specified in the Standard Specifications. Section properties were calculated and used to compute the critical stresses produced by the applied loads. W36 steel sections with adequate resistances were selected in order to be consistent with the analyses performed by Moulton et al. In addition to selecting appropriate W36 steel sections, alternate Grade 36 and Grade 50 wide flange steel sections were chosen for each bridge.

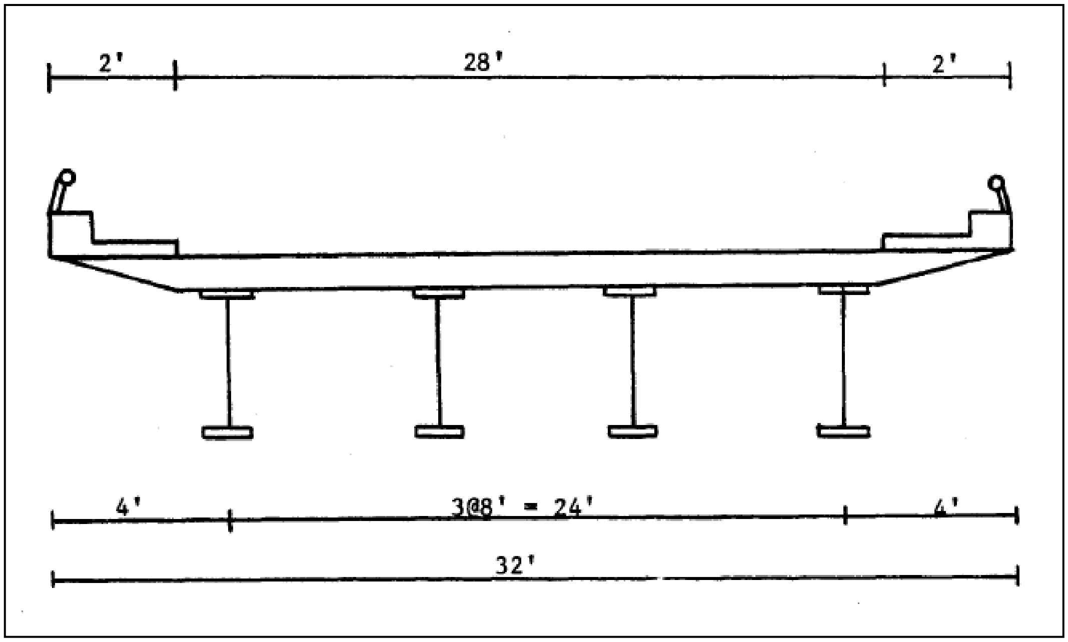


Figure 3.2.1: Bridge Cross-Section Assumed for Design
(Haslebacher, 1980)

3.2.2: Settlement Analyses

Once designed, STAAD.Pro (Research Engineers International, 2004) was used model and analyze the rolled steel sections. Traditionally, composite concrete decks are assumed to only assist the girders in resisting the applied loads when the member is experiencing positive flexure. When designing composite steel girders, it is generally assumed that the composite section resists positive moments and the steel section alone resists negative moments. For settlement analysis, however, the composite cross-section of a given girder should be utilized to determine force effects in both positive and negative moment regions. Because stiffness is a key component in the response of a girder to differential settlement, the use of the composite section over the length of the bridge ensures that the actual stiffness of the bridge is considered. Including the stiffness of the deck will provide more accurate results that err on the side of conservatism.

The effect of differential vertical movement was examined by applying settlements of 1, 2, and 3 inches to either the exterior or center support in order to produce maximum negative or maximum positive stress, respectively. Figure 3.2.2 shows the settlement of an exterior support and the expected location of the stress increase. The center support settlement scenario is displayed in Figure 3.2.3. Moulton et al. only investigated the increase in positive stress at the location of the maximum positive stress caused by the applied loads, which is at, or near, the mid-point of the loaded span of a two-span continuous bridge. The maximum positive stress experienced by the girder due to differential settlement, however, will occur over the center support, as indicated in Figure 3.2.3. In addition to investigating the stress increase at the center of the first span, the positive stress over the center support was also examined.

In an attempt to reproduce the results reported by Moulton et al., moment increases (which are proportional to stress increases) were reported as percentages as compared to the maximum negative and positive moments caused by the applied loads. In order to obtain the maximum negative and positive moments, dead loads were

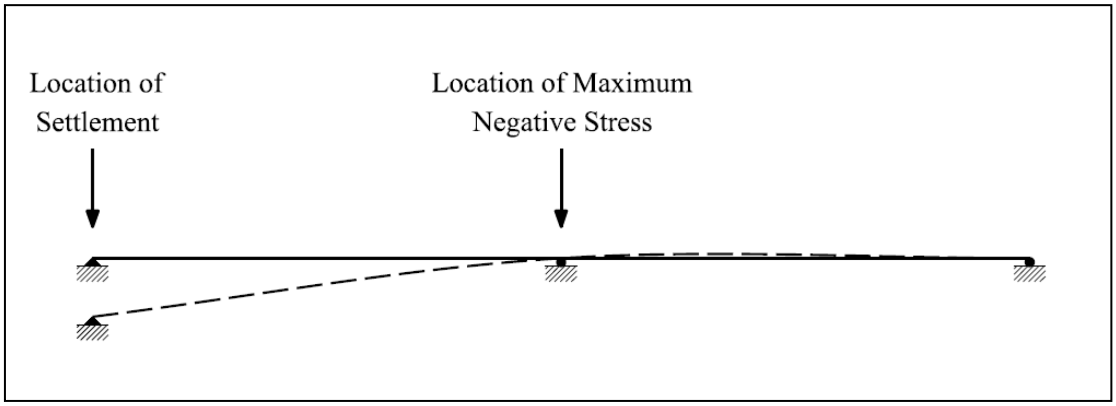


Figure 3.2.2: Exterior Support Settlement of a Two-Span Continuous Bridge

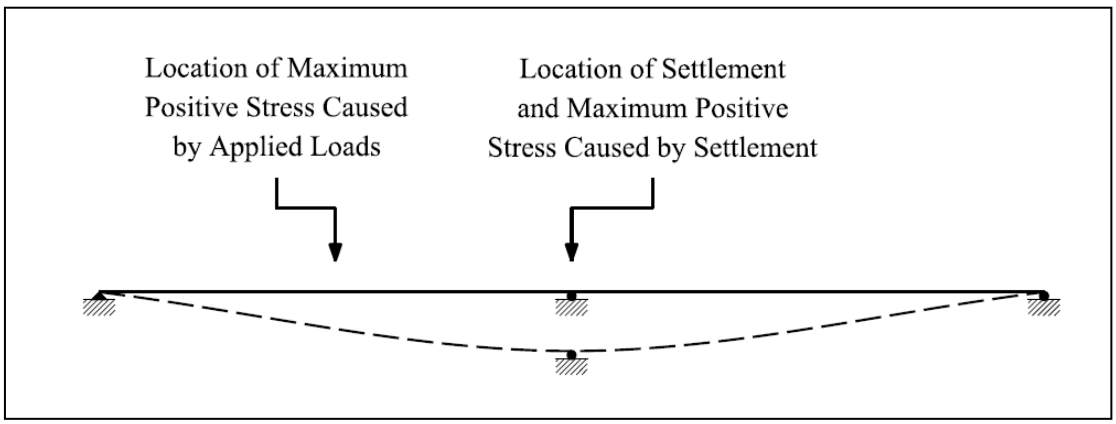


Figure 3.2.3: Interior Support Settlement of a Two-Span Continuous Bridge

applied over the length of the bridge and live loads were positioned for maximum effect as specified in the Standard Specifications. Percent increases in moment due to settlement were calculated by dividing the negative or positive moment caused by differential settlement at the location of interest by the corresponding maximum negative or positive moment caused by the applied loads.

3.2.3: Tolerance to Differential Settlement

While representing the effects of differential settlement by calculating percent increases in member stresses is interesting and allows for certain trends to be observed, no conclusions regarding the ability of a girder to tolerate the assumed movement can be drawn. Instead, the moment increase caused by differential settlement must be compared to the reserve moment capacity of each girder. Such a comparison provides a simple method of determining the tolerance of a given bridge to differential vertical movements.

Once the moments caused by settlement were computed for each bridge, the tolerance of that bridge to differential settlement was investigated. By comparing the moments caused by differing magnitudes of settlement to the reserve moment capacity of a given girders, the ability of the bridge to resist the applied substructure settlements was determined. Tolerances to both exterior and interior support settlements were investigated.

Two situations were examined for each bridge. First, the tolerance of a given bridge to differential settlement was determined by limiting the member stresses to the allowable stress limit specified in the Standard Specifications. The tolerances were compared to the angular distortion limit proposed by Moulton et al., which specifies that angular distortions be limited to 0.004 inches per inch for continuous bridges in order to limit any damage caused by the differential settlement to a tolerable level. Applying the angular distortion limit to the bridges analyzed suggests that differential settlements up to 1.44, 1.92, 2.40, and 2.88 inches should be tolerable for bridges with span lengths of 30, 40, 50, and 60 feet, respectively.

The angular distortion limit suggested by Moulton et al. is based on empirical data because in-service bridges are known to behave differently than assumed during the design process. Since bridges are known to be more tolerant to differential settlement than suggested by simple analyses of the bridge, the tolerance of the studied bridges is expected to be larger than determined through analysis.

In an attempt to produce more realistic differential settlement tolerances, the settlement data was re-analyzed assuming that the girders had more reserve moment capacity than initially assumed. The additional capacity was determined by allowing girder stresses to exceed the design limits and reach the yield stress. Realistically, once a bridge is in service, the maximum stress experienced by the girders is not limited to a portion of the yield stress of the girder material. Structural damage would generally not be noticed until yielding of the members begins. If girder stresses are allowed to exceed the design limits, then the tolerance of the girders to stresses caused by differential settlement will be larger and more realistic.

3.3: Updated Results

The bridges designed and analyzed in an attempt to reproduce the work by Moulton et al. were also designed using the AASHTO LRFD Bridge Design Specifications. Studying the effects of differential settlement on bridges designed using the current code will shed light on the tolerance of newly designed bridges to differential substructure movements.

3.3.1: Girder Design

The cross-section displayed in Figure 3.2.1 was again utilized in the design of rolled steel girders. The same assumptions made during the Standard Specifications design process were used for the LRFD designs. In order to produce girder designs representative of current design practice, the use of Grade 50 steel was assumed. Dead loads were again distributed equally among the girders. The HL-93 live load specified in the LRFD Bridge Design Specifications was applied to the girders using distribution factors calculated as specified in the code. Factored nominal moments were then calculated by applying the appropriate load factors. The Strength I limit state was found to control the girder designs. Note that nominal flexural resistances were calculated based on the provisions of Appendix A6 in the LRFD Specifications.

Again, adequate W36 steel sections were selected, which allowed for comparisons to be made with the girders designed using the Standard Specifications. In addition to determining appropriate W36 sections, smaller, more economical sections were selected as alternatives in order to provide more realistic results.

3.3.2: Settlement Analyses

STAAD.Pro was used to investigate the effects of 1-, 2-, and 3-inch differential settlements on the simple LRFD bridge models. Settlements were applied to the exterior support in order to produce maximum negative moment and to the center support in order to produce maximum positive moment. The girders were again modeled as fully composite in both the negative and positive moment regions in order to properly account for the total stiffness of the composite members.

Moment increases due to differential settlement were again represented as percentages based on the unfactored maximum negative and positive moments caused by the applied loads. In order to produce the maximum moments, dead loads were applied over the entire length of the bridge and live loads were positioned for maximum effect as specified in the LRFD Specifications. Unfactored load moments were utilized in order to provide a better comparison to the moment increases determined for the bridged designed using the Standard Specifications. Moment increases were again calculated by dividing the settlement moment by the corresponding maximum load moment.

3.3.3: Tolerance to Differential Settlement

The tolerance of the bridge girders designed using the LRFD Bridge Design Specifications to differential settlement was investigated by examining the moment increases due to substructure settlements. The internal moments caused by exterior and interior settlements of 1, 2, and 3 inches were compared to the reserve moment capacities of the bridge girders. Reserve moment capacities were initially calculated using the Strength I limit state load factors since Strength I was determined to be the controlling limit state during the design of the bridge girders. Because actual bridges in service can generally withstand more differential settlement than determined analytically (as discussed in Section 3.2.3), additional analyses were performed. In order to provide a more realistic scenario, reserve moment capacities were also calculated by using the unfactored moments caused by the applied loads. The results of tolerance studies using both methods of computing the reserve girder capacity were compared to the angular distortion limit proposed by Moulton et al.

3.4: In-Service Bridge Analyses

In order to investigate the responses of actual bridges to differential settlement, detailed line girder analyses were performed using AASHTOWare, specifically the Opis/Virtis software package (AASHTO, 2009). A database of bridges was obtained from the Strategic Highway Research Program (SHRP) 2 R19B research project. A total of 24 bridges were chosen from the database and analyzed using Virtis. Three different bridge types were investigated: rolled steel girder, steel plate girder, and prestressed concrete I-girder bridges. Each bridge type was represented by both two- and three-span continuous bridges. Various span lengths were chosen for each bridge type. Table 3.4.1 lists the bridge identification number, girder type, and span lengths of each two-span bridge. Bridge numbers, girder types, and span lengths of the three-span bridges can be found in Table 3.4.2. Further details of the selected bridges, including material and section properties, can be found in Appendix A.

An interior girder from each of the selected bridges was subjected to two unit differential settlements. For each bridge, an exterior support was assumed to settle in

Table 3.4.1: Two-Span Continuous Bridge Properties

Bridge ID	Bridge Type	Span 1 (ft)	Span 2 (ft)
1166	Prestressed Concrete I-Girder	37.25	37.25
15991	Prestressed Concrete I-Girder	82.41	82.41
7715	Prestressed Concrete I-Girder	107.13	107.13
2937	Rolled Steel Girder	45.00	45.00
6114	Rolled Steel Girder	64.75	64.75
16464	Rolled Steel Girder	93.50	93.50
16830	Steel Plate Girder	67.26	67.26
7953	Steel Plate Girder	84.32	84.32
16266	Steel Plate Girder	108.27	108.27

order to investigate the negative stress increase over the adjacent interior support. The exterior support settlement scenario for a two-span continuous bridge was previously shown in Figure 3.2.2. Figure 3.4.1 displays a general exterior support settlement scenario for three-span continuous bridges. Imposing settlement at the first interior support allowed for the positive moment increases occurring in the first interior span and over the settled interior support to be investigated. For three-span bridges, the positive moment increase at the mid-point of the center span was also examined. Additionally, negative moment increases due to interior support settlements for three-span bridges were studied. The interior support settlement scenario for two-span continuous bridges was given in Figure 3.2.3. The scenario for three-span continuous bridges is shown in Figure 3.4.2.

Table 3.4.2: Three-Span Continuous Bridge Properties

Bridge ID	Bridge Type	Span 1 (ft)	Span 2 (ft)	Span 3 (ft)
8787	Prestressed Concrete I-Girder	26.75	41.00	26.75
8442	Prestressed Concrete I-Girder	50.25	56.00	50.25
3015	Prestressed Concrete I-Girder	74.30	96.78	74.30
3191	Rolled Steel Girder	33.23	30.02	33.23
16548	Rolled Steel Girder	34.00	102.00	34.00
3766	Rolled Steel Girder	34.67	35.375	34.67
4789	Rolled Steel Girder	54.14	67.26	54.14
2178	Rolled Steel Girder	73.00	85.00	73.00
14035	Steel Plate Girder	52.49	68.90	52.49
8873	Steel Plate Girder	67.00	155.00	67.00
8850	Steel Plate Girder	74.25	99.00	74.25
1620	Steel Plate Girder	89.92	117.52	89.73
2735	Steel Plate Girder	115.40	91.24	99.60
8894	Steel Plate Girder	125.00	240.00	125.00
14096	Steel Plate Girder	188.98	202.43	216.54

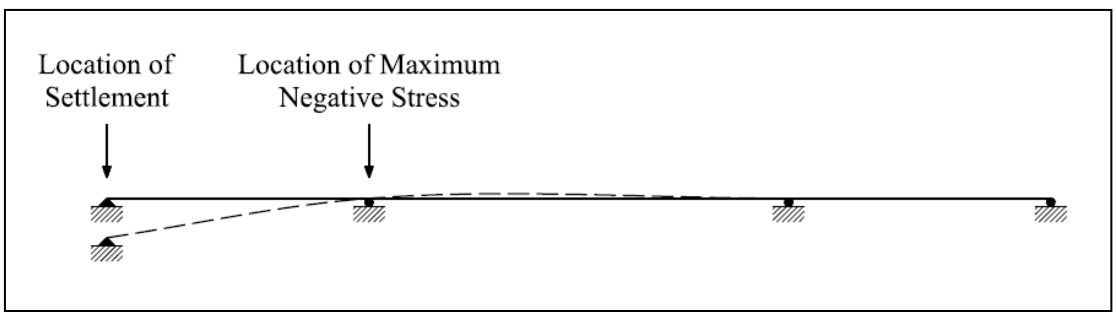


Figure 3.4.1: Exterior Support Settlement of a Three-Span Continuous Bridge

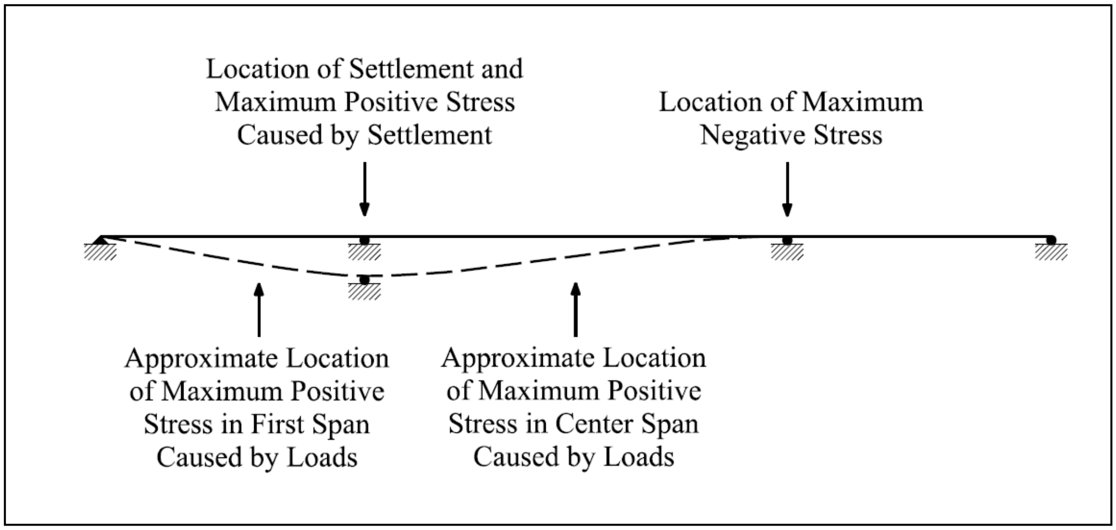


Figure 3.4.2: Interior Support Settlement of a Three-Span Continuous Bridge

3.4.1: Settlement Analyses

A typical analysis was first performed on each bridge using Virtis. Virtis provided the moments caused by each applied load at tenth points along each bridge span. The moment capacity and allowable flexural stress of the girder at each analysis point was also provided. Upon completion of the initial analysis, settlement conditions were imposed. Again, the moment caused by each settlement condition was provided at the tenth points of each bridge girder. The results of the original analysis and settlement analysis were then collected and used to investigate the response of each bridge to different differential settlement conditions.

The moments caused by the applied differential settlements were again presented as percent increases as compared to the maximum unfactored load moments. Though the percentages cannot be used to comment on the tolerance of a bridge to differential settlement, certain trends can likely be observed. Differences in the responses of bridges with different types of girders or different span lengths may be noticeable.

3.4.2: Tolerable Differential Settlement

The moment capacity and settlement moment data obtained from the analyses of the in-service bridges was used to determine the tolerance of each bridge to the applied differential settlements. The reserve moment capacities of the bridge girders were determined by two different methods. First, the Strength I limit state factored nominal moment was computed and subtracted from the total flexural resistance of each girder. An alternate moment capacity was calculated by subtracting the unfactored load moments from the total flexural resistance of each girder. The alternate reserve moment capacity was calculated in an attempt to produce more realistic results, as was discussed in Section 3.2.3. The moments experienced by each member due to the specified differential settlements were then compared to the calculated moment capacities in order to determine the tolerance of each bridge to differential settlements.

3.5: Relationship between Stiffness and Settlement-Induced Stresses

In order to apply the findings of the analyses described in the previous sections to a wide range of bridge girders, the general relationship between a given girder and the stress increase experienced when that girder is subjected to differential settlement must be investigated. Since stiffness is the major factor affecting the amount of stress induced by settlement, it is logical to use the moment of inertia (I) and span length (ℓ) of a given girder to estimate the moment increase that will be experienced due to differential settlement.

In addition to the moment of inertia and span length, the number of bridge spans also affects the stiffness of a bridge. Bridges with two to six spans were analyzed in order to study the influence of number of bridge spans on the moments caused by differential settlement. Various composite sections were subjected to differential settlements of 1, 2, and 3 inches in order to study the effect of settlement. Plotting moment increase versus moment of inertia divided by the span length (I/ℓ) as suggested by Moulton et al. was found to be ineffective and potentially misleading. Instead, plotting the moment due to settlement versus the moment of inertia of the composite section divided by the square of the girder span length (I/ℓ^2) was found to be the best method of investigating the relationship between member stiffness and settlement stresses.

The relationship between girder displacement and the moment caused by that displacement can be observed by considering beam theory for a cantilever beam, such as the one shown in Figure 3.5.1. Equation 3.5.1 shows that the moment experienced by a beam is proportional to the second derivative of the displacement of that beam.

$$\frac{d^2u}{dx^2} = \frac{M}{EI} \tag{3.5.1}$$

where u is the displacement, x is the location along the beam, M is the moment in the member, E is the modulus of elasticity of the girder material, and I is the moment of

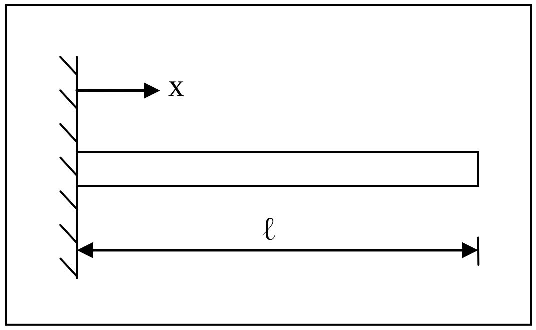


Figure 3.5.1: Typical Cantilever Beam

inertia of the beam cross-section. Integrating Equation 3.5.1 twice with respect to x gives

$$u=\frac{M}{2EI}x^2+C_1x+C_2 \tag{3.5.2}$$

where C_1 and C_2 are constants of integration. In order to solve for C_1 and C_2, two boundary conditions must be known. For a cantilever beam, the displacement at $x = 0$ (with x as defined in Figure 3.5.1) is zero. The slope (the derivative of displacement) at $x = 0$ is also equal to zero. Taking the boundary conditions into account and rearranging displacement equation provides an equation that will calculate the moment at the fixed end of a cantilever beam due to a specified displacement at $x = \ell$.

$$M=\frac{2uEI}{\ell^2} \tag{3.5.3}$$

The units of the variables in Equation 3.5.3 are as follows: M in kip-inches, u in inches, E in kips per square inch, I in quartic inches, and ℓ in inches.

Though the girders investigated as part of this study are not cantilevered, the continuity of the spans restricts the movement of the girders and allows only small rotations at the continuous end. Because the support opposite the settled support is assumed to not move and the moment increase due to no support settlement is zero, the moment equation for continuous girders will take the same form as Equation 3.5.3. Making a minor change to Equation 3.5.3 that allows for more general use gives

$$M_{SE} = \frac{C_s u E I}{12\ell^2} \tag{3.5.4}$$

where M_{SE} is the moment caused by the settlement, u, and C_s is a unitless differential settlement coefficient determined from the analyses of various bridges with different numbers of spans, span lengths, and stiffnesses. The numeral 12 placed in the denominator converts the resulting moment to kip-feet.

Fitting a linear curve through the data obtained from the analyses provides a slope in the following form:

$$M_{SE} = S\frac{I}{\ell^2} \tag{3.5.5}$$

where S is the slope of the fitted line. Comparing Equations 3.5.4 and 3.5.5, it is obvious that the equations are of the same form. If Equations 3.5.4 and 3.5.5 are to describe the same systems, then

$$S = \frac{C_s u E}{12} \tag{3.5.6}$$

or

$$C_s = \frac{12S}{uE} \tag{3.5.7}$$

The results of the analyses of various bridges containing the same number of spans provides a C_s value that can then be used along with the desired support settlement to determine the theoretical moment caused by an applied support settlement. Analyzing different groups of bridges with different numbers of spans allows for a trend in the resulting C_s values to be determined. The final result would ideally be an equation or set of equations used to determine a C_s value for the bridge of interest. That C_s value can then be used to calculate an expected moment due to a specified magnitude of settlement.

Note that the stiffness analyses were only performed using bridges with all spans of equal length. Additionally, cross-sectional properties were held constant over the entire length of each bridge.

3.6: Bridge Design Considering Settlement

Although the analyses presented in Sections 3.2.3, 3.3.3, and 3.4.2 attempt to prove that bridges can realistically withstand some level of differential settlement without including the increased stresses caused by settlement in the design process, ignoring the effects of differential settlement when designing a bridge could potentially lead to structural damage or serviceability issues. The reasoning behind including settlement-induced moments in the design of bridges is discussed in more detail in Section 5.5.

In order to investigate the effects of including the moments caused by differential settlement in the design process, the alternate girders designed using the AASHTO LRFD Bridge Design Specifications (as discussed in Section 3.3.1) were redesigned to be able to accommodate settlement moments caused by 3 inches of differential settlement. Since the moments induced by settlement are affected by girder dimensions, the process was iterative. In the end, girders capable of tolerating a 3-inch differential settlement occurring at either the exterior or interior support were selected. A load factor of 1.0 was applied to the settlement moments when calculating the factored nominal moment.

Chapter 4

RESULTS OF ANALYTICAL STUDIES

The results of the differential settlement analyses described in Chapter 3 are presented in the following sections and discussed in Chapter 5. While Moulton et al. chose to present their results as stress increases, the results of this study are presented as moment increases. For elastic analyses, the stress experienced by a member at a given location is a function of the moment at that location; thus, the moment is proportional to the stress. Therefore, reporting moment increases (as percentages) is an equivalent to reporting stress increases. Additionally, studying moments is more appropriate when investigating bridges designed using the LRFD Specifications.

4.1: Reproduction of Past Results

The difficulties encountered in attempting to reproduce the analytical results obtained by Moulton et al., including the lack of section and load information, required that the two-span continuous bridges of interest be designed prior to analysis. The bridges were designed using the Allowable Stress Design provisions of the Standard Specifications. Appropriate W36 rolled steel girders were selected in an attempt to reproduce the results obtained by Moulton et al., which were presented in Figures 2.4.1 and 2.4.2. In addition, the bridges were redesigned in an attempt to create economical designs. The responses of both groups of bridges to differential vertical movements were analyzed.

4.1.1: Moment Increases in W36 Bridges

The lightest W36 section (a W36x135) was found to limit girder stresses caused by the applied loads to an allowable level for the 30-, 40-, and 50-foot span bridges. A W36x160 girder was found to be appropriate for the 60-foot span bridge. Note that the full capacities of the W36x135 girders were not being utilized in any of the three bridges. The W36x135 and W36x160 girders with composite concrete decks were then subjected to differential settlement.

Figure 4.1.1 shows the theoretical increases in maximum negative moment over the center support due to settlement of the exterior support for two-span continuous bridges with span lengths of 30, 40, 50, and 60 feet. Figure 4.1.2 shows the theoretical increases in positive moment at the mid-point of the first span due to settlement of the center support. The maximum theoretical increases in positive moment, which are presented in Figure 4.1.3, were found by comparing the maximum positive moment caused by settlement (over the center support) to the maximum positive moment caused by the applied loads (near the mid-point of the first span) for a given girder.

4.1.2: Moment Increases in Alternate Bridges

In addition to the W36 girders, lighter, more economical girders were designed using the AASHTO Standard Specifications for Highway Bridges. When designed

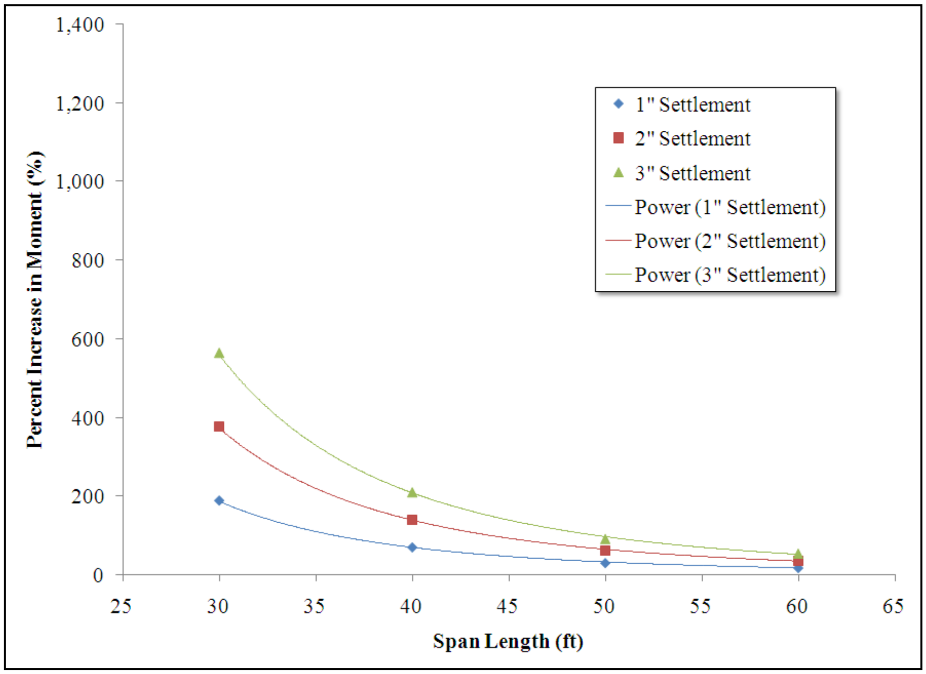


Figure 4.1.1: Negative Moment Increases in W36 Girders

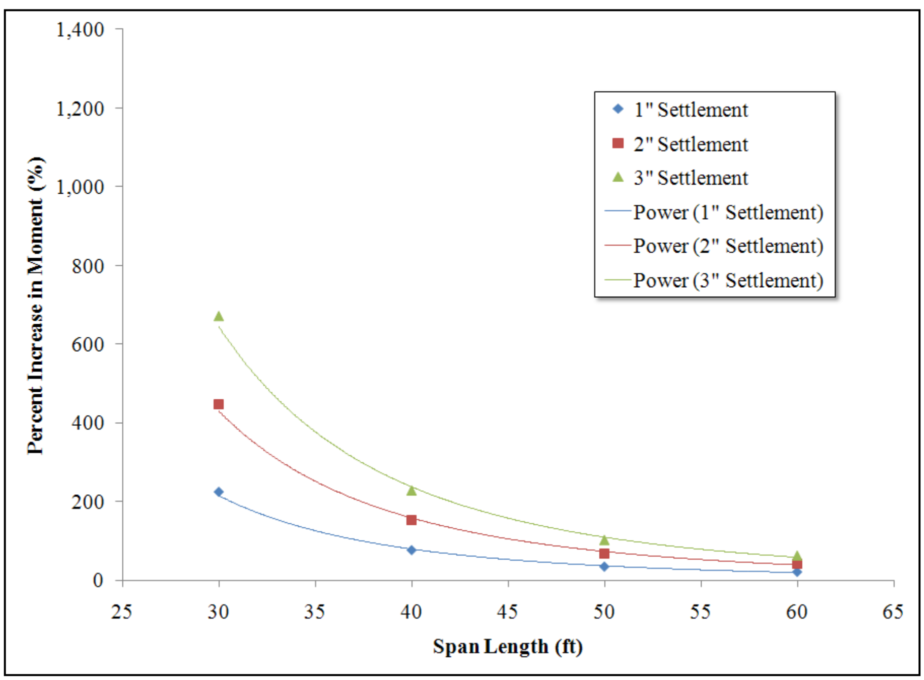


Figure 4.1.2: Positive Moment Increases in W36 Girders at Mid-Span

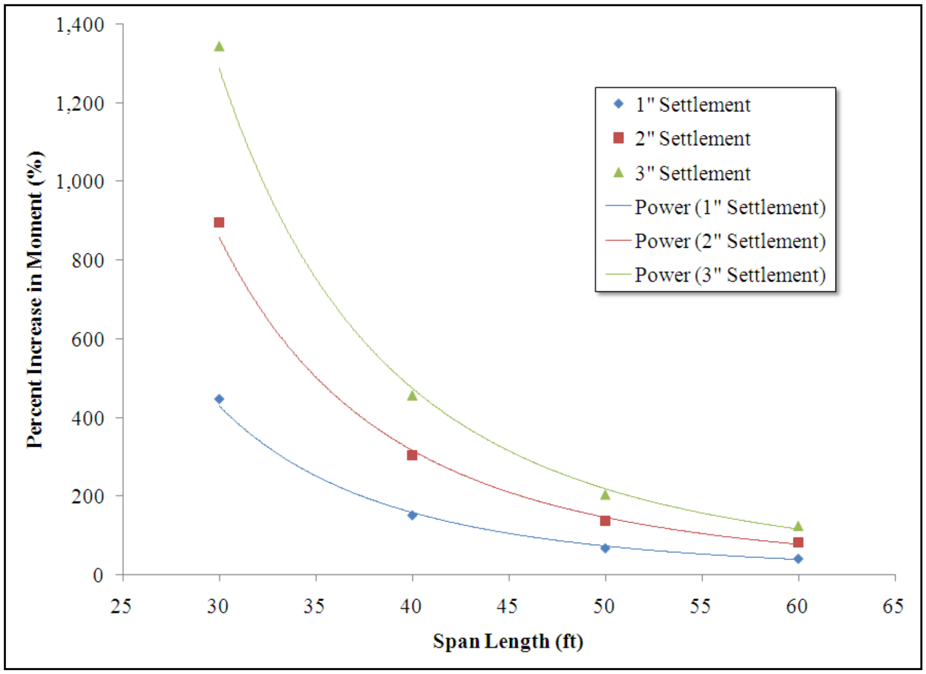


Figure 4.1.3: Positive Moment Increases in W36 Girders at Center Support

using Grade 36 steel, W24x62 girders were chosen for the 30-foot span bridge, W30x90 girders were chosen for the 40-foot span bridge, and W33x118 girders were chosen for the 50-foot span bridge. The selected girders were found to resist the required load much more efficiently than the W36x135 girders. W36x160 girders were again used for the 60-foot span bridge.

The rolled steel girders were also designed assuming Grade 50 steel was used. Design with the higher strength steel resulted in W21x50 girders being selected for the 30-foot span bridge, W24x68 girders being selected for the 40-foot span bridge, W30x90 girders being selected for the 50-foot span bridge, and W33x118 girders being selected for the 60-foot span bridge.

Both sets of alternate bridge girders were subjected to the same settlement analyses as the W36 girders. Figure 4.1.4 compares the negative moment increases for the W36 girders and the two sets of alternate girders. The increases in positive moment at the mid-point of the first span and overall in the girder are compared in Figures 4.1.5 and 4.1.6, respectively.

4.1.3: Tolerance of W36 Girders to Settlement

The tolerance of the W36 girders were first determined assuming that internal member stresses should be limited to the design stresses. The tolerance of investigated bridges to differential settlement can be quantified by determining the allowable angular

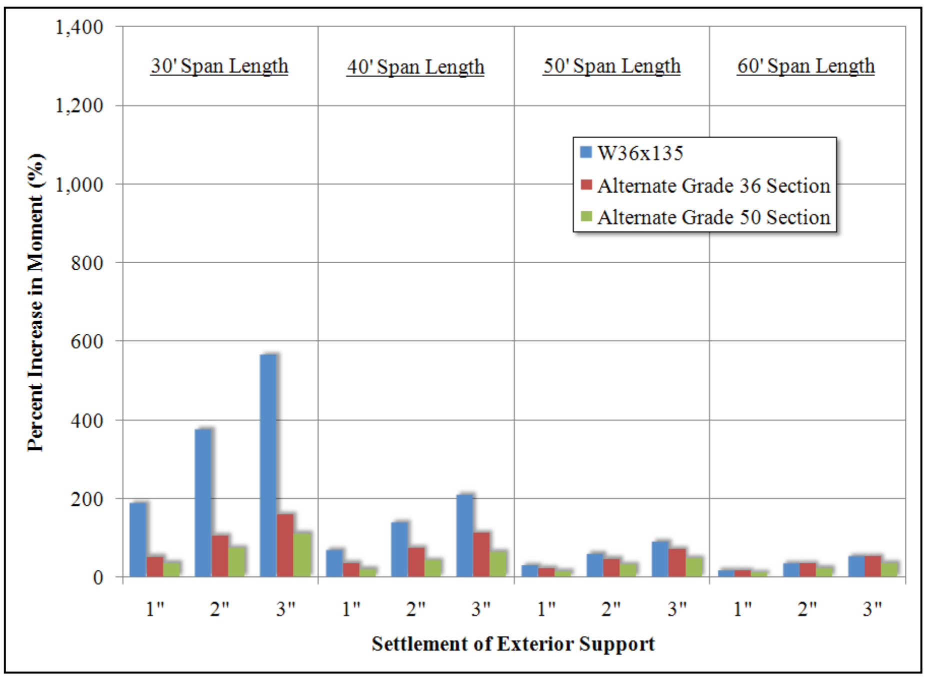


Figure 4.1.4: Comparison of Negative Moment Increases

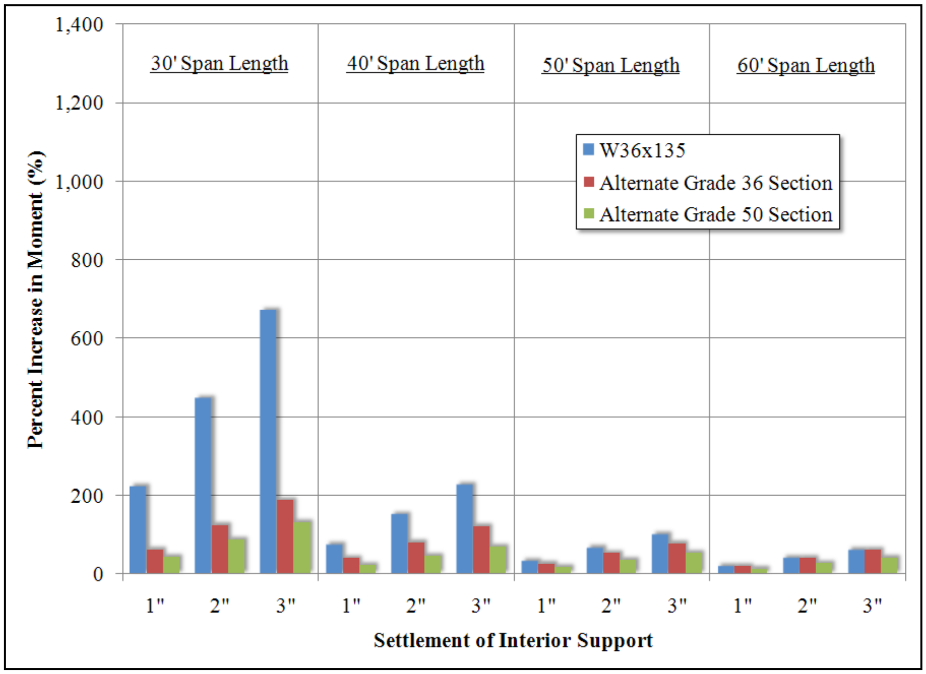


Figure 4.1.5: Comparison of Positive Moment Increases at Mid-Span

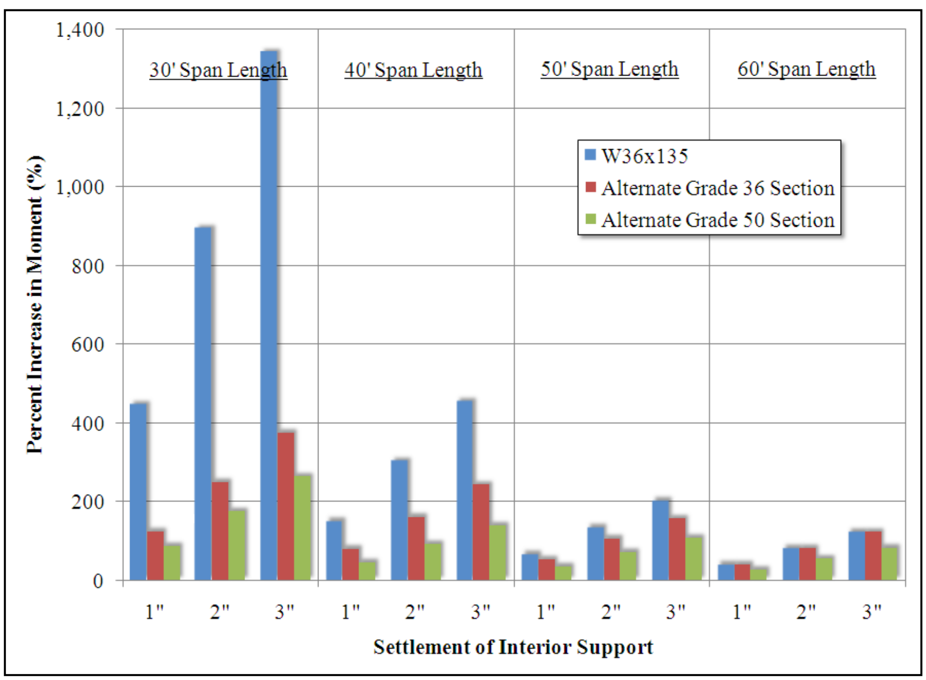


Figure 4.1.6: Comparison of Positive Moment Increases at the Center Support

distortion of each bridge. Differential settlement of the exterior support of two-span continuous bridges will always be limited by the ability of the bridge girder to resist the negative moment caused by the settlement. Based on the moment capacities of the girders investigated, tolerable angular distortions were calculated for each W36 girder bridge. The results are presented in Table 4.1.1. Likewise, based on the moments induced by the center support settlement for the critical positive moment case and the flexural resistances of the bridge girders, the allowable angular distortions for each bridge were calculated. The location of the critical positive moment for the W36 girders was determined to be at the center support for the 30-foot span bridge and at mid-span for the other bridges. The differential settlement tolerances based on limiting the positive moment at mid-span are given in Table 4.1.2.

In order to determine whether the bridges could realistically withstand more settlement than suggested by the limits given in Tables 4.1.1 and 4.1.2, further analyses were needed. Table 4.1.3 shows the tolerance of the W36 bridges to exterior support

Table 4.1.1: Tolerable Exterior Support Settlements for W36 Girders Based on Design Stress

Span Length (ft)	**Allowable Differential Settlement (in)**	**Allowable Angular Distortion (in/in)**
30	0.96	0.0027
40	1.17	0.0024
50	0.76	0.0013
60	0.34	0.0005

Table 4.1.2: Tolerable Center Support Settlements for W36 Girders Based on Design Stress

Span Length (ft)	**Allowable Differential Settlement (in)**	**Allowable Angular Distortion (in/in)**
30	0.94	0.0026
40	1.59	0.0033
50	1.59	0.0027
60	1.78	0.0025

settlement assuming that yield stress is used as the allowable stress limit. When considering the yield stress as the allowable stress limit, the critical positive moment location was found to be at the center support for the 30-, 40-, and 50-foot span bridges and at mid-span for the 60-foot span bridge. The resulting center support settlement limits are provided in Table 4.1.4.

Table 4.1.3: Tolerable Exterior Support Settlements for W36 Girders Based on Yield Stress

Span Length (ft)	**Allowable Differential Settlement (in)**	**Allowable Angular Distortion (in/in)**
30	2.15	0.0060
40	3.25	0.0068
50	4.00	0.0067
60	5.09	0.0071

Table 4.1.4: Tolerable Center Support Settlements for W36 Girders Based on Yield Stress

Span Length (ft)	**Allowable Differential Settlement (in)**	**Allowable Angular Distortion (in/in)**
30	1.60	0.0045
40	2.96	0.0062
50	4.92	0.0082
60	7.07	0.0098

4.1.4: Tolerance of Grade 36 Alternate Girders to Settlement

Tolerances of the Grade 36 steel girders designed with efficiency in mind were investigated. Table 4.1.5 contains the tolerable exterior support settlement data for the Grade 36 alternate girder bridges. Positive moment tolerances at mid-span were found to be critical for all of the bridges. Tolerable center settlements determined based on limiting the positive stress at mid-span are given in Table 4.1.6. The moment capacities used to compute the tolerances were calculated assuming internal girder stresses were limited to the design stresses.

Increasing the allowable stress limit to the yield stress of the girder material creates additional moment capacity in the bridge girders. The exterior support settlement limits determined to keep settlement-induced stresses below the yield stress of the member are given in Table 4.1.7. Moment increases at mid-span were again found to limit the magnitude of center support settlement that could be tolerated. Table

Table 4.1.5: Tolerable Exterior Support Settlements for Grade 36 Alternate Girders Based on Design Stress

Span Length (ft)	**Allowable Differential Settlement (in)**	**Allowable Angular Distortion (in/in)**
30	0.03	0.0001
40	0.27	0.0006
50	0.14	0.0002
60	0.34	0.0005

Table 4.1.6: Tolerable Center Support Settlements for Grade 36 Alternate Girders Based on Design Stress

Span Length (ft)	**Allowable Differential Settlement (in)**	**Allowable Angular Distortion (in/in)**
30	0.49	0.0014
40	0.78	0.0016
50	1.06	0.0018
60	1.78	0.0025

4.1.8 contains the center support settlement limits determined based on limiting mid-span stresses to the yield stress of the girder.

4.1.5: Tolerance of Grade 50 Alternate Girders to Settlement

Tolerable differential settlements for the alternate girders fabricated with Grade 50 steel were also investigated. Assuming the design stresses limit the flexural

Table 4.1.7: Tolerable Exterior Support Settlements for Grade 36 Alternate Girders Based on Yield Stress

Span Length (ft)	**Allowable Differential Settlement (in)**	**Allowable Angular Distortion (in/in)**
30	1.55	0.0043
40	2.61	0.0054
50	3.56	0.0059
60	5.09	0.0071

Table 4.1.8: Tolerable Center Support Settlements for Grade 36 Alternate Girders Based on Yield Stress

Span Length (ft)	**Allowable Differential Settlement (in)**	**Allowable Angular Distortion (in/in)**
30	2.15	0.0060
40	3.38	0.0070
50	4.89	0.0081
60	7.07	0.0098

resistances of the girders, the tolerances of the girders to differential settlement at the exterior and center supports were determined. Table 4.1.9 provides the differential settlement and angular distortion limits for settlement of the exterior support. Mid-span positive stresses were found to be critical and, therefore, limit settlement of the center support. Table 4.1.10 gives the center support settlement limits.

Table 4.1.9: Tolerable Exterior Support Settlements for Grade 50 Alternate Girders Based on Design Stress

Span Length (ft)	Allowable Differential Settlement (in)	Allowable Angular Distortion (in/in)
30	0.17	0.0005
40	0.10	0.0002
50	0.17	0.0003
60	0.09	0.0001

Table 4.1.10: Tolerable Center Support Settlements for Grade 50 Alternate Girders Based on Design Stress

Span Length (ft)	Allowable Differential Settlement (in)	Allowable Angular Distortion (in/in)
30	0.78	0.0022
40	0.85	0.0018
50	1.45	0.0024
60	2.29	0.0032

Allowing internal stresses to reach the yield stress of the girders increases the moment carrying capacity of the bridges. Exterior support settlement limits based on limiting negative stresses above the center support are given in Table 4.1.11. Tolerable center support settlements determined from limiting the positive stresses at mid-span to the yield stress of the girders are presented in Table 4.1.12.

Table 4.1.11: Tolerable Exterior Support Settlements for Grade 50 Alternate Girders Based on Yield Stress

Span Length (ft)	Allowable Differential Settlement (in)	Allowable Angular Distortion (in/in)
30	2.49	0.0069
40	3.93	0.0082
50	5.29	0.0088
60	6.88	0.0096

Table 4.1.12: Tolerable Center Support Settlements for Grade 50 Alternate Girders Based on Yield Stress

Span Length (ft)	Allowable Differential Settlement (in)	Allowable Angular Distortion (in/in)
30	3.28	0.0091
40	5.05	0.0105
50	7.15	0.0119
60	10.01	0.0139

4.2: Updated Results

The AASHTO LRFD Bridge Design Specifications were used to design and analyze two sets of bridges. Two-span continuous bridges utilizing W36 composite girders were again used as a basis of comparison. Alternate girders were also chosen in an attempt to increase the efficiency and economy of the designs. Both sets of bridges were subjected to differential vertical movements in order to gain an understanding of the effects of differential settlement on bridges design using the LRFD Bridge Design Specifications.

4.2.1: Moment Increases in W36 Bridges

W36x135 sections fabricated from Grade 50 steel were found to sufficiently carry the required loads for all four bridges. Moment increases due to settlement experienced by the W36 girders were then investigated. Figure 4.2.1 shows the theoretical increases in negative moment in W36 girders for span lengths of 30, 40, 50, and 60 feet subjected to exterior support settlements of 1, 2, and 3 inches. Theoretical increases in positive moment near the mid-point of the first span due to settlement of the center support are presented in Figure 4.2.2. Increases in positive moment above the settled center support as compared to the maximum positive moment experienced by the girder are displayed in Figure 4.2.3. Note that unfactored load moments were used to calculate moment increases due to settlement.

4.2.2: Moment Increases in Alternate Bridges

Alternatives to the W36x135 girders were also designed. W21x57, W24x84, W30x90, and W30x99 sections were found to be adequate for resisting the loads carried by the 30-, 40-, 50-, and 60-foot spans, respectively. The theoretical increases in negative moment for the various bridges are compared in Figure 4.2.4. Figure 4.2.5 shows the difference in theoretical positive moment increases near the mid-point of the first span for the different girders. Overall theoretical increases in positive moment for

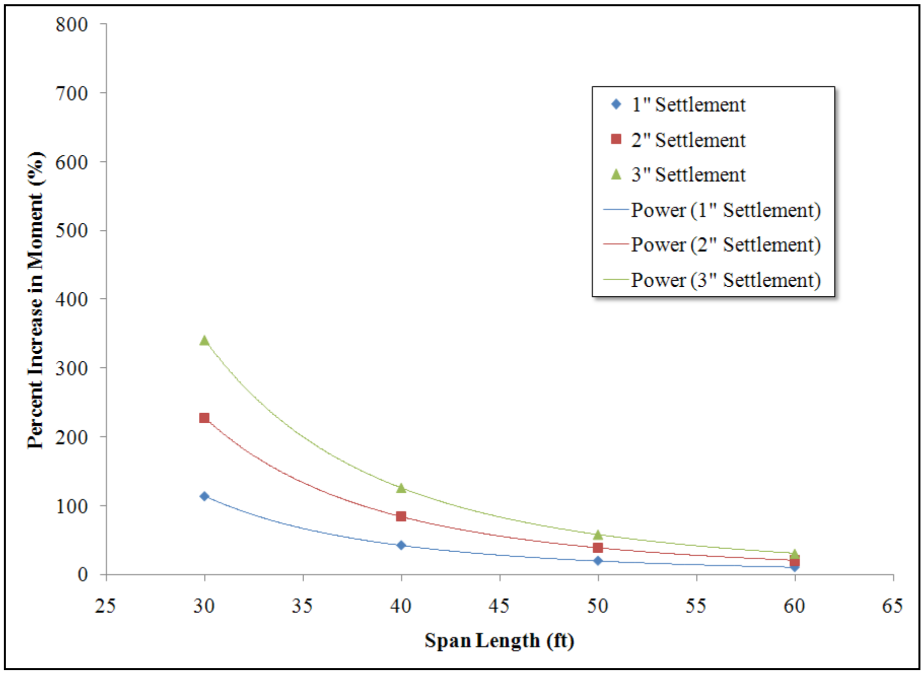


Figure 4.2.1: Negative Moment Increases for W36 Girders

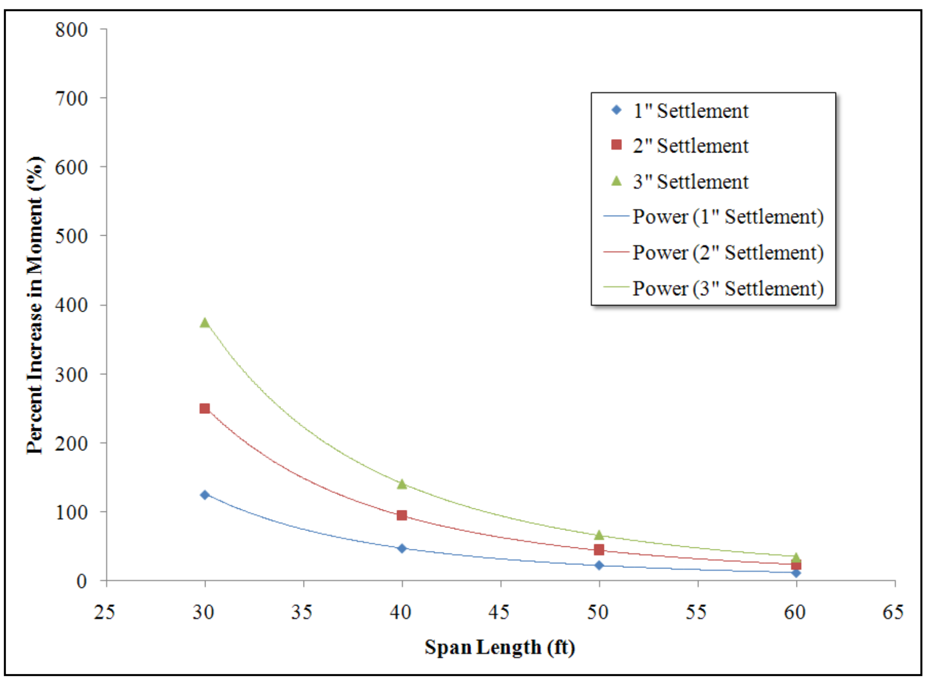


Figure 4.2.2: Positive Moment Increases in W36 Girders at Mid-Span

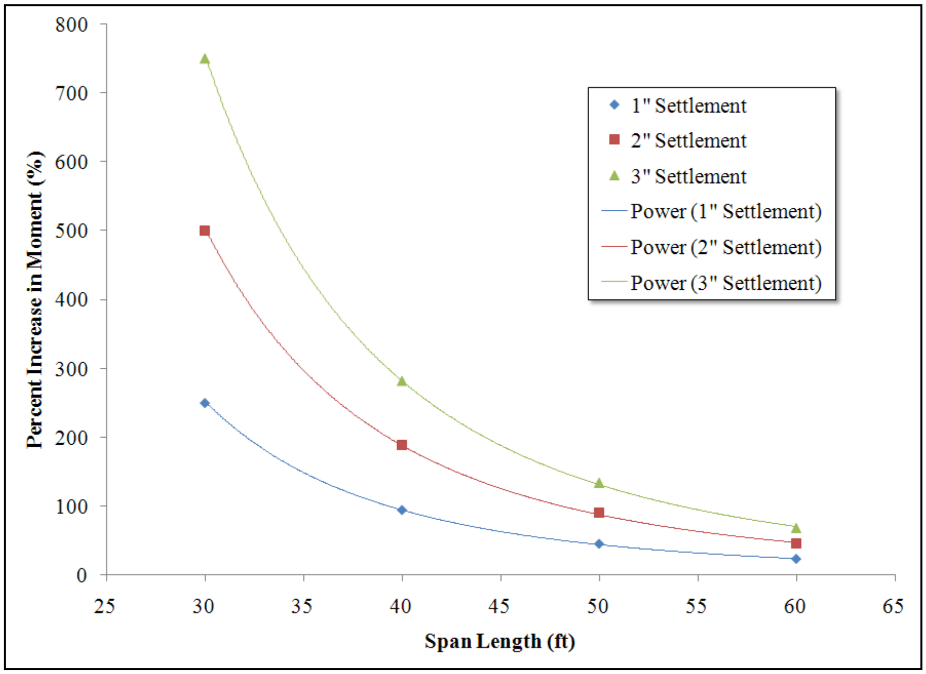


Figure 4.2.3: Positive Moment Increases in W36 Girders at Center Support

the W36 and alternate girders are compared in Figure 4.2.6. Again, unfactored moments were used in the moment increase calculations.

4.2.3: Tolerance of W36 Girders to Settlement

Once again, allowable angular distortions were investigated as a means of quantifying the tolerance of studied bridges to differential vertical movement. The negative and positive reserve moment capacity of each W36 girder bridge was calculated by taking the nominal flexural resistance of the girder and subtracting the factored nominal load moment determined by applying the Strength I limit state load factors. The reserve capacities, along with settlement-induced moments, were then used

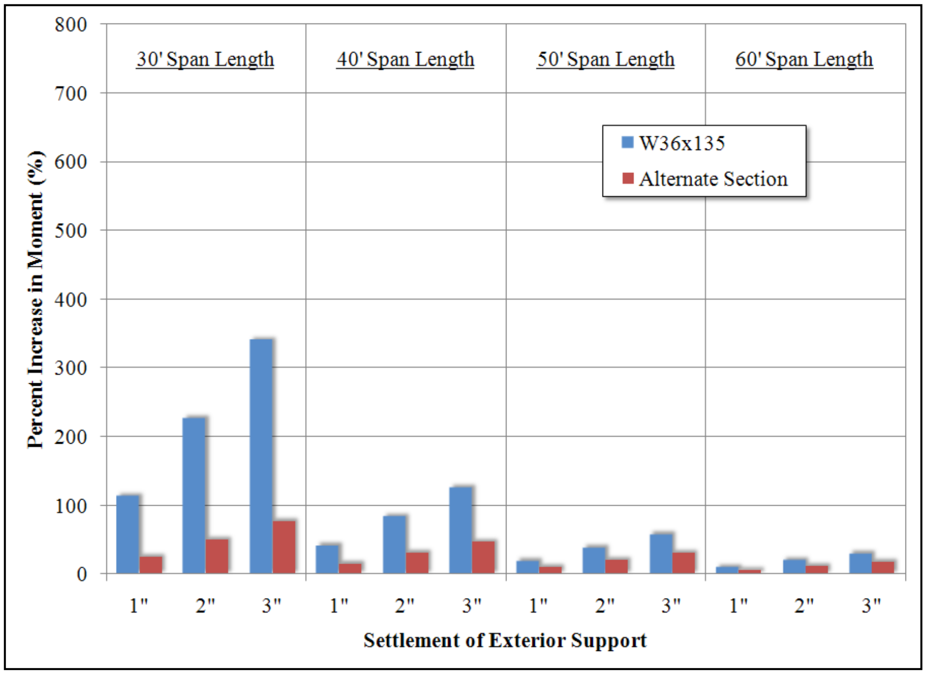


Figure 4.2.4: Comparison of Negative Moment Increases

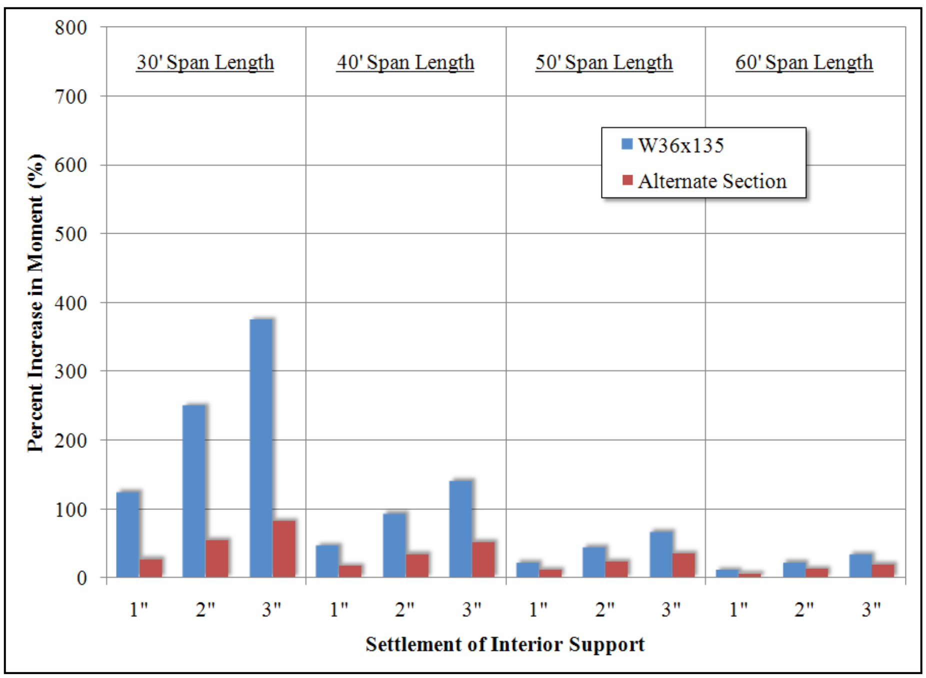


Figure 4.2.5: Comparison of Positive Moment Increases at Mid-Span

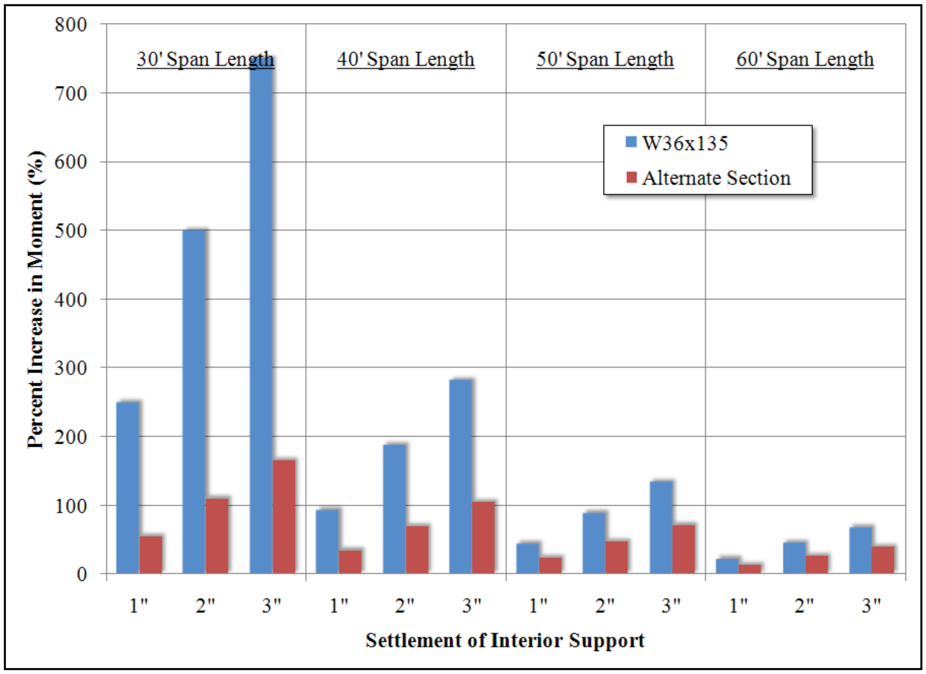


Figure 4.2.6: Comparison of Positive Moment Increases at Center Support

to determine the largest support settlement that would be considered tolerable. Exterior support settlement limits determined by the reserve moment capacities in the girders at the location of the center support are given in Table 4.2.1. Positive moment increases at mid-span and over the center support need to be considered when determining the allowable settlement of the center support. For the W36 girders, the positive moment over the center support was found to be critical. Table 4.2.2 presents the corresponding center support settlement limits.

In-service bridges are known to tolerate larger magnitudes of differential settlement than predicted through typical structural analysis. In an attempt to produce more realistic tolerable angular distortion values, reserve moment capacities calculated

Table 4.2.1: Tolerable Exterior Support Settlements for W36 Girders Based on Factored Moment Capacity

Span Length (ft)	**Allowable Differential Settlement (in)**	**Allowable Angular Distortion (in/in)**
30	3.65	0.0101
40	5.46	0.0114
50	6.79	0.0113
60	6.87	0.0095

Table 4.2.2: Tolerable Center Support Settlements for W36 Girders Based on Factored Moment Capacity

Span Length (ft)	**Allowable Differential Settlement (in)**	**Allowable Angular Distortion (in/in)**
30	3.69	0.0102
40	6.68	0.0139
50	10.71	0.0179
60	15.93	0.0221

using the nominal flexural resistance of the W36 girders and the unfactored load moments were used to determine the tolerable exterior and center support differential settlement values. Negative moments occurring in the girders at the center support limit the exterior support settlement to the values given in Table 4.2.3. Moment increases at the center support were again found to be critical. Table 4.2.4 provides the tolerable center support settlement limits.

Table 4.2.3: Tolerable Exterior Support Settlements for W36 Girders Based on Unfactored Moment Capacity

Span Length (ft)	**Allowable Differential Settlement (in)**	**Allowable Angular Distortion (in/in)**
30	3.94	0.0109
40	6.07	0.0126
50	7.86	0.0131
60	8.57	0.0119

Table 4.2.4: Tolerable Center Support Settlements for W36 Girders Based on Unfactored Moment Capacity

Span Length (ft)	**Allowable Differential Settlement (in)**	**Allowable Angular Distortion (in/in)**
30	3.69	0.0102
40	6.68	0.0139
50	10.71	0.0179
60	15.93	0.0221

4.2.4: Tolerance of Alternate Girders to Settlement

The tolerance of the efficiently design girders to differential vertical movement was also investigated. Tolerable exterior support settlements determined based on limiting the negative moments over the center support are provided in Table 4.2.5. Positive moment increases at the center support determined the allowable settlements of the center support for the 30-, 40-, and 50-foot span bridges. Allowable center support settlements for the 60-foot span bridge was computed based on positive moment increases at mid-span. Tolerable center support settlements are summarized in Table 4.2.6. All reserve moment capacities were calculated utilizing factored load moments.

Table 4.2.5: Tolerable Exterior Support Settlements for Alternate Girders Based on Factored Moment Capacity

Span Length (ft)	**Allowable Differential Settlement (in)**	**Allowable Angular Distortion (in/in)**
30	0.08	0.0002
40	0.29	0.0006
50	2.74	0.0046
60	0.97	0.0013

Table 4.2.6: Tolerable Center Support Settlements for Alternate Girders Based on Factored Moment Capacity

Span Length (ft)	**Allowable Differential Settlement (in)**	**Allowable Angular Distortion (in/in)**
30	5.68	0.0158
40	9.51	0.0198
50	12.88	0.0215
60	16.13	0.0224

The use of reserve moment capacities calculated utilizing unfactored load moments allows for tolerable angular distortion limits that are potentially more realistic than the results given in Tables 4.2.5 and 4.2.6 to be examined. Table 4.2.7 contains the exterior support settlement limits found by considering the negative moment increases at the center support. Center support settlement limits based on positive moment increases are given in Table 4.2.8. Positive moment increases experienced at the center support controlled for all but the 60-foot span bridge.

Table 4.2.7: Tolerable Exterior Support Settlements for Alternate Girders Based on Unfactored Moment Capacity

Span Length (ft)	**Allowable Differential Settlement (in)**	**Allowable Angular Distortion (in/in)**
30	0.82	0.0023
40	1.38	0.0029
50	4.24	0.0071
60	3.21	0.0045

Table 4.2.8: Tolerable Center Support Settlements for Alternate Girders Based on Unfactored Moment Capacity

Span Length (ft)	**Allowable Differential Settlement (in)**	**Allowable Angular Distortion (in/in)**
30	5.68	0.0158
40	9.51	0.0198
50	12.88	0.0215
60	17.78	0.0247

4.3: In-Service Bridge Analyses

The models of actual in-service rolled steel girder, steel plate girder, and prestressed concrete I-girder bridges were subjected to an exterior support settlement of 1 inch and a center support settlement of 1-inch. Since moments induced by settlement increase linearly, the results of the settlement analyses were extended to include 2 and 3 inches of settlement. The settlement moments were then compared to the moments caused by the applied loads. The percent increases were calculated based on unfactored load moments.

The responses of the bridges to the vertical movements were also used to investigate the tolerance of in-service bridges to differential settlement. Examination of settlement tolerance data for real bridges provides insight into the need for tolerable movement limits and allows for the accuracy of the simplistic models to be determined. Additionally, the effect of girder type, varying stiffnesses, and varying span lengths can be studied.

4.3.1: Two-Span Continuous Bridges

Table 4.3.1 contains the moment increase data for two-span continuous bridges subjected to exterior support settlement. The increases in negative moment experienced the each girder type due to 1 inch of differential settlement are displayed in Figure 4.3.1. Center support settlement was also investigated. Increases in positive moment at the location of the maximum positive load moment are provided in Table 4.3.2. Additionally, the maximum positive moment, which occurs at the center support, was compared to the maximum positive moment caused by the applied loads. The positive moment increases experienced at the center support are also given in Table 4.3.2. For each girder type, the positive moment increases at the two locations caused by 1 inch of settlement are shown graphically in Figure 4.3.3.

Table 4.3.1: Negative Moment Increases in In-Service Bridge Girders

Girder Type	**Span (ft)**	***Percent Moment Increase***		
		Settlement		
		1"	**2"**	**3"**
Rolled Steel	45	7.9	15.9	23.8
	64.75	5.4	10.7	16.1
	93.5	3.4	6.9	10.3
Steel Plate	67.26	7.6	15.2	22.7
	84.32	4.6	9.2	13.8
	108.27	1.3	2.5	3.8
P/S Concrete	37.25	67.8	135.7	203.5
	82.41	43.0	86.0	129.0
	107.13	14.5	28.9	43.4

4.3.2: Three-Span Continuous Bridges

Three-span continuous bridges are more complex in nature than two-span continuous bridges and, thus, produce more complicated results when analyzed. Two negative moment cases and three positive moment cases were examined. Table 4.3.3 contains the data describing the negative moment increases occurring in the girder of interest at the location of the first interior support due to settlement of the adjacent exterior support for rolled steel girder, steel plate girder, and prestressed concrete I-

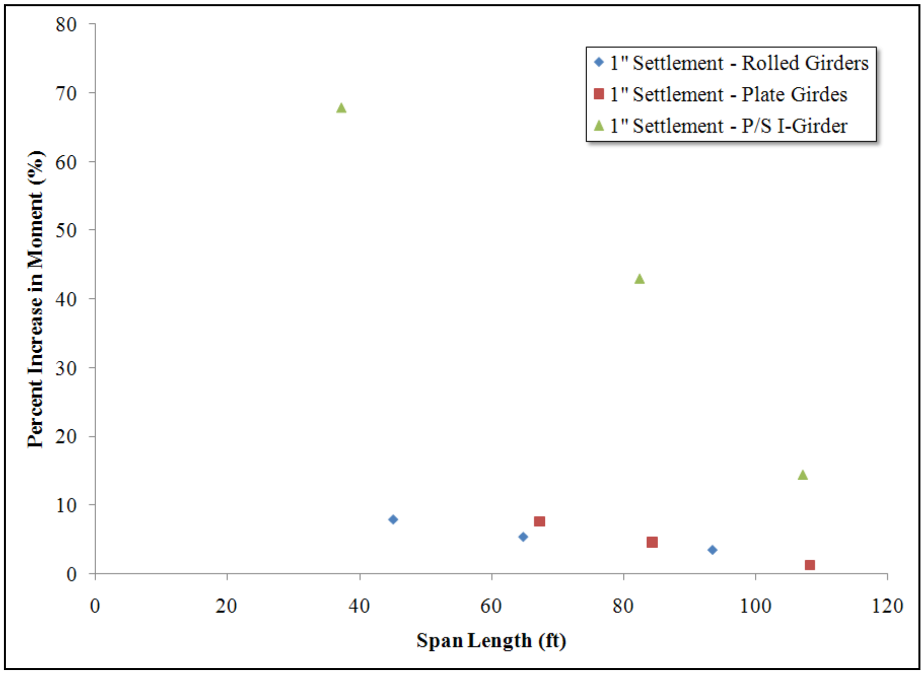


Figure 4.3.1: Negative Moment Increases in In-Service Bridge Girders

Table 4.3.2: Positive Moment Increases in In-Service Bridge Girders

Girder Type	**Span (ft)**	***Percent Moment Increase at Mid-Span***			***Percent Moment Increase at Center Support***		
		Settlement			**Settlement**		
		1"	**2"**	**3"**	**1"**	**2"**	**3"**
Rolled Steel	45	7.2	14.3	21.5	13.3	26.7	40.0
	64.75	4.1	8.3	12.4	10.4	20.7	31.1
	93.5	4.7	9.4	14.1	9.4	18.8	28.1
Steel Plate	67.26	7.2	14.3	21.5	17.9	35.8	53.8
	84.32	6.2	12.4	18.6	12.4	24.8	37.2
	108.27	1.9	3.9	5.8	4.8	9.7	14.5
P/S Concrete	37.25	40.2	80.4	120.6	100.5	200.9	301.4
	82.41	20.9	41.7	62.6	41.7	83.5	125.2
	107.13	7.4	14.8	22.2	14.8	29.5	44.3

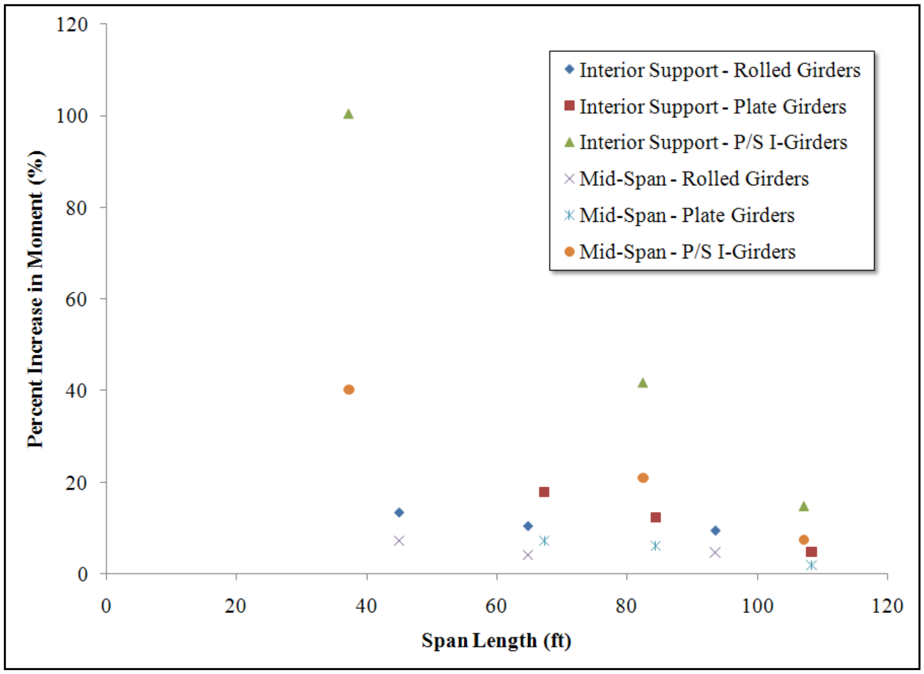


Figure 4.3.2: Positive Moment Increases in In-Service Bridge Girders

girder bridges. Negative moment is also induced when one of the interior supports settles. Settlement of one interior support creates a negative moment in the bridge girder at the location of the other interior support. The negative moment increases caused by interior support settlement are given in Table 4.3.4. The negative moment increases experienced by each type of bridge due to 1 inch of interior support settlement, along with the effects of 1 inch of exterior support settlement, are displayed in Figure 4.3.3. Positive moment increases were investigated at the mid-point of both the first and center spans. Moment increases due to interior support settlement were evaluated at the location of the maximum positive load moment, which was generally at or near mid-span. Table 4.3.5 contains the moment increase results for the first and center spans. Positive moment increases in the two spans due to 1 inch of settlement are compared in Figure 4.3.4 for each bridge type. In addition to the positive moment increases in each span, the positive moment increase occurring at the settlement location was investigated for each bridge. The increase in positive moment at the settled interior support as compared to the maximum positive moment experienced by each bridge is detailed in Table 4.3.6. For each bridge type, Figure 4.3.5 shows the increase in positive moment at the location of settled interior support due to 1 inch of differential settlement.

Table 4.3.3: Negative Moment Increases in In-Service Bridge Girders Due to Exterior Support Settlement

Girder Type	**Span 1 (ft)**	**Span 2 (ft)**	**Average Span (ft)**	**Span Ratio**	***Percent Moment Increase*** **Settlement**		
					1"	**2"**	**3"**
Rolled Steel	33.23	30.02	31.6	0.9	37.2	74.5	111.7
	34.67	35.375	35.0	1.0	22.6	45.1	67.7
	54.14	67.26	60.7	1.2	10.0	19.9	29.9
	34	102	68.0	3.0	14.2	28.4	42.6
	73	85	79.0	1.2	3.4	6.9	10.3
Steel Plate	52.49	68.9	60.7	1.3	10.6	21.2	31.8
	74.25	99	86.6	1.3	4.3	8.6	12.9
	115.4	91.24	103.3	0.8	2.7	5.4	8.0
	89.92	117.52	103.7	1.3	2.1	4.2	6.4
	67	155	111.0	2.3	4.5	9.0	13.5
	125	240	182.5	1.9	1.6	3.1	4.7
	188.98	202.43	195.7	1.1	2.3	4.5	6.8
P/S Concrete	26.75	41	33.9	1.5	299.1	598.1	897.2
	50.25	56	53.1	1.1	98.4	196.7	295.1
	74.3	96.78	85.5	1.3	28.9	57.7	86.6

Table 4.3.4: Negative Moment Increases in In-Service Bridge Girders Due to Interior Support Settlement

Girder Type	**Span 1 (ft)**	**Span 2 (ft)**	**Average Span (ft)**	**Span Ratio**	***Percent Moment Increase*** **Settlement**		
					1"	**2"**	**3"**
Rolled Steel	33.23	30.02	31.6	0.9	56.6	113.1	169.7
	34.67	35.375	35.0	1.0	32.7	65.4	98.1
	54.14	67.26	60.7	1.2	11.9	23.7	35.6
	34	102	68.0	3.0	12.4	24.8	37.1
	73	85	79.0	1.2	4.4	8.8	13.2
Steel Plate	52.49	68.9	60.7	1.3	13.4	26.8	40.2
	74.25	99	86.6	1.3	5.1	10.1	15.2
	115.4	91.24	103.3	0.8	6.8	13.5	20.3
	89.92	117.52	103.7	1.3	2.6	5.3	7.9
	67	155	111.0	2.3	4.2	8.4	12.5
	125	240	182.5	1.9	1.6	3.1	4.7
	188.98	202.43	195.7	1.1	2.0	4.0	6.0
P/S Concrete	26.75	41	33.9	1.5	346.5	693.0	1039.5
	50.25	56	53.1	1.1	138.5	276.9	415.4
	74.3	96.78	85.5	1.3	36.7	73.5	110.2

4.3.3: Tolerance of Two-Span Continuous Bridges to Settlement

Differential settlement and angular distortion values were used to examine the tolerance of the in-service bridges to vertical substructure movement. Moment capacities at critical locations along each bridge girder were determined and compared to the moments caused by differential settlement. Initially, reserve moment capacities were calculated by subtracting the Strength I limit state loads from the nominal flexural resistance. For two-span continuous bridges, settlement of the exterior support creates additional negative moment in the girder at the center support. Tolerable exterior

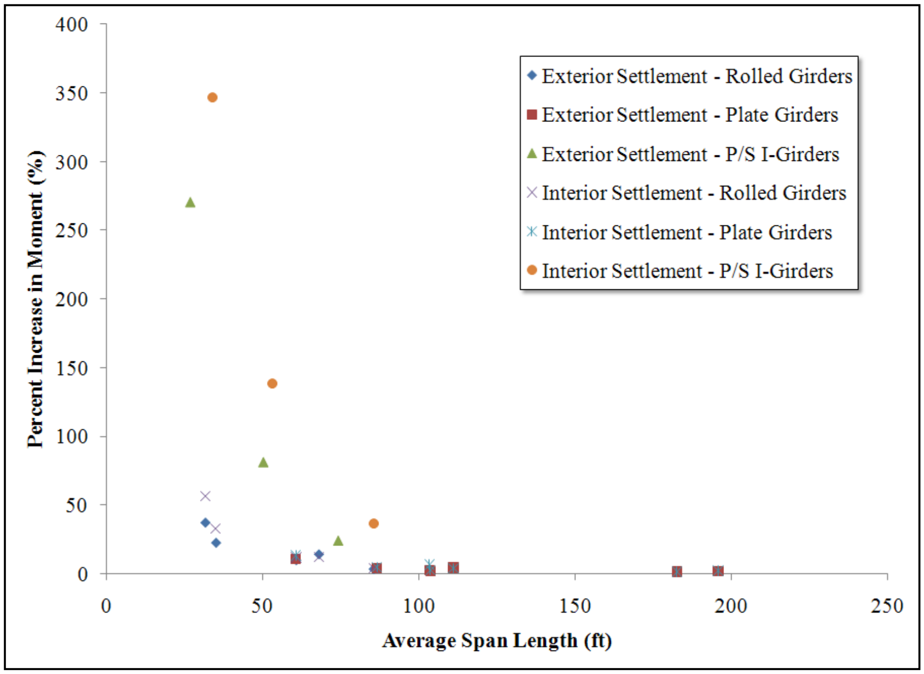


Figure 4.3.3: Negative Moment Increases in In-Service Bridge Girders

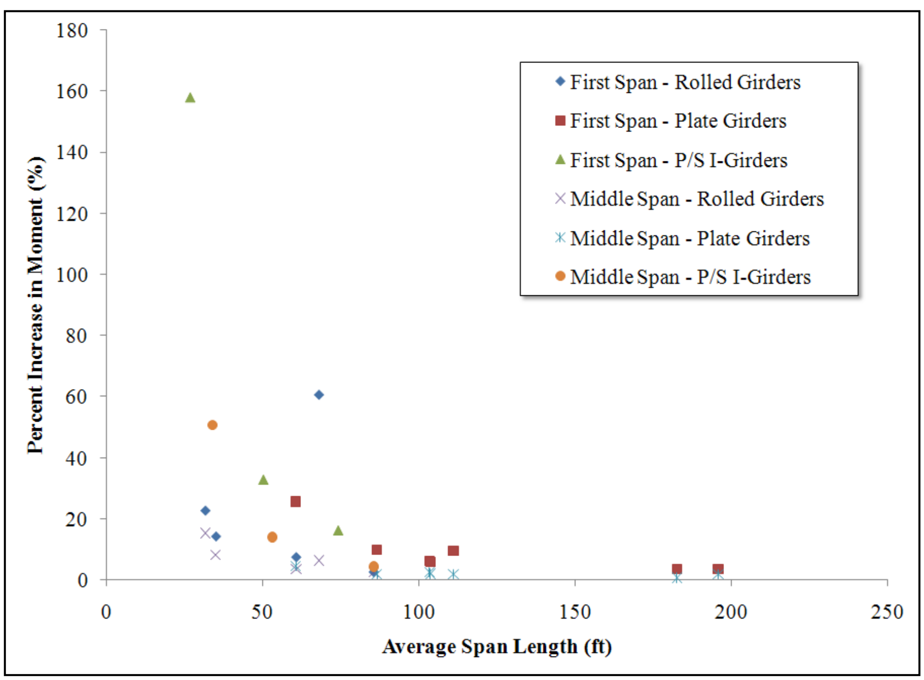


Figure 4.3.4: Positive Moment Increases in Spans of In-Service Bridge Girders

Table 4.3.5: Positive Moment Increases in First and Center Spans of In-Service Bridge Girders

Girder Type	**Span 1 (ft)**	**Span 2 (ft)**	**Average Span (ft)**	**Span Ratio**	***Percent Moment Increase at Mid-Point of First Span*** Settlement 1"	2"	3"	***Percent Moment Increase at Mid-Point of Second Span*** Settlement 1"	2"	3"
Rolled Steel	33.23	30.02	31.6	0.9	22.7	45.4	68.1	15.3	30.7	46.0
	34.67	35.375	35.0	1.0	14.3	28.6	42.9	8.2	16.4	24.6
	54.14	67.26	60.7	1.2	7.5	15.0	22.6	3.6	7.1	10.7
	34	102	68.0	3.0	60.5	121.1	181.6	6.3	12.6	19.0
	73	85	79.0	1.2	2.7	5.4	8.0	2.6	5.2	7.8
Steel Plate	52.49	68.9	60.7	1.3	10.8	21.7	32.5	4.6	9.3	13.9
	74.25	99	86.6	1.3	4.6	9.2	13.8	1.9	3.9	5.8
	115.4	91.24	103.3	0.8	2.9	5.7	8.6	2.6	5.1	7.7
	89.92	117.52	103.7	1.3	2.5	4.9	7.4	2.0	4.0	6.0
	67	155	111.0	2.3	15.9	31.8	47.8	2.0	3.9	5.9
	125	240	182.5	1.9	3.6	7.2	10.8	0.7	1.5	2.2
	188.98	202.43	195.7	1.1	0.7	1.4	2.1	1.5	3.0	4.5
P/S Concrete	26.75	41	33.9	1.5	157.9	315.8	473.7	50.8	101.6	152.4
	50.25	56	53.1	1.1	32.9	65.8	98.6	14.0	28.0	42.0
	74.3	96.78	85.5	1.3	16.3	32.6	48.8	4.4	8.8	13.2

Table 4.3.6: Positive Moment Increases in In-Service Bridge Girders at Settled Support

Girder Type	**Span 1 (ft)**	**Span 2 (ft)**	**Average Span (ft)**	**Span Ratio**	***Percent Moment Increase*** **Settlement**		
					1"	**2"**	**3"**
Rolled Steel	33.23	30.02	31.6	0.9	56.1	112.2	168.4
	34.67	35.375	35.0	1.0	35.7	71.5	107.2
	54.14	67.26	60.7	1.2	17.9	35.8	53.7
	34	102	68.0	3.0	31.1	62.3	93.4
	73	85	79.0	1.2	6.7	13.4	20.1
Steel Plate	52.49	68.9	60.7	1.3	25.6	51.2	76.8
	74.25	99	86.6	1.3	9.9	19.8	29.7
	115.4	91.24	103.3	0.8	6.1	12.1	18.2
	89.92	117.52	103.7	1.3	5.8	11.6	17.4
	67	155	111.0	2.3	9.5	18.9	28.4
	125	240	182.5	1.9	3.6	7.2	10.8
	188.98	202.43	195.7	1.1	3.1	6.3	9.4
P/S Concrete	26.75	41	33.9	1.5	270.3	540.5	810.8
	50.25	56	53.1	1.1	81.4	162.7	244.1
	74.3	96.78	85.5	1.3	24.4	48.7	73.1

support settlements were determined and are provided in Table 4.3.7. Center support settlement causes increased positive moments over the entire length of the girder. The most critical locations, however, are the point of maximum positive load moment (generally near mid-span) and at the settled support. Allowable settlement limits based on positive moment increases in two-span continuous bridges are given in Table 4.3.8.

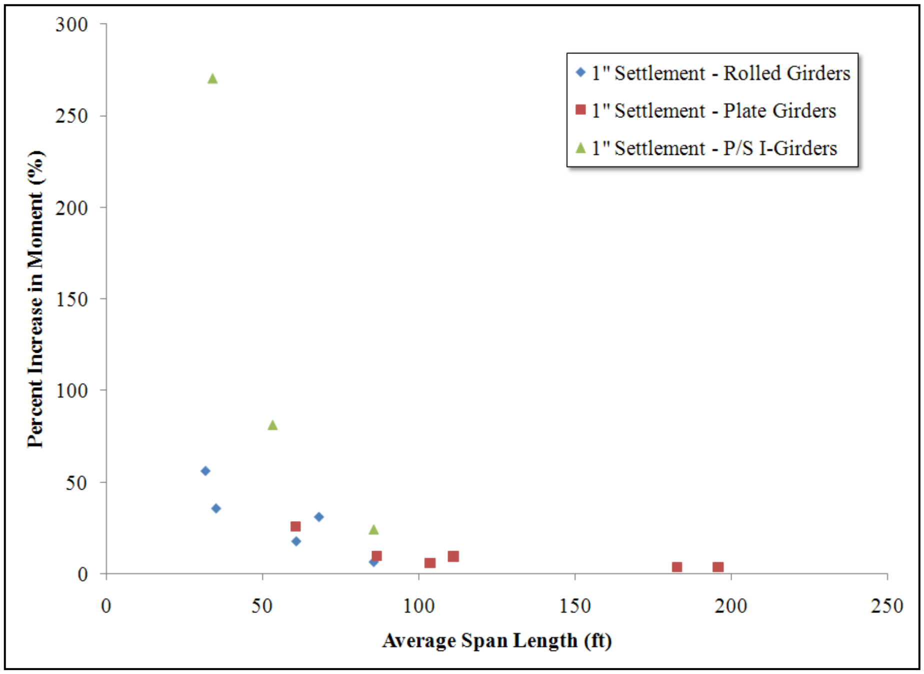


Figure 4.3.5: Positive Moment Increases in In-Service Bridge Girders at Settled Support

Table 4.3.7: Tolerable Exterior Support Settlements for In-Service Bridge Girders Based on Factored Moment Capacity

Girder Type	Span (ft)	Allowable Differential Settlement (in)	Allowable Angular Distortion (in/in)
Rolled Steel	45	1.54	0.0028
	64.75	6.49	0.0084
	93.5	8.36	0.0075
Steel Plate	67.26	1.78	0.0022
	84.32	8.28	0.0082
	108.27	-2.42	-0.0019
P/S Concrete	37.25	1.80	0.0040
	82.41	1.09	0.0011
	107.13	2.72	0.0021

Table 4.3.8: Tolerable Center Support Settlements for In-Service Bridge Girders Based on Factored Moment Capacity

Girder Type	Span (ft)	*Based on Mid-Span Moment*		*Based on Moment at Center Support*	
		Allowable Differential Settlement (in)	**Allowable Angular Distortion (in/in)**	**Allowable Differential Settlement (in)**	**Allowable Angular Distortion (in/in)**
Rolled Steel	45	14.85	0.0275	24.13	0.0447
	64.75	38.74	0.0499	30.66	0.0395
	93.5	44.50	0.0397	40.85	0.0364
Steel Plate	67.26	28.75	0.0356	19.65	0.0243
	84.32	18.68	0.0185	29.66	0.0293
	108.27	27.41	0.0211	77.40	0.0596
P/S Concrete	37.25	0.65	0.0014	2.15	0.0048
	82.41	2.84	0.0029	2.51	0.0025
	107.13	6.94	0.0054	7.16	0.0056

Tolerances likely to be more representative of what is observed in the field were also calculated. Rather than using factored moments to determine the reserve moment capacities of the bridge girders, unfactored load moments, along with flexural resistances, were used to calculate the reserve capacity of each girder. The unused capacities were then used to determine settlement limits for both the exterior and center supports. Tolerable limits for the exterior and center supports are given in Tables 4.3.9 and 4.3.10, respectively.

Table 4.3.9: Tolerable Exterior Support Settlements for In-Service Bridge Girders Based on Unfactored Moment Capacity

Girder Type	**Span (ft)**	**Allowable Differential Settlement (in)**	**Allowable Angular Distortion (in/in)**
Rolled Steel	45	8.15	0.0151
	64.75	15.34	0.0197
	93.5	21.82	0.0194
Steel Plate	67.26	8.49	0.0105
	84.32	18.54	0.0183
	108.27	34.81	0.0268
P/S Concrete	37.25	2.83	0.0063
	82.41	2.69	0.0027
	107.13	7.40	0.0058

Table 4.3.10: Tolerable Center Support Settlements for In-Service Bridge Girders Based on Unfactored Moment Capacity

Girder Type	**Span (ft)**	***Based on Mid-Span Moment***		***Based on Moment at Center Support***	
		Allowable Differential Settlement (in)	**Allowable Angular Distortion (in/in)**	**Allowable Differential Settlement (in)**	**Allowable Angular Distortion (in/in)**
Rolled Steel	45	23.74	0.0440	24.13	0.0447
	64.75	53.00	0.0682	30.66	0.0395
	93.5	44.50	0.0397	40.85	0.0364
Steel Plate	67.26	36.74	0.0455	19.65	0.0243
	84.32	28.09	0.0278	29.66	0.0293
	108.27	55.14	0.0424	77.40	0.0596
P/S Concrete	37.25	2.09	0.0047	2.15	0.0048
	82.41	5.00	0.0051	2.51	0.0025
	107.13	12.98	0.0101	7.16	0.0056

4.3.4: Tolerance of Three-Span Continuous Bridges to Settlement

In addition to determining the tolerances of two-span continuous bridges to differential settlement, the settlement tolerances of three-span bridges were investigated. Because three-span bridges are more complex than two-span bridges, additional moment increases needed to be studied. The tolerances of bridge girders to negative moment increases caused by both exterior and interior support settlements were determined and are presented in Table 4.3.11. Three positive moment increases cases were examined. Tolerances to positive moment increases near the mid-point of the first span and center span, along with tolerances to positive moment increases at the settled support, were computed. Allowable interior support settlement limits for each bridge based on positive moment increases are given in Table 4.3.12. The tolerances contained in Tables 4.3.11 and 4.3.12 were determined assuming that factored loads limited the reserve capacity of the bridge girders.

Tolerable differential settlement limits for each in-service bridge were also calculated using reserve moment capacities determined by utilizing the unfactored load moments. The limits resulting from allowable negative moment increases are provided in Table 4.3.13 and the limits determined as a result of controlling positive moment increases at the critical locations are given in Table 4.3.14

Table 4.3.11: Tolerable Support Settlements Limiting Negative Moments in In-Service Bridge Girders Based on Factored Moment Capacity

Girder Type	Span 1 (ft)	Span 2 (ft)	***Exterior Support Settlement***		***Interior Support Settlement***	
			Allowable Differential Settlement (in)	**Allowable Angular Distortion (in/in)**	**Allowable Differential Settlement (in)**	**Allowable Angular Distortion (in/in)**
Rolled Steel	33.23	30.02	4.03	0.0101	2.65	0.0067
	34	102	1.41	0.0035	1.62	0.0013
	34.67	35.375	4.27	0.0103	2.95	0.0070
	54.14	67.26	1.92	0.0030	1.61	0.0020
	73	85	-1.57	-0.0018	-1.23	-0.0012
Steel Plate	52.49	68.9	1.17	0.0019	0.92	0.0011
	67	155	6.22	0.0077	6.70	0.0036
	74.25	99	4.38	0.0049	3.71	0.0031
	89.92	117.52	7.02	0.0065	5.63	0.0040
	115.4	91.24	10.98	0.0079	1.72	0.0012
	125	240	37.08	0.0247	37.52	0.0130
	188.98	202.43	19.58	0.0086	24.10	0.0099
P/S Concrete	26.75	41	0.57	0.0018	0.74	0.0015
	50.25	56	2.45	0.0041	1.74	0.0026
	74.3	96.78	0.59	0.0007	0.46	0.0004

Table 4.3.12: Tolerable Interior Support Settlements Limiting Positive Moments in In-Service Bridge Girders Based on Factored Moment Capacity

Girder Type	Span 1 (ft)	Span 2 (ft)	***Based on Moment at Mid-Point of First Span***		***Based on Moment at Mid-Point of Second Span***		***Based on Moment at Settled Support***	
			Allowable Differential Settlement (in)	**Allowable Angular Distortion (in/in)**	**Allowable Differential Settlement (in)**	**Allowable Angular Distortion (in/in)**	**Allowable Differential Settlement (in)**	**Allowable Angular Distortion (in/in)**
Rolled Steel	33.23	30.02	11.21	0.0281	31.08	0.0780	7.34	0.0184
	34	102	43.93	0.0359	23.65	0.0193	11.27	0.0092
	34.67	35.375	14.62	0.0344	30.58	0.0720	9.84	0.0232
	54.14	67.26	19.37	0.0240	34.74	0.0430	19.21	0.0238
	73	85	30.38	0.0298	42.27	0.0414	43.30	0.0425
Steel Plate	52.49	68.9	24.13	0.0292	50.79	0.0614	17.18	0.0208
	67	155	59.36	0.0319	27.22	0.0146	29.29	0.0157
	74.25	99	35.17	0.0296	59.40	0.0500	29.34	0.0247
	89.92	117.52	39.85	0.0283	41.48	0.0294	54.78	0.0388
	115.4	91.24	28.30	0.0204	139.93	0.1010	37.13	0.0268
	125	240	107.37	0.0373	137.31	0.0477	72.28	0.0251
	188.98	202.43	64.60	0.0266	231.45	0.0953	93.31	0.0384
P/S Concrete	26.75	41	0.21	0.0004	1.11	0.0023	0.61	0.0012
	50.25	56	1.76	0.0026	6.10	0.0091	1.92	0.0029
	74.3	96.78	1.69	0.0015	17.24	0.0148	0.85	0.0007

Table 4.3.13: Tolerable Support Settlements Limiting Negative Moments in In-Service Bridge Girders Based on Unfactored Moment Capacity

Girder Type	Span 1 (ft)	Span 2 (ft)	***Exterior Support Settlement***		***Interior Support Settlement***	
			Allowable Differential Settlement (in)	**Allowable Angular Distortion (in/in)**	**Allowable Differential Settlement (in)**	**Allowable Angular Distortion (in/in)**
Rolled Steel	33.23	30.02	5.55	0.0139	3.66	0.0092
	34	102	4.98	0.0122	5.71	0.0047
	34.67	35.375	6.81	0.0164	4.70	0.0111
	54.14	67.26	6.80	0.0105	5.70	0.0071
	73	85	13.02	0.0149	10.23	0.0100
Steel Plate	52.49	68.9	6.13	0.0097	4.85	0.0059
	67	155	16.19	0.0201	17.44	0.0094
	74.25	99	15.65	0.0176	13.27	0.0112
	89.92	117.52	29.66	0.0275	23.80	0.0169
	115.4	91.24	27.94	0.0202	8.43	0.0061
	125	240	63.55	0.0424	64.29	0.0223
	188.98	202.43	39.64	0.0175	46.81	0.0193
P/S Concrete	26.75	41	0.80	0.0025	0.93	0.0019
	50.25	56	3.11	0.0052	2.21	0.0033
	74.3	96.78	3.18	0.0036	2.50	0.0022

Table 4.3.14: Tolerable Interior Support Settlements Limiting Positive Moments in In-Service Bridge Girders Based on Unfactored Moment Capacity

Girder Type	Span 1 (ft)	Span 2 (ft)	***Based on Moment at Mid-Point of First Span***		***Based on Moment at Mid-Point of Second Span***		***Based on Moment at Settled Support***	
			Allowable Differential Settlement (in)	**Allowable Angular Distortion (in/in)**	**Allowable Differential Settlement (in)**	**Allowable Angular Distortion (in/in)**	**Allowable Differential Settlement (in)**	**Allowable Angular Distortion (in/in)**
Rolled Steel	73	85	52.75	0.0517	66.33	0.0650	43.30	0.0425
	54.14	67.26	27.40	0.0339	51.96	0.0644	19.21	0.0238
	34	102	45.65	0.0373	31.82	0.0260	11.27	0.0092
	33.23	30.02	14.00	0.0351	35.77	0.0897	7.34	0.0184
	34.67	35.375	19.14	0.0451	39.05	0.0920	9.84	0.0232
Steel Plate	89.92	117.52	64.00	0.0454	71.00	0.0503	54.78	0.0388
	115.4	91.24	46.06	0.0333	168.83	0.1219	37.13	0.0268
	67	155	64.62	0.0347	53.17	0.0286	29.29	0.0157
	125	240	126.22	0.0438	203.95	0.0708	72.28	0.0251
	188.98	202.43	130.05	0.0535	267.47	0.1101	93.31	0.0384
	52.49	68.9	29.85	0.0361	63.91	0.0773	17.18	0.0208
	74.25	99	48.28	0.0406	90.17	0.0759	29.34	0.0247
P/S Concrete	74.3	96.78	2.72	0.0023	27.34	0.0235	0.85	0.0007
	50.25	56	3.32	0.0049	9.72	0.0145	1.92	0.0029
	26.75	41	0.70	0.0014	2.15	0.0044	0.61	0.0012

4.4: Relationship between Stiffness and Settlement-Induced Stresses

Figure 4.4.1 shows the relationship involving the moment of inertia and the square of the span length of a given composite steel girder in a two-span continuous bridge and the theoretical negative moment experienced by that member due to settlement of the exterior support. Likewise, theoretical maximum positive moments experienced by two-span continuous steel bridge girders with various I/ℓ^2 values are presented in Figure 4.4.2. For the sake of simplicity, the moment of inertia of the short-term composite sections was used to investigate the stiffness-settlement relationship. If the moment of inertia of the long-term composite section was used instead, the slope of the trendline would change, but the moment caused by differential settlement obtained from the graph would remain the same.

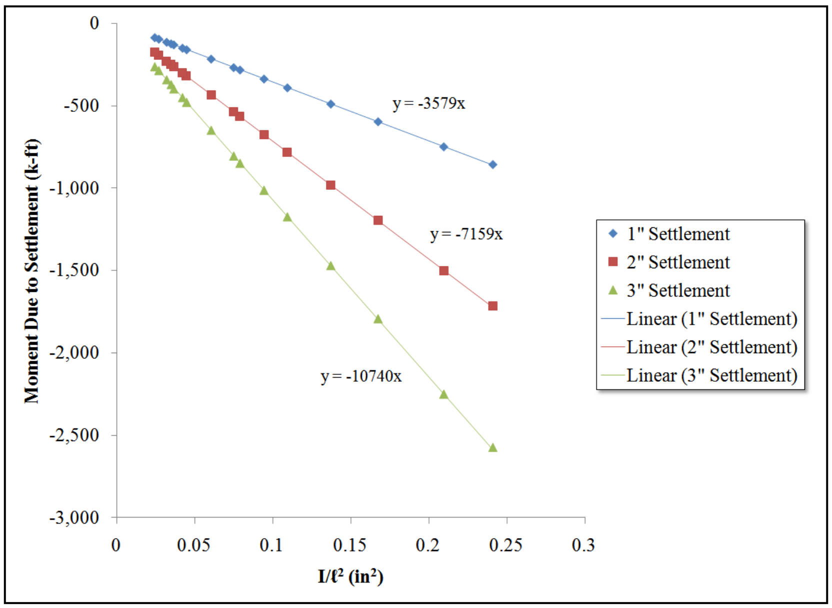


Figure 4.4.1: Negative Moments Caused by Differential Settlement of Two-Span Continuous Bridges

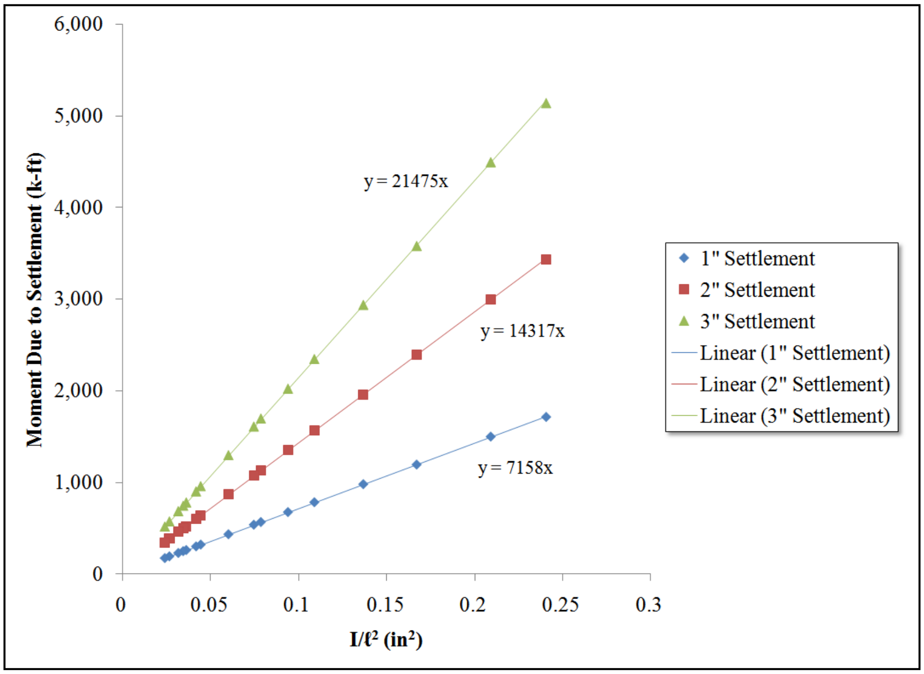


Figure 4.4.2: Positive Moments Caused by Differential Settlement of Two-Span Continuous Bridges

4.4.1: Number of Spans

In addition to analyzing various two-span continuous bridges, continuous bridges of three, four, five, and six spans were also analyzed. The theoretical negative moments caused by differential settlement of three-span bridge supports are summarized in Figure 4.4.3. Negative moments due to both exterior and interior support settlements were investigated. Figure 4.4.4 shows the theoretical positive moment that can be expected due to settlement of an interior support of a three-span continuous bridge. Similar results for four-, five-, and six-span bridges can be found in Appendix B.

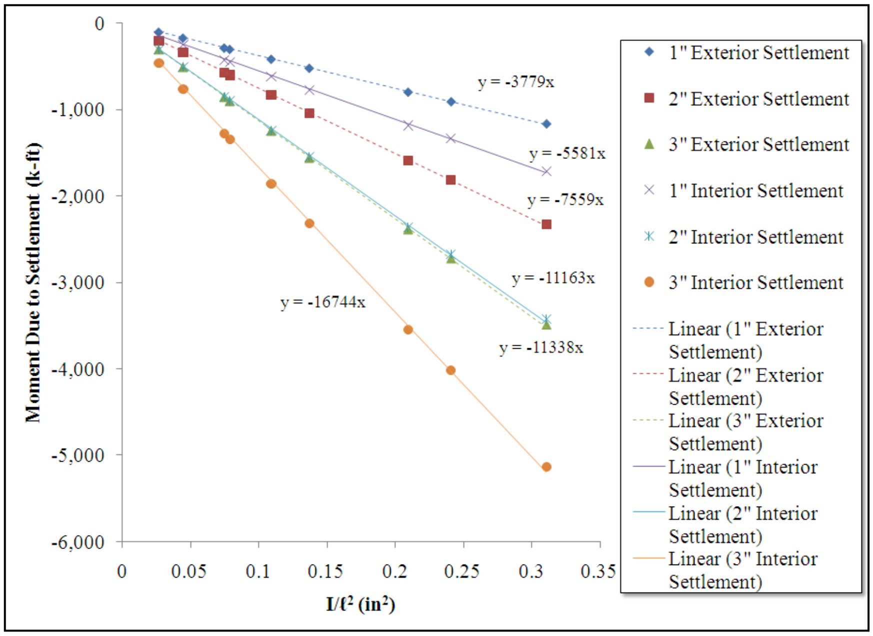


Figure 4.4.3: Negative Moments Caused by Differential Settlement of Three-Span Continuous Bridges

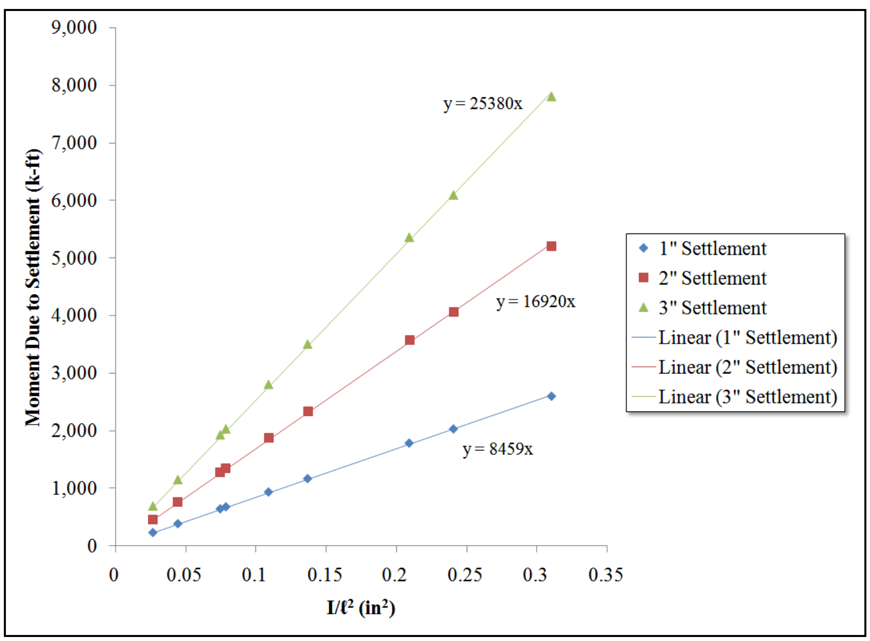


Figure 4.4.4: Positive Moments Caused by Differential Settlement of Three-Span Continuous Bridges

The trends found by plotting the resulting data were used to examine the relationship between the number of bridge spans (N_s) and the magnitude of the experienced moment. Differential settlement coefficient (C_s) values were calculated using the slopes of the trendlines obtained from the various settlement moment versus I/ℓ^2 plots as discussed in Section 3.5. Plotting the C_s values displays the relationship between the theoretical moment increase and the number of bridge spans. The C_s values used to determine the negative moments caused by settlement of the exterior and inner most supports are given in Table 4.4.1 and displayed in Figure 4.4.5. C_s values used to determine the maximum positive moment caused by settlement of the inner most support are provided in Table 4.4.2 and displayed in Figure 4.4.6.

Table 4.4.1: C_s Values Calculated from Negative Settlement Moment Plots

Number of Spans	C_s Values	
	From Negative Moment Due to Exterior Support Settlement	From Maximum Negative Moment
2	-1.48	-1.48
3	-1.56	-2.31
4	-1.57	-2.46
5	-1.57	-2.62
6	-1.56	-2.59

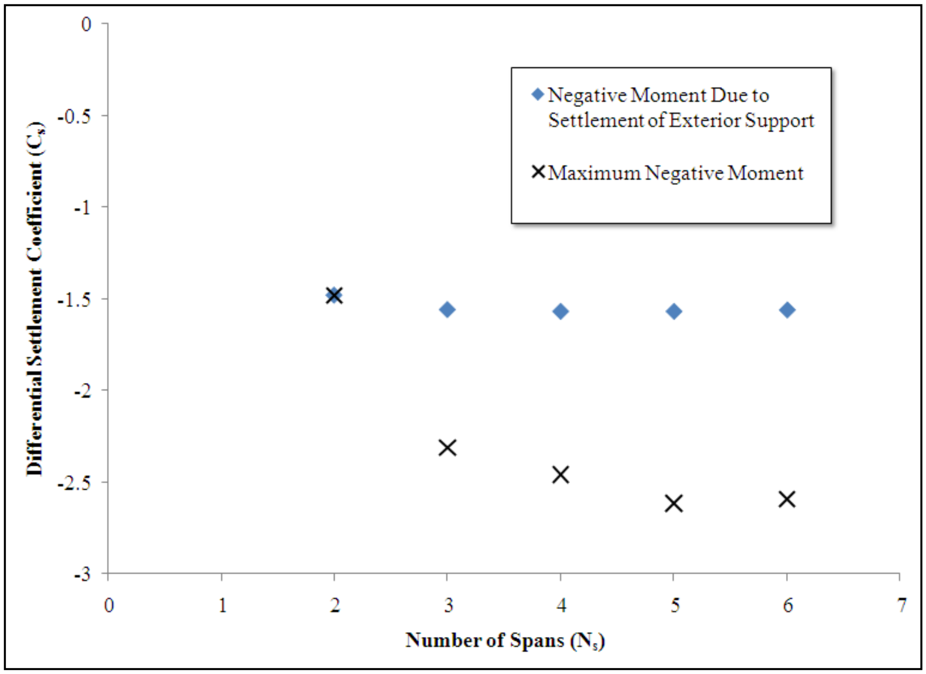


Figure 4.4.5: Relationship between C_s and N_s for Negative Settlement Moments

Table 4.4.2: C_s Values Calculated from Positive Settlement Moment Plots

Number of Spans	**C_s Values**
	From Maximum Positive Moment
2	2.96
3	3.50
4	4.12
5	4.16
6	4.16

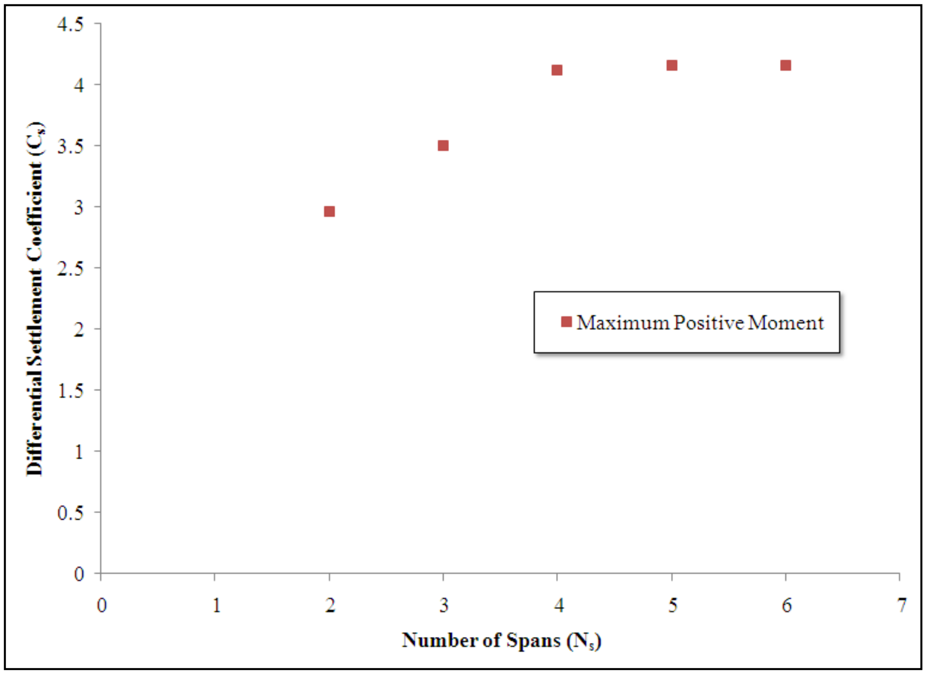


Figure 4.4.6: Relationship between C_s and N_s for Positive Settlement Moments

4.5: Bridge Design Considering Settlement

In order to determine the impact of accounting for potential differential settlements during the bridge design process, the economical two-span continuous bridge girders designed using the LRFD Bridge Design Specifications were redesigned to account for the moment caused by 3 inches of differential settlement. Both exterior and center support settlements were examined. Negative moment increases caused by settlement of the exterior support were found to control the design in all cases. Table 4.5.1 compares the rolled steel sections selected for each bridge with and without accounting for differential settlement.

Table 4.5.1: Girder Design Comparison

Span Length (ft)	Girder Selected without Considering Differential Settlement	Girder Selected when Considering Differential Settlement
30	W21x57	W30x90
40	W24x84	W30x90
50	W30x90	W30x99
60	W30x99	W30x116

Chapter 5

DISCUSSION OF RESULTS

Investigation of the results of the various analyses presented in Chapter 4 allows for general trends regarding the response of bridges to differential vertical movements to be observed. The trends can then be used to determine tolerable movement limits and, eventually, create general design guidelines. The ultimate goal of the analyses is to translate tolerable bridge movement values into data that can be used to incorporate the potential effects of differential settlements into the bridge design process by modifying the LRFD limit states.

5.1: Reproduction of Past Results

The results of the analyses presented in Figures 4.1.1 and 4.1.2 differ from the results obtained by Moulton et al., which were given as Figures 2.4.1 and 2.4.2. The negative stress increases determined by Moulton et al. are less than the increases displayed in Figure 4.1.1. For a 1-inch exterior support settlement of a 30-foot span bridge, Moulton et al. calculated a stress increase of approximately 150 percent whereas an increase of 188 percent was computed in an effort to reproduce the past results. Several issues may have led to the discrepancy, but two factors, in particular, likely led to the difference. First, without matching the exact loading conditions used by Moulton et al. (live load discrepancy and use of impact factor), an increase in stress as a percentage of the stress caused by the applied loads is impossible to match exactly. Additionally, the model used by Moulton et al. to determine settlement stresses is unclear. This study utilized a girder and composite concrete deck over the entire length of the girder to characterize stiffness. Moulton et al. may have modeled the beam as non-composite in the negative moment region.

When comparing the positive stress increases near the mid-point (at the location of the maximum positive stress due to the applied loads) of the first span, there is a clear difference in results. Moulton et al. found that an interior support settlements of 1 inch applied to a two-span continuous bridge with 30-foot spans would cause a stress increase of approximately 150 percent. The corresponding moment increase calculated as part of this study is 224 percent. Again, several issues may have led to the differences between the results obtained by Moulton et al. and the results calculated in an effort to reproduce the past results. The discrepancy in applied loads and uncertainty in analytical models likely explain some of the difference. Also, the exact location of the measured stress increases calculated by Moulton et al. is not known. The current study calculated load moments at tenth points along the span and determined the stress increase at the location of maximum load, which was often 0.4ℓ, not mid-span. In addition, the results reported by Moulton et al. must be questioned. As the magnitude of the applied settlement increases, the moment caused by settlement should increase linearly for a given member. Therefore, the percent increase in stress should also increase linearly. For the 30-foot span bridge, Moulton et al. claimed that 1 inch of settlement caused a 150 percent increase in stress, 2 inches of settlement caused a 400 percent increase in stress, and 3 inches of settlement caused a 700 percent increase in stress. These increases are clearly not linear and suggest that the data obtained by Moulton et al. may not be accurate.

In addition, to the discrepancies with the stress increases at the mid-point of the first span, a lack of positive stress increase information in the work of Moulton et al. was discovered. Moulton et al. claimed that the critical positive stress increases would occur near the mid-point of the span. The maximum positive moment due to settlement of the interior support, however, was found to occur over the settled support. The moment increases over the center support of a two-span continuous bridge due to settlement of the center support were displayed in Figure 4.1.3. The increases in Figure 4.1.3 were calculated by comparing the moment at the center support caused by settlement to the maximum positive moment caused by the applied loads (located near the mid-point of the span), not the moment at the support caused by the applied loads. When comparing the information in Figure 4.1.3 to that contained in Figure 4.1.2, it is obvious that the maximum increase in positive stress will occur over the center support. The fact that the moment increases at the center support are roughly double the moment increases at the location of maximum positive moment caused by loading should be expected since the moment caused by settlement increases linearly over the length of the girder. Although the largest positive moment caused by differential settlement occurs in the girder at the settled center support, that is not necessarily the critical positive stress location. Under normal loading conditions, the moment in a continuous girder over a support will be negative. When a large enough settlement occurs at the support, the moment will become positive. Since continuous girders do not carry positive moment over the support, the entire positive moment capacity of the girder can resist the large moments caused by differential settlement. Both the mid-point of a span and the continuous end of that span need to be considered when determining the tolerance of a bridge to positive moments caused by differential settlement. Either location could be critical.

Though determining the stress increases caused by differential settlement provides a convenient method of examining the general effects of settlement on various bridges, stress increases represented as percentages have little value in determining the tolerance of a bridge to settlement. Instead, the actual increases in stresses experienced by a given girder must be compared with the ability of that girder to accommodate the stresses. Calculating the magnitudes of differential settlement that can be tolerated by a variety of bridges allows for a more realistic examination of tolerable substructure movements.

5.1.1: Tolerance of W36 Girders to Settlement

Since many of the W36 girders were found to be overdesigned, the reserve moment capacity can be expected to alleviate some the stresses caused by settlement. Despite the reserve capacity, the tolerable settlement values given in Table 4.1.1 suggest that an exterior support settlement of 1 inch would cause intolerable negative stresses in all bridges except the 40-foot span bridge. More than 1.5 inches of center settlement was tolerable for all bridges except the 30-foot span bridge, which was limited by the positive moment increase at the center support. All other bridges were limited by the increase in moment near mid-span.

The allowable angular distortions computed for the W36 bridge girders did not initially agree with the tolerable angular distortion limit proposed by Moulton et al. For 30-, 40-, 50-, and 60-foot span bridges, the 0.004-inch per inch distortion limit suggests that differential settlements of 1.44, 1.92, 2.40, and 2.88 inches, respectively, would be tolerable. All of the tolerable angular distortion values calculated for the W36 girders were less than limit suggested by Moulton et al, which was created based on field survey data.

Since it is widely accepted that actual bridges behave differently than analytically modeled bridges, modifications to the tolerable differential settlement limit were deemed necessary in order to produce results consistent with the proposed limit. Modifying the reserve moment capacity calculations to allow girder stresses to reach the yield stress before movements were considered intolerable led to results, which were given in Tables 4.1.3 and 4.1.4, that are in agreement with the angular distortion limit suggested by Moulton et al. For exterior support settlement, angular distortions up to 0.006 inches per inch were found to be tolerable. The angular distortion limit for center support settlement was found to be just over the tolerable limit suggested by Moulton et al. for the 30-foot span bridge. Tolerable center support settlements were controlled by positive stresses in the girders at the center support for all but the 60-foot span bridge, which was controlled by mid-span stresses. Allowable angular distortion limits appear to increase as span length increases, particularly for the center support settlement limit.

5.1.2: Tolerance of Grade 36 Alternate Girders to Settlement

Figures 4.1.4, 4.1.5, and 4.1.6 show that the stress increases presented in Figures 4.1.1, 4.1.2, and 4.1.3 are over-exaggerated and would not typically occur. Because the W36 girders were larger than required in most cases, the moments caused by settlement were large in comparison to the moments caused by the design loads. Therefore, the large stress increases due to settlement of a support in a W36 bridge are misleading. In order to properly investigate the increase in stress due to support settlement, properly designed girders should be utilized. Figures 4.1.4, 4.1.5, and 4.1.6 show that the increases in settlement-induced moments are significantly less in girders that are selected based on efficiency than in girders chosen arbitrarily.

Selecting girders that have a moment capacity only slightly larger than that required to carry the design loads reduces the size of the section and, therefore, the cost of the bridge. Reducing the size of a girder also reduces the stiffness of that girder, which leads to smaller moment increases due to settlement. Although the moment increases caused by settlement will be smaller in magnitude for smaller sections, the reserve moment capacity in each girder will also be less.

Table 4.1.5 suggests that a 1-inch settlement of an exterior support would cause stress increases in all of the alternate girders that would exceed the allowable design stress for each girder, which is inconsistent with what would be expected based on previous empirical studies. Tolerable center support settlements were determined from limiting mid-span stresses. The results given in Table 4.1.6 are also well below the 0.004-inch per inch angular distortion limit. However, if stresses in the girders are limited to the yield stress of the member rather than the allowable limit specified in the Standard Specifications, the tolerances of the bridges to settlement fall in line with the empirical limit proposed by Moulton et al. Comparing Tables 4.1.7 and 4.1.8 suggests that the exterior support settlements are more critical than the center support settlements. Again, tolerable angular distortions were found to increase as span length increased.

5.1.3: Tolerance of Grade 50 Alternate Girders to Settlement

Constructing a bridge with girders fabricated from higher strength steel reduces the size of the girders required to carry the design load. The shallower, lighter Grade 50 alternate girders given in Section 4.1.2 are less stiff than the Grade 36 alternate girders and significantly more flexible than the W36 girders. Figures 4.1.4, 4.1.5, and 4.1.6 show that, for all of the cases investigated, the Grade 50 alternate sections experienced smaller moment increases due to settlement than the Grade 36 alternate sections, which is expected based on the stiffness differences.

Despite the smaller increase in settlement stresses, Table 4.1.9 suggests the allowable design stresses would be exceeded for all bridges by settling the exterior support by less than 0.25 inches, which is far below the allowable settlements predicted by the 0.004-inch per inch angular distortion limit. With allowable angular distortion values ranging from 0.001 to 0.003 inches per inch, tolerable center support settlements determined from the critical stresses near mid-span were also below the prescribed limit. The results of increasing the allowable design stress to the yield stress for all of the bridge girders were summarized in Tables 4.1.11 and 4.1.12. Increasing the allowable stresses to 50 kips per square inch drastically increased the tolerable settlement limits of both the exterior and center supports to well above the limit proposed by Moulton et al. Settlement of exterior supports was again found to be more critical than the same magnitude of settlement occurring at the center support for all bridges. Increasing span length appears to increase the allowable angular distortion caused by differential settlement of either support.

The large increase in tolerable angular distortion observed by increasing the allowable stress to the yield stress for the Grade 50 alternate girders should be expected. Since allowable stress design limits internal stresses to a percentage of the yield stress, the unused reserve capacity of Grade 50 steel girders is larger than the reserve capacity of Grade 36 girders. Increasing allowable stresses from the design stress to the yield stress for Grade 50 steel members results in a 22.5-kip per square inch increase in allowable stresses, whereas increasing the allowable stress from the design stress to the yield stress for Grade 36 steel members only results in a 16-kip per square inch increase in allowable stresses. Therefore, bridge girders fabricated from Grade 50 steel should be more tolerant to differential settlements than Grade 36 steel girders.

5.2: Updated Results

Any potential difference in the tolerances of bridges designed using the Standard Specification and LRFD Specifications is difficult to determine based on the moment increase plots presented in Sections 4.1 and 4.2. Although the responses of the two sets of bridges to differential settlement are significantly different, the reasons for those differences must be examined in order to determine whether any significant difference truly exists. Design requirements and assumptions differ between the two specifications and must be taken into account when analyzing the results.

Figure 4.2.1 shows that a 1-inch settlement of the exterior support of a two-span continuous bridge with W36 composite girders designed using the LRFD Specifications would cause the negative moment over the center support to increase by 113 percent when 30-foot spans are considered. The same situation was found to cause a 188 percent increase in moment for the 30-foot span W36 composite girder bridge designed using the Standard Specifications. The maximum increase in positive moment in the girder of a 30-foot span bridge caused by 1 inch of settlement was 250 percent for the bridge designed using LRFD and 488 percent for the bridge designed using the Standard Specifications. Likewise, theoretical negative and positive moment increases due to settlement were found to be larger for the bridges designed using the Standard Specifications for bridges of all four span lengths.

Since the moments caused by settlement are the same for W36 girders of a given span length regardless of the design code used to design the bridge, differences in the applied loading cause the discrepancies with the percent increases in moment. The

LRFD Specifications require the live load applied to a bridge to consist of a lane load in addition to the vehicular load. Additionally, special live loadings are permitted to produce increased negative moments at the critical point above supports. The live load specified in the Standard Specifications consists of either a lane load or a truck load. Based on the requirements of the design codes, the bridges designed using the LRFD Specifications are expected to carry heavier loads than bridges designed using the Standard Specifications. Because the specified load is larger for LRFD bridges, the moment caused by settlement will be smaller in comparison to the applied loads than for bridges designed using the Standard Specifications. Although the percent increase in moment is smaller, bridges designed using the LRFD Specifications are not necessarily more tolerant to differential settlement.

In addition to the differences in applied loads, the LRFD Specifications allowed for the moment capacity of the steel girders to exceed the capacity that would be recognized using the Standard Specifications. Appendix A6 in the LRFD Specifications was used for the flexural design of the steel girders, which allows for some girder plastification to occur. Girders designed using LRFD, therefore, generally have larger moment capacities than equivalent girders designed using the Standard Specifications. The larger moment capacity, however, does not ensure that the girders will be more tolerant to differential settlement.

5.2.1: Tolerance of W36 Girders to Settlement

The W36 rolled steel girders selected for the LRFD design of the four bridges were greatly oversized. In fact, the reserve moment capacities of all four bridges was adequate to properly resist the moments induced 3 inches of settlement at either the exterior or center support in addition to the factored loads applied to each girder. Tables 4.2.1 and 4.2.2 provided the tolerable settlement limits for exterior and center supports of the W36 composite bridges, respectively. All allowable angular distortion values were found to be significantly larger than the 0.004-inch per inch limit proposed by Moulton et al.

Actual bridges in service would likely be capable of withstanding larger differential settlements than determined analytically. Since the loads actually applied to bridges are not factored, unfactored loads were used to calculate the reserve moment capacity of each bridge girder in an attempt to produce more realistic differential settlement limits. Comparing the unfactored moment capacities of the girders to the moments caused by differential settlement resulted in the tolerable settlement limits given in Tables 4.2.3 and 4.2.4. Positive moment increases at the center support were found to limit settlement of the center support. The increase in positive moment at the center support was the critical settlement case overall for the 30-foot span bridge. Settlement of the exterior support was critical for the remaining W36 bridges. Tolerable angular distortions were again found to increase as the span length of the bridges increased.

5.2.2: Tolerance of Alternate Girders to Settlement

The tolerances of the W36 girders to differential settlement do not reflect the typical tolerances that would be seen in properly designed bridges. Figures 4.2.4, 4.2.5, and 4.2.6 show that the efficiently designed girders would be expected to experience much smaller moment increases due to differential settlement than the W36 girders. The heavier, deeper W36 girders are more stiff and, therefore, more affected by differential settlement. Figures 4.2.4 and 4.2.6 show that the maximum negative and positive moment increases of W36 members are overstated and misleading, especially for the shorter span lengths. As would be expected, efficiently designed girders are more affected by settlement as the span length decreases; however, the moment increases caused by settlement are not as extreme as those found by analyzing oversized members.

Since the alternate girders were designed with efficiency in mind, very little reserve moment capacity is available. Even small settlements, therefore, would cause intolerable moment increases. Since the LRFD Specifications contain special loading conditions to produce maximum negative moment, negative moment resistance controlled the design of all four bridges. Settlement of the exterior support was, therefore, found to be more critical than settlement of the center support for the LRFD bridges. In fact, the girders for all four bridges were able to tolerate over 5.5 inches of settlement causing positive moment, but only the 50-foot span bridge could tolerate more than 1 inch of settlement causing negative moment. The tolerable limits for settlement of the exterior and center supports were given in Tables 4.2.5 and 4.2.6, respectively. Note that the larger exterior settlement tolerance of the 50-foot span bridge can be attributed to the larger reserve moment capacity of the selected section. Center support settlements for all but the 60-foot span bridge were limited by moment increases occurring at the center support.

Once again, realizing that bridges can usually tolerate larger magnitudes of differential settlement than suggested by typical analyses, unfactored loads were used to compute reserve moment capacities. The tolerable settlement limits determined using the increased reserve moment capacities were presented in Tables 4.2.7 and 4.2.8. Allowable exterior support settlements for the 30- and 40-foot span bridges remained below the tolerable limit proposed by Moulton et al. suggesting that either the bridge girders were designed unrealistically or that means of alleviating the additional settlement moments were not accounted for in the settlement analyses. Exterior support settlements are critical since large magnitudes of center support settlement were again determined to be tolerable.

Note that the 30-, 40-, and 50-foot span bridges were found to have the same allowable center support settlements regardless of the method used to determine the reserve moment capacity. Since the center support settlement was limited by positive moment increases at the center support for those bridges and the applied loads cause no positive moment at that location, the entire capacity of the bridge girders are available to alleviate settlement stresses in both cases. The limit for the 60-foot span bridge differed for each case because positive moments are present at mid-span and, therefore, affected the reserve moment capacity.

5.3: In-Service Bridge Analyses

While the analyses discussed in Sections 5.1 and 5.2 provide good theoretical insight into how differential vertical movements affect bridge superstructures, the results are not completely realistic. Typical bridge designs are generally not as simplistic as the bridges previously discussed. In order to gain an understanding of the accuracy of the simplistic models, real bridge designs need to be analyzed. The applicability of the trends and tolerances observed by analyzing the simplistic models to actual design procedures can be determined by comparing the results of the discussed in the previous sections to the results of the in-service bridge analyses.

5.3.1: Two-Span Continuous Bridges

The moment increases calculated for in-service two-span continuous bridges were generally less than the moment increases calculated for the simple bridges (for equivalent span lengths). The negative moment increases determined for the rolled steel girder bridges presented in Table 4.3.1 are slightly smaller than the increases computed for LRFD alternate bridges. The positive increases, however, were found to be significantly less for the in-service bridges than the simple bridges. Table 4.3.2 contains moment increases at both the mid-span and center support locations for the in-service bridges. The relative differences in results between the in-service bridges and the simplistic bridges may be attributed to the method of analysis. The sample bridges designed for use with this study were modeled as fully composite when subjected to differential support settlements. The model used by Virtis to compute the response of the in-service bridge models is unclear.

Figure 4.3.1 shows that the bridges supported by rolled steel girders experienced the smallest negative moment increases due to differential settlement. The plate girder bridges experienced similar, but slightly larger moment increases than the rolled girders. Finally, the prestressed concrete girders experienced very large moment increases as compared to the steel girders. These trends should be expected based on the typical properties of the respective girders. Rolled steel sections are generally shallow compared to other types of girders and, therefore, have smaller stiffness values. Plate girders are usually designed to carry load as efficiently as possible, which causes stiffness properties somewhat greater than those of rolled girders. Prestressed concrete girders have moment of inertia values significantly larger than either type of steel beam. Based on the stiffnesses of the different girder types, the results presented in Figure 4.3.1 should not be surprising.

The positive moment increases displayed in Figure 4.3.2 compliment trends discussed for the negative moment increases well. Again, rolled steel beams were found to have the smallest increase in moment due to settlement, steel plate girders experienced increases slightly larger than the rolled girders, and the prestressed concrete girders experienced significantly larger increases. Figure 4.3.2 also compares the positive moment increases at the location of the maximum positive load moment and at the settled center span. The moment increases experienced by the girders at mid-span were found to be roughly one-half of the increases caused at the support, which is expected.

5.3.2: Three-Span Continuous Bridges

In order to be certain that all possible critical moment cases were considered, moment increases at five different locations were investigated for three-span continuous bridges. Negative moment increases above the first interior support caused by settlement of the exterior support and above the second interior support caused by settlement of the first interior support were examined. Additionally, positive moment increases at the mid-point of the first and center spans, along with the moment increase at the settled support, were investigated.

The theoretical increases in negative moment for all three girder types due to exterior support settlement were given in Table 4.3.3 and increases due to interior support settlement were provided in Table 4.3.4. As would be expected, the effect of exterior support settlement appears to be related to the length of the first bridge span. In most cases, as the span length of the first span increases, the increase in negative moment due to a given settlement decreases. Likewise, the negative moment increases due to interior support settlement are closely related to the length of the interior span. Any exceptions to these trends likely occur in bridges with center spans that are relatively large as compared to the first span.

Data from the two tables is compared in Figure 4.3.3, which shows that, in general, settlement of the interior support causes a larger moment increase than an equal exterior support settlement. Some exceptions can be seen and are explained by looking closer at the data in the corresponding tables. Any bridge which was found to experience larger negative moment increases due to exterior support settlement has a long center span in comparison to the end spans. All of the bridges experiencing maximum negative moment increases occurring with the settlement of the exterior span have a span ratio between the first and center spans of 2.0 or greater. One exception exists and can be explained by the strange span configuration of the bridge. Note that the average of the first and center span lengths were used to plot the relationship between stress increase and span length.

For three-span continuous bridges, rolled steel girders and plate girders experienced approximately equivalent negative moment increases due to settlement. Prestressed concrete girders again experienced significantly larger negative moment increases than the steel section due to the much larger stiffnesses of the concrete members. There is not sufficient data to effectively compare the moment increases experienced by the three girder types in two-span continuous bridges to that of the three-span continuous bridges.

The results presented in Tables 4.3.5 and 4.3.6 detail the positive moment increases experienced by the girders of three-span continuous bridges. The positive moment increase data for the 1-inch settlement case is also plotted in Figures 4.3.4 and 4.3.5. The maximum positive moment increases experienced by the three-span bridges were found to occur at the settled interior support. Positive moment increases in the first spans of the studied bridges were generally larger than the moment increases in the center span. Since the majority of the center spans are longer than the first spans, larger load moments can be expected in the center spans, which would cause the percent increase in moment due to settlement to be smaller. Because the increases in positive moment at the three critical locations are not directly related to the moment capacity of the girders, however, the controlling positive moment location could be at any three of the locations investigated.

Prestressed concrete girders were again found to experience the largest increases in moment. At all three of the locations where positive moment increases were examined the concrete girders were found to be much more affected by differential settlement moments than the steel girders. The positive moment increases for the two types of steel girders were found to experience similar stress increases.

5.3.3: Tolerance of Two-Span Continuous Bridges to Settlement

Aside from examining the general affects of differential settlement on bridge girders, investigating percent increases in moments due to settlements is not overly useful. The tolerance of bridges to differential settlement cannot be determined using percent increases in moments at any location. Instead, the magnitude of moments caused by differential settlements must be compared to the reserve moment capacity of the bridge girders.

For the two-span in-service bridges, tolerable limits for settlement of the exterior and center supports were given in Tables 4.3.7 and 4.3.8, respectively. The tolerable limits were calculated using the reserve moment capacities for the bridge girders. The limits contained in Tables 4.3.7 and 4.3.8 were computed assuming that the reserve moment capacity of the bridge girders was calculated using factored load moments.

Increases in negative moments above the center support were used to determine the allowable settlement of the exterior support for each bridge. The results of the exterior support settlement analyses show that two of the three rolled steel girder bridges were tolerant to angular distortions above the 0.004-inch per inch limit suggested by Moulton et al. Only one of the steel plate girder bridges was found to have a tolerance to differential settlement resulting in an angular distortion above the empirical limit. Note that one bridge has a negative tolerance to differential settlement. The bridge rating calculated by Virtis for that bridge was less than 1.0, which suggests that the nominal flexural resistance of the bridge is not large enough to resist the Strength I limit state loads, much less any additional moments caused by differential settlement. All of the prestressed concrete bridges were found to have tolerances at or below the expected level.

Center support settlements were found to be limited by positive moment increases occurring either at the mid-point of the first span or at the center support. Table 4.3.8 shows that the location of the critical positive moment increase is not the same for all of the bridges. These results support the argument that the positive moment increase at both the mid-point of the loaded span and the center support need to be considered.

All of the steel bridge girders were found to have tolerable angular distortion limits well above the 0.004-inch per inch limit. The large tolerance of the steel bridges is to be expected since many of the bridge designs were controlled by negative moment requirements. Steel bridge girders appear to typically have enough additional moment capacity to withstand large magnitudes of differential settlement. The prestressed concrete bridges, however, were not largely tolerant to the center support settlement. Only the longest span bridge examined was found to have a tolerable limit greater that the empirical limit.

In addition to examining the differential settlement tolerances of the bridge girders based on factored moment capacities, reserve moment capacities calculated using unfactored loads were also used to investigate allowable support settlements. All of the two-span continuous steel bridges investigated were found to be tolerant of differential settlements well above the prescribed limit. Large settlements of both exterior and center settlements were found to be tolerable, though much larger center support settlements were determined to be tolerable. Two of the three prestressed concrete girders would be expected to tolerate angular distortions of at least 0.004 inches per inch caused by settlement of the exterior or center support. The bridge that fell below the expected tolerable distortion limit was unable to withstand angular distortions larger than 0.0027 inches per inch caused by exterior support settlement or larger than 0.0025 inches per inch caused by interior support settlement.

Based on the differential settlement tolerance data, steel bridges appear to be more tolerant to differential settlement than concrete bridges. It should be noted, however, that complex analyses are required to accurately model the behavior of prestressed concrete bridges. The results of the analyses may not be completely representative of prestressed concrete bridges. The load factors applied to the bridge loads that are used to design girders appear to provide enough additional moment capacity in the bridge girders to account for modest levels of differential settlement. Enough additional moment capacity to withstand realistic differential settlements does not appear to be present in prestressed concrete bridges.

5.3.4: Tolerance of Three-Span Continuous Bridges to Settlement

The allowable settlements of the exterior and interior supports based on limiting negative moments at the supports of three-span continuous bridges were given in Table 4.3.11. Reserve moment capacities calculated assuming factored loads were compared to the negative moments caused by settlement of each support. The negative moment increases discussed in Section 5.3.2 suggested that settlement of the interior support caused the maximum negative settlement moment in a given girder. The results in Table 4.3.11 confirm that observation. It should be noted, however, that interior support settlement does not cause the maximum negative moment in every case. Therefore, both settlement cases need to be considered for all bridges.

Only two of the five rolled girder bridges, three of the seven plate girder bridges and none of the prestressed concrete bridges were found to be able to tolerate angular distortions of at least 0.004 inches per inch. Note that one of the rolled steel girder bridges was found to have a rating factor of less than 1.0 (represented as a negative tolerance in Table 4.3.11) and, therefore, cannot withstand any differential vertical movement. When comparing the tolerable settlement limits given in Table 4.3.11 to the limits presented in Table 4.3.12, it is clear that negative moment increases control the magnitude to tolerable differential settlement.

Table 4.3.12 presents the tolerable angular distortion limits determined by limiting positive moments to allowable levels based on factored moment capacities. Critical positive moment increases were found to generally occur either at the mid-point of the first span or above the settled support. However, positive moment increases could be critical at any of the mid-span or settled support locations. Every possible critical location should be checked to ensure that a representative tolerance is calculated.

All of the steel bridges were found to have sufficient tolerances to support settlements causing positive moment increases. In fact, all of the tolerable angular distortion values determined for the steel bridges were above 0.0090 inches per inch with the majority being above 0.0200 inches per inch. Prestressed concrete bridges were found to be quite intolerant to interior support settlements. Allowable angular distortions of only 0.0004, 0.0007, and 0.0026 were calculated for the concrete bridges based on reserve moment capacities.

When more realistic moment capacities are considered, the tolerances of the in-service bridges were found to be more in line with the angular distortion limit proposed by Moulton et al. The use of reserve moment capacities calculated utilizing unfactored loads likely produces settlement tolerances more representatives of what would be observed in the field. The tolerances given in Table 4.3.13 based on negative moment increases due to exterior and interior support settlements suggest that all of the steel bridges studied should be able to withstand an angular distortion of at least 0.004 inches per inch. In many cases, the exterior support can be allowed to settle more than the interior support. All of the prestressed concrete bridges, however, were found to have low tolerances to differential settlement. In order to be considered tolerable, concrete bridge support settlements should be limited to less than 0.002 inches per inch.

The tolerable settlement limits presented in Table 4.3.14 are based on unfactored moment capacities and critical positive moment locations. The data provided in Table 4.3.14 suggests that all of the three-span continuous steel bridge investigated should be able to tolerate angular distortions greater than 0.0200 inches per inch. Again, all of the prestressed concrete bridges were found to have low tolerances to differential settlement.

Based on the results of the three-span continuous bridge analyses, steel bridges appear to be more tolerant to differential settlements than concrete bridges. The difference between the two types of bridges is likely the high stiffnesses of the prestressed concrete I-girders. Stiff girders led to high moments caused by differential settlement. It is important to note that the time effects of concrete would likely alleviate some of the stresses caused by differential settlement. This study, however, was concerned with the affects of immediate settlement.

For all bridge types, differential support settlements were limited by negative stress increases. Tolerable support settlement limits determined based on positive moment increases were found to be larger than the limits controlled by negative moment increases, especially for the steel bridges. Though the tolerable angular distortion values was different for each bridge studied, the 0.004-inch per inch limit proposed by Moulton et al. appears to be a decent representation of the lower end of the determined tolerances.

5.3.5: Additional Factors Affecting Settlement Tolerances

The true tolerance of the studied bridges to differential settlement is likely not accurately represented by the results discussed in Sections 5.3.3 and 5.3.4. As many of the early researchers found, accurately analyzing the response of a structure to settlement is very difficult. Many conservative assumptions are made to simplify the analyses, which lead to results that are not truly representative of what in-service bridges experience in the field.

The negative moment increases caused by differential settlement were found to limit support settlement in all cases. The reinforcement in the bridge deck was not considered when determining the reserve negative moment capacities of the steel bridge girders. Accounting for the additional capacity provided by the reinforcing steel would increase the tolerable support settlement limits. The prestressed concrete bridges, however, were made continuous for live load with the deck reinforcement. The full negative moment capacities of the concrete bridges are due to the deck reinforcement. Therefore, no additional increases in tolerance would be possible.

Much of any discrepancy between the tolerances of a bridge to differential settlement determined through analysis and observed in the field would likely be due to the behavior of the given bridge as a system. Interactions between structural members in a bridge are complex and not fully characterized by line-girder analyses. Moment increases due to settlement could be relieved through inelastic redistribution of stresses or other similar behaviors. Additionally, skewed bridges would not be expected to behave in the same way non-skewed bridges behave. As skew angles increase, the response of a bridge system to differential settlement would likely change.

The use of integral bridge abutments will also likely affect the response of a given bridge to differential vertical movement. Since integral abutment bridges are gaining popularity, the response of these bridges needs to be investigated. The ends of exterior integral abutment bridge spans are not free to rotate, but also are not completely fixed. Since the exterior support conditions differ from traditional exterior supports, the actual girder behavior is unknown. Additional stresses due to the support conditions are possible and need to be investigated. Overall, the response of integral abutment bridges to differential settlement is not clear and, therefore, requires further investigation beyond this study.

5.4: Relationship between Stiffness and Settlement-Induced Stresses

Figures 4.4.1 and 4.4.2 provide insight into the response of a two-span continuous bridge to settlement. As the moment of inertia of a member gets larger, the moment induced in that section by differential settlement will also get larger. As the length that the member is spanning gets larger, the moment induced by settlement quickly decreases. These trends can clearly be seen for negative settlement moments in Figure 4.4.1 and positive settlement moments in Figure 4.4.2.

Since the moment due to settlement increases linearly as the differential settlement increases, the slopes of the trendlines in each of the figures are also related linearly. Therefore, the slope of the 1-inch settlement trendline can be used as the basis upon which moments due to any magnitude of settlement can be determined. Calculating the negative moment caused by settlement of the exterior support of a two-span continuous bridge is as simple as using the following equation.

$$M_{SE} = S_1 \frac{I}{\ell^2} u \tag{5.4.1}$$

The slope of the 1-inch settlement trendline (S_1) from Figure 4.4.1 (-3,579) is first multiplied by the I/ℓ^2 value (in inches) of the girder of interest. Finally, multiply the resulting value by the anticipated settlement (u) will result in the negative moment (in kip-feet) that can be expected due to the anticipated settlement. Likewise, the positive moment increase at the center support of a two-span continuous bridge due to settlement of the center support can be determined in the same way by using the 1-inch trendline slope from Figure 4.4.2 (7,158) rather than -3,579. Positive moment increases at any point along a girder can be easily determined since the positive moment due to settlement increases linearly from zero at the free end of the girder to the maximum value at the center support.

5.4.1: Number of Spans

In order to provide a more meaningful and comprehensive method of determining the moments caused by differential settlement, bridges with varying numbers of spans needed to be investigated. Because the stiffness of a bridge increases as the number of spans increase, any moment caused by settlement will also increase. Figures 4.4.3, 4.4.4, and the figures in Appendix B relate moment increases due to settlement to the I/ℓ^2 values for three-, four-, five-, and six-span continuous bridges. The procedure described in the previous section for two-span continuous bridges can be modified using the 1-inch settlement trendline slope value from the figure pertaining to the number of spans of interest. It should be noted that the plotted data is based on analyses of prismatic continuous girders with equal span lengths over the entire length of the bridge. The effects of varying span lengths or member properties for a single bridge were not investigated.

For two-span continuous bridges, the maximum negative moment due to settlement occurred with the settlement of an exterior support. Figure 4.4.3 shows that settlement of the exterior support does not produce the maximum negative moment for three-span continuous bridges. The negative moment plots in Appendix B show similar trends for the other multi-span continuous bridges. The differential settlement coefficient (C_s) values plotted in Figure 4.4.5, which relate directly to the slope of the trendlines used to calculate moments due to settlement, suggest that the number of spans that a bridge has does not significantly affect the negative moment caused by settlement of the exterior support. The negative moments caused by settlement of an interior support for bridges with three or more spans were found to be considerably larger than the negative moments caused by exterior support settlement.

Converting the slope values from the various plots to C_s values as discussed in Section 3.5 provides a useful method of determining the theoretical moment that can be expected due to a specified settlement for a bridge consisting of any number of spans. The C_s values pertaining to negative moment increases for bridges with different numbers of spans are plotted in Figure 4.4.5. As was discussed, the C_s value for two-span continuous bridges was determined from the settlement of an exterior support and is, therefore, smaller in magnitude than the other C_s values. As the number of spans increase, the increase in moment due to settlement quickly converges to a constant value. Bridges with more than four spans will experience similar negative moment increases due to settlement. C_s values can, therefore, be held constant for bridges with five or more spans. When calculating negative moments in steel girders due to differential support settlement, the following equation should be used to determine appropriate C_s values.

$$C_{s-} = \begin{cases} 0 & N_s < 2 \\ 0.34N_s^2 - 2.53N_s + 2.22 & 2 \leq N_s \leq 4 \\ -2.60 & N_s > 4 \end{cases} \tag{5.4.1}$$

where N_s is the number of continuous bridge spans.

Figure 4.4.6 shows the C_s values determined from the slopes of the trendlines of the positive settlement moment plots. A clear limit to the value of the settlement coefficient can be seen. Bridges containing four or more spans were found to have the same C_s value, which indicates that the maximum positive moment caused by settlement is virtually the same for bridges with four or more spans. The following equation should be used to determine appropriate C_s values required to calculate the maximum positive moment in steel girders due to differential support settlement.

$$C_{s+} = \begin{cases} 0 & N_s < 2 \\ 0.60N_s + 1.75 & 2 \leq N_s \leq 4 \\ 4.15 & N_s > 4 \end{cases} \tag{5.4.2}$$

The C_s values calculated using Equations 5.4.1 and 5.4.2 are specific to steel girders. C_s values for prestressed concrete girders can be calculated. Equations that can be used to calculate the C_s values for prestressed concrete girders will not be given here because the values will vary depending on compressive strength of the concrete. Equation 3.5.7 along with the graphs presented in Section 4.4 and Appendix B should be used to determine C_s values for concrete.

Note once again that the C_s values given in Equations 5.4.1 and 5.4.2 are for use with prismatic continuous girder bridges with equal span lengths. Additional research into the effects of varying span lengths in a given bridge and varying cross-sectional properties for a given span on moment increases caused by differential settlement is needed. The results of such research could be used to modify the differential settlement coefficient. Also, any material properties that would limit the effect of the increased moments, such as concrete creep, could be accounted for. A more representative C_s value would allow for the moment increases in a wider range of bridge girders to be more easily calculated.

Once appropriate negative and positive C_s values are calculated, Equation 3.5.4 should be used to compute the theoretical negative and positive moments that will be experienced by a girder due to the specified settlement, u. This method of determining the moments caused by differential settlement is intended to replace the design aids created by Moulton et al. Rather than requiring approximate values to be determined from multiple charts, a single set of equations can be used to determine the expected moment increase. The calculated differential settlement moments can then be compared to the reserve moment capacities of existing bridges or used in the design of new bridge girders.

5.5: Bridge Design Process

Increases in structural stress due to differential settlement of bridge substructures are typically not accounted for when designing bridges. The usual method of dealing with settlement is designing the bridge foundations so that measurable settlement will not occur. Several past researchers have commented on the poor economic aspects designing foundations that will settle very little. Despite the recommendations of past researchers, a specific method of accounting for the response of a bridge to settlement has not yet been included in the bridge design specifications.

The bridge analyses discussed in Sections 5.1, 5.2, and 5.3 confirmed angular distortions of 0.004 inches per inch or less should be tolerable for most bridges. Though the design stresses of bridges designed using the Standard Specifications and moment capacities of bridges designed using the LRFD Specifications and the in-service bridges were often exceeded due to small settlements, the yield stresses and unfactored moment capacities were not surpassed in most cases. The additional safety built into bridge designs will generally be enough to offset the detrimental effects of differential settlements that cause angular distortions less than 0.004 inches per inch. When the additional load carrying capacity of a bridge is used to offset stresses induced by differential settlement, however, the safety of the bridge is reduced. Oversized loads or other unexpected forces would be unaccounted for and would likely cause intolerable damage to the bridge. Despite the fact that certain amounts of differential settlement can be tolerated by a bridge, settlement stresses need to be incorporated into bridge designs in order to maintain the safety, reliability, and serviceability of future bridges.

The goal of this research is to use the recommendations of past researchers to create a design method that can easily be incorporated into the AASHTO LRFD Bridge Design Specifications. Using the differential settlement moment calculation method presented in Section 5.4 along with the several of the recommendations discussed in Chapter 2, the design code can easily be modified to account for potentially damaging differential settlements. Inclusion of differential settlement criteria will allow future bridges to better resist damage and last longer.

Various researchers pointed out that designing for differential settlement involves two main tasks. First the differential settlement that can be expected at the bridge site must be estimated. Second, the response of the bridge superstructure to the differential settlement of the substructure must be determined.

5.5.1: Differential Settlement Estimation

Fairly large margins of error often accompany settlement predictions. The potential for underestimation of differential settlements and, therefore, additional stresses in the bridge superstructure can be accounted for in two different ways. First, the method proposed by Duncan and Tan, and discussed in Section 2.7.2, can be used. Assuming the maximum moment provided by a fairly simple settlement estimation method occurs at one end of a span and no settlement at the other end should provide a conservative differential settlement value. The second method involves using a more complex and accurate settlement estimation method. Smaller, less conservative differential settlement values can likely be obtained using a more sophisticated settlement estimation method.

Differential settlement values should be calculated for all applicable foundation types. Deep foundations are typically used in an attempt to reduce vertical moment of the bridge, but if movement is expected and designed for, shallow foundations may provide an economical alternative. If the cost of increasing girder size to tolerate expected settlements is less than the additional cost of using deep foundations, designing for settlement would be cost-effective. Investigating multiple foundation designs will help engineers create adequate and economical designs.

5.5.2: Differential Settlement Moment Calculation

The differential settlement value determined as described in Section 5.5.1 should be substituted into Equation 3.5.4 along with the appropriate C_s value calculated using Equation 5.4.1 or 5.4.2 (for steel girders), modulus of elasticity, moment of inertia, and span length. Alternatively, settlement moments can be obtained by assembling and analyzing models with an appropriate structural analysis program. Either way, both negative and positive moments due to settlement should be determined for use in the bridge design process.

5.5.3: Properly Incorporating Settlement into the Design Process

Moulton et al. and Samtani and Nowatzki suggest that incorporating the effects of differential settlement in the design of a bridge will be an iterative process. The settlement moment equations developed in the previous section confirm the need for an iterative design process. The general design methodology suggested by Moulton et al. and given in Figure 2.7.1 provides a good starting point for developing an effective design process. First, a bridge layout must be selected based on site requirements and restrictions. A preliminary bridge should then be designed based on typical loading conditions. Once the general locations of the bridge foundation are determined, settlements can be estimated based on the soil conditions. Multiple foundation types should be investigated when possible.

Once the settlement data is obtained, negative and positive moments due to the expected differential settlement can be calculated based on the expected differential settlements and the properties of members in the preliminary design. In order to provide a worst case scenario, immediate settlements should be assumed rather than slow settlement, which can allow the structural members to adjust and relieve the additional stresses. Load factors should then be applied to the settlement moments. The load factor applied to the settlement moments will likely depend on the method of differential settlement estimation used. If the conservative method proposed by Duncan and Tan is used, a relatively low load factor, perhaps 1.0, would likely be appropriate. If a more complicated method of settlement estimation is used, a load factor will need to be determined based on the reliability of the settlement predictions.

All applicable moments should then be compared to the calculated flexural resistance. Necessary changes to the layout and/or structural design should be made. This process will repeat and continue until at least one appropriate design is obtained. Ideally, designs utilizing different foundations will be created so that economic comparisons can be made.

5.5.4: Construction Sequence

In addition to designing bridges to withstand a certain amount of differential vertical movement, proper construction methods should be specified. In order to reduce the likelihood of settlement occurring over the life of the bridge, all soils, especially fill soils, should be adequately consolidated prior to the start of construction. Proper consolidation can significantly reduce substructure movement in all directions.

Once construction begins, the placement of bridge components are constructed and assembled should be sequenced in such a way that the members most affected by differential settlement are placed as late as possible. Since members are only subjected to the settlement that occurs after placement, members placed near the end of the construction period will not be subjected to any settlements that may occur during bridge construction.

5.6: Bridge Design Considering Settlement

Comparing bridges designed with and without differential settlement consideration allows for the feasibility of adding tolerable substructure movement criteria into the design code to be examined. The comparison of girders determined to be adequate for the simplistic bridges designed using the AASHTO LRFD Bridge Design Specifications presented in Table 4.5.1 suggests that accounting for differential settlements during the design process is feasible. With the exception of the 30-foot span bridge, all of the bridge designs required only small increases in section size to produce girder designs capable of withstanding 3 inches of differential settlement. Though a significant section increase was required for the 30-foot span bridge, the likelihood of using a section as small as a W21x57 for bridge girders under normal circumstances is likely low.

Increasing section size and the amount of steel used by a small amount should be considered a small, but necessary additional investment in a bridge. Designing a bridge capable of withstanding a certain amount of differential settlement will ensure that the bridge remains strong and serviceable up to the end of and beyond its intended service life. Though a great majority of bridges are designed without accounting for the potential effects of differential support movement and do not display signs of damage due to differential vertical movement, the cost associated with slight increases in members sizes is a small price to pay to ensure that no future problems will arise.

Chapter 6

DESIGN EXAMPLE

The following is a simple example demonstrating how to properly account for an expected differential settlement during the design of bridge girders. The bridge of interest is a two-span continuous rolled steel girder bridge with 50-foot spans. Assume that a potential differential settlement of 3 inches has been calculated. The procedures given in the AASHTO LRFD Bridge Design Specifications will be followed, including the provisions outlined in Appendix A6. An interior girder will be designed assuming the following:

-The bridge cross-section is as given in Figure 3.2.1

-8.5" composite concrete deck including 0.5" wearing surface

-150 pcf reinforced concrete

-4000 psi concrete

-Grade 50 steel

-15 plf/girder for dead loads of cross-frames, stiffeners, miscellaneous details

-165 plf/girder for dead load of curbs and railings

-No future wearing surface

-Dead load equally distributed to all beams

-Reinforcement yield strength = 60 ksi

-η_D, η_R, η_I = 1

-No moment redistribution

-Live load deflections are not limited

-Axial loading is negligible

Since a W30x90 was found to be adequate for the 50-foot span bridge without settlement consideration, that is a logical section to begin with. Applying the dead and live loads specified in the LRFD Specifications to produce maximum negative and positive moments and analyzing produces the unfactored moments given in Table 6.1. Note that appropriate distribution factors have been applied.

Table 6.1: Unfactored Moments for a W30x90 Girder

% Span	**Distance (ft)**	**DC1 (k-ft)**	**DC2 (k-ft)**	**LL (k-ft)**	
				-	**+**
0	0	0.00	0.00	0.00	0.00
0.1	5	77.58	13.42	46.58	197.62
0.2	10	131.33	22.67	82.98	385.08
0.3	15	161.17	27.83	109.15	454.16
0.4	20	167.17	28.92	125.22	486.02
0.5	25	149.25	25.75	131.11	507.72
0.6	30	107.42	18.58	126.84	383.99
0.7	35	41.75	7.25	92.60	250.16
0.8	40	-47.75	-8.25	-255.20	99.33
0.9	45	-161.17	-27.83	-278.48	-88.72
1	50	-298.42	-51.58	-477.75	-286.94

Next, the theoretical moment caused by the expected differential settlement must be calculated. Settlements of both the exterior and interior support must be examined. The short-term moment of inertia of the composite section is needed to calculate the settlement moment. Additionally, the appropriate C_s values must be computed.

b_e =	8	ft
	96	in
t_s =	8	in
n_{short} =	8	
n_{long} =	24	
I_x =	3610	in^4
A =	26.4	in^2
d =	29.5	in
$d_{top, steel}$ =	14.75	
$d_{bot, steel}$ =	14.75	
$S_{top, steel}$ =	244.75	in^3
$S_{bot, steel}$ =	244.75	in^3

	A (in^2)	d (in)	A*d (in^3)	A*d^2 (in^4)	I_0 (in^4)	I (in^4)
beam	26.4					3610
slab	96	18.75	1800	33750	512	34262
	122.4		1800			37872

d_s = 14.71 in

	-26470.59
I_{short} =	11401.41 in^4

Since $N_s = 2$, Equations 5.4.1 and 5.4.2 give

$C_{s-} = 0.34N_s^2 – 2.53N_s + 2.22 = -1.48$
$C_{s+} = 0.60N_s + 1.75 = 2.95$

Equation 3.5.4 can then be used to find the expected moment increase due to a 3-inch settlement of the exterior and center supports.

$$M_{SE} = \frac{C_s uEI}{12\ell^2}$$

$$M_{SE-} = \frac{(-1.48)(3in)(29000ksi)(11401.41in^4)}{12(50\times 12in)^2} = -339.83 \text{ k-ft}$$

$$M_{SE+,\ support} = \frac{(2.95)(3in)(29000ksi)(11401.41in^4)}{12(50\times 12in)^2} = 677.35 \text{ k-ft}$$

$$M_{SE+,\ mid\text{-}span} = \frac{677.35\text{k-ft}}{2} = 338.68 \text{ k-ft}$$

Using the Strength I load combination with a settlement load factor of 1.0, the factored moment required to be resisted by the girder can be calculated at the locations of maximum load moment. Additionally, the factored moment at the support needs to be calculated. The most critical case will involve the permanent dead loads calculated with a load factor of 1.0 and the moment due to settlement.

$$M_{u-,\ support} = (1.75)(-477.75\text{k-ft}) + (1.25)(-298.42\text{k-ft} + -51.58\text{k-ft}) + (1.0)(-339.83\text{k-ft})$$
$$M_{u-,\ support} = -1613 \text{ k-ft}$$

$$M_{u+,\ mid\text{-}span} = (1.75)(507.72\text{k-ft}) + (1.25)(149.25\text{k-ft} + 25.75\text{k-ft}) + (1.0)(338.68\text{k-ft})$$
$$M_{u+,\ mid\text{-}span} = 1446 \text{ k-ft}$$

$$M_{u+,\ support} = (1.0)(-298.42\text{k-ft} + -51.58\text{k-ft}) + (1.0)(667.35\text{k-ft})$$
$$M_{u+,\ support} = 317 \text{ k-ft}$$

The nominal flexural resistance of the W30x90 girder must be calculated. Following the procedure for calculating the nominal flexural resistance provided in Appendix A6 in the LRFD Specifications provides the nominal flexural resistances given. The nominal flexural resistances can then be compared to the factored moments calculated above in order to determine the adequacy of the selected section.

$\varphi_f M_{n-} = -1588$ k-ft
$\varphi_f M_{n-} \geq M_{u-}$? → NO GOOD

$\varphi_f M_{n+} = 2602$ k-ft
$\varphi_f M_{n+} \geq M_{u-,\ mid\text{-}span}$? → OK
$\varphi_f M_{n+} \geq M_{u-,\ support}$? → OK

The factored negative moment is larger than the resistance of the W30x90 section; therefore, a larger section must be selected. Since the required resistance and actual resistance were close for the W30x90 section, it is logical to try the next largest section. The process must be repeated assuming W30x99 girders.

Table 6.2: Unfactored Moments for a W30x99 Girder

% Span	**Distance (ft)**	**DC1 (k-ft)**	**DC2 (k-ft)**	**LL (k-ft)**	
				-	**+**
0	0	0.00	0.00	0.00	0.00
0.1	5	78.33	13.42	47.01	199.48
0.2	10	132.58	22.67	83.76	388.69
0.3	15	162.67	27.83	110.17	458.42
0.4	20	168.75	28.92	126.39	490.58
0.5	25	150.67	25.75	132.34	512.48
0.6	30	108.50	18.58	128.03	387.59
0.7	35	42.17	7.25	93.47	252.51
0.8	40	-48.17	-8.25	-257.60	100.26
0.9	45	-162.67	-27.83	-281.09	-89.55
1	50	-301.25	-51.58	-482.23	-289.63

b_e =	8	ft
	96	in
t_s =	8	in
n_{short} =	8	
n_{long} =	24	
I_x =	3990	in^4
A =	29.1	in^2
d =	29.7	in
$d_{top, steel}$ =	14.85	
$d_{bot, steel}$ =	14.85	
$S_{top, steel}$ =	268.69	in^3
$S_{bot, steel}$ =	268.69	in^3

	A (in^2)	d (in)	A*d (in^3)	A*d^2 (in^4)	I_0 (in^4)	I (in^4)
beam	29.1					3990
slab	96	18.85	1809.6	34110.96	512	34622.96
	125.1		1809.6			38612.96

d_s =	14.47	in		-26176.28
			I_{short} =	12436.68 in^4

$C_{s-} = 0.34N_s^2 – 2.53N_s + 2.22 = -1.48$
$C_{s+} = 0.60N_s + 1.75 = 2.95$

$$M_{SE-} = \frac{(-1.48)(3in)(29000ksi)(12436.68in^4)}{12(50 \times 12in)^2} = -370.68 \text{ k-ft}$$

$$M_{SE+, support} = \frac{(2.95)(3in)(29000ksi)(12436.68in^4)}{12(50 \times 12in)^2} = 738.86 \text{ k-ft}$$

$$M_{SE+, mid\text{-}span} = \frac{738.86\text{k-ft}}{2} = 369.43 \text{ k-ft}$$

$M_{u-, support}$ = (1.75)(-482.23k-ft) + (1.25)(-301.25k-ft + -51.58k-ft) + (1.0)(-370.68k-ft)
$M_{u-, support}$ = -1656 k-ft

$M_{u+, mid\text{-}span}$ = (1.75)(512.48k-ft) + (1.25)(150.67k-ft + 25.75k-ft) + (1.0)(369.43k-ft)
$M_{u+, mid\text{-}span}$ = 1487 k-ft

$M_{u+,\ support} = (1.0)(-301.25k\text{-}ft + -51.58k\text{-}ft) + (1.0)(738.86k\text{-}ft)$
$M_{u+,\ support} = 386\ k\text{-}ft$

$\varphi_f M_{n-} = -1802\ k\text{-}ft$
$\varphi_f M_{n-} \geq M_{u-}\ ? \rightarrow OK$

$\varphi_f M_{n+} = 2850\ k\text{-}ft$
$\varphi_f M_{n+} \geq M_{u-,\ mid\text{-}span}\ ? \rightarrow OK$
$\varphi_f M_{n+} \geq M_{u-,\ support}\ ? \rightarrow OK$

Use W30x99 girders

Note that the girders the above calculations are for interior girders only. Additionally, only flexural resistance was checked. Any lateral bending stresses were neglected, but should be considered if found to exist. All other applicable resistances and load combinations should be calculated and checked.

Chapter 7

CONCLUSIONS

Based on the review of literature and the bridge analyses performed, the following conclusions can be drawn regarding the tolerances of bridges to substructure movements:

- Although typical elastic modeling does not account for the true inelastic behavior of bridge members, models created in accordance with the LRFD Bridge Design Specifications can be useful in predicting bridge behaviors.
- Despite the need for conservative assumptions, analytical studies involving the behavior of bridges when subjected to differential settlement can produce acceptable results.
- Percent increases in stresses due to settlements do not relate directly to settlement tolerances.
- Because negative moments control the design of prismatic bridge girders, negative moment increases limit the tolerable differential support settlement of all bridges with prismatic girders.
- General critical settlement conditions cannot be applied to all bridges. Moment increases at all potentially critical locations need to be investigated.

- A lower bound of girder span lengths for which the effects of differential settlement were not a concern could not be determined.
- Steel bridges are more tolerant to differential settlements than concrete bridges because steel girders are less stiff than concrete girders, though the accuracy of the concrete bridge results is in question.
- The tolerable angular distortion limit proposed by Moulton et al. is accurate for bridges designed with the Standard Specifications and the LRFD Specifications.
- The reserve moment capacities included in bridge designs through the use of allowable stresses or load factors are generally large enough to facilitate typical magnitudes of differential settlement.
- Relying on the reserve moment capacities of bridge girders to offset the negative effects of differential settlement reduces the safety and reliability of bridges.
- Damage caused by substructure movements can be avoided by anticipating differential movement and designing the superstructure to withstand the expected movements.
- Moments caused by differential bridge settlement should be included in the design of bridges.
- Anticipated differential settlements should be conservatively estimated and used to determine the expected settlement moment using the proposed equation or an appropriate analysis program.
- Including settlement moments in a bridge design requires an iterative design process.

- Designing a bridge to tolerate differential settlement may be more economical than designing the substructure not to move.
- The inelastic rotational capacity of bridge girders will help to offset the negative effects of differential settlement.
- Specifying a proper construction sequence can reduce the differential settlement experienced by a bridge superstructure.
- Issues such as drainage, clearance, riding quality, and the tolerances of attached utilities also need to be considered when determining tolerable substructure movements.
- The relationship of I/ℓ^2 best relates the stiffness of a composite girder to the moment increase caused by differential vertical movement.
- Horizontal substructure movements cannot be easily investigated analytically and should be limited to 1.5 inches as proposed by Moulton et al.
- Bridges designed with integral abutments should be less susceptible to damage from horizontal substructure movements.
- Good engineering judgment must always be used throughout the design process.

Chapter 8

RECOMMENDATIONS

Based on the goals of the analyses and the obtained results, the following actions are recommended:

- Inclusion of the force effects caused by anticipated differential vertical movements of bridge substructures in the design of bridge girders should be required.
- Further investigation and calibration of the settlement load factor is needed.
- Further development of the settlement moment calculation equation is needed in order to provide a method of determining settlement moments applicable to a wide range of bridges.
- Settlement analyses of prestressed concrete bridges accounting for the time-dependent effects of concrete are needed.
- Three-dimensional analyses are needed to investigate the response of entire bridge systems to differential vertical movement.
- Analyses of integral abutment bridges are needed in order to determine the effect of integral abutments on bridge movement tolerances.
- Multiple foundation designs, including deep and shallow foundation options, should be created in an attempt to produce the most economical bridge design.

- Proper construction sequences should be used in order to reduce the differential settlement experienced by bridge superstructures.
- Rather than attempting to determine whether an expected movement will be tolerable, engineers should design for the effects of the movement to ensure that it will be tolerable.
- Though a bridge may be designed to tolerate potential settlements from a strength standpoint, all serviceability aspects of the bridge system must be considered.
- Riding quality, clearances, utilities, drainage, etc., must be able to be properly maintained throughout the life of the bridge.

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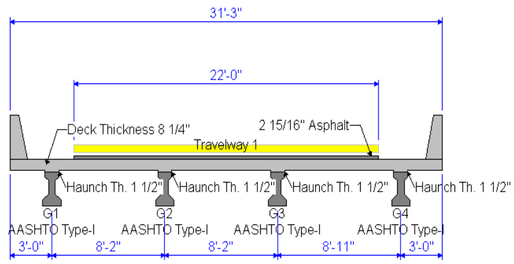
Appendix A

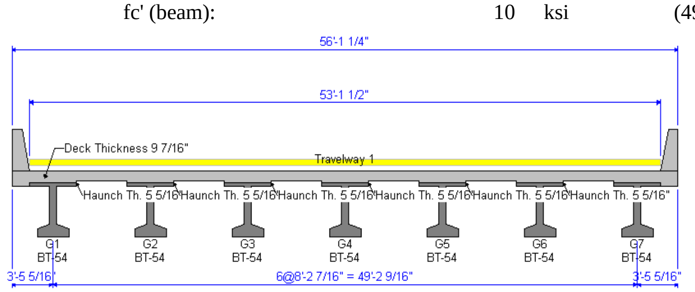
IN-SERVICE BRIDGE DETAILS

Two-Span Bridges

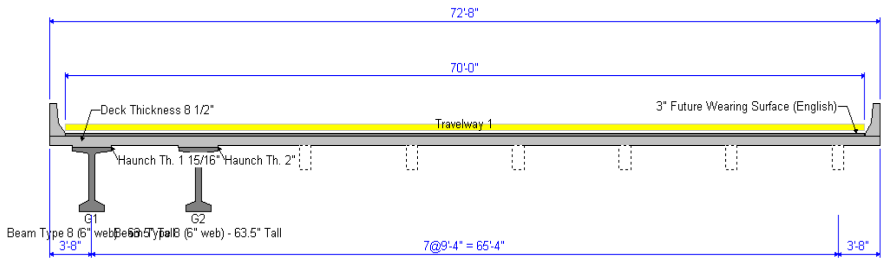
Bridge:	1166 (55-00876-0059)			
	Year:	2007		
	Skew:	0		
Material Properties:				
	fu (prestressing strand):	270	ksi	
	fy (stirrups):	60	ksi	
	fy (mild longitudinal):	60	ksi	
	fc' (deck):	4	ksi	
	fc' (beam):	5	ksi	



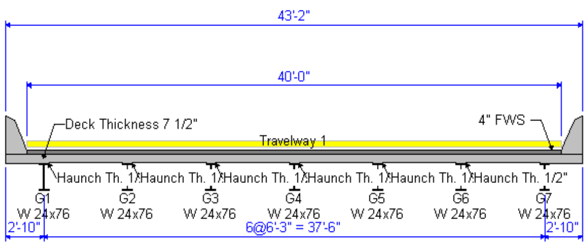
Bridge:	15991 (1003330)			
	Year:	2008		
	Skew:	0		
Material Properties:				
	fu (prestressing strand):	270	ksi	
	fy (stirrups):	60	ksi	(420 MPA)
	fy (mild longitudinal):	60	ksi	(420 MPA)
	fc' (deck):	4	ksi	(28 MPA)
	fc' (beam):	10	ksi	(49 MPA)



Bridge:	7715 (A7359)		
	Year:	2007	
	Skew:	-21.2	
Material Properties:			
	fu (prestressing strand):	270	ksi
	fy (stirrups):	60	ksi
	fy (mild longitudinal):	60	ksi
	fc' (deck):	4	ksi
	fc' (beam):	7	ksi



Bridge:	2937 (0810164)		
	Year:	2005	
	Skew:	-15	
Material Properties:			
	fy (girder):	50	ksi
	fc' (deck):	3.5	ksi
	fy (deck r/f):	60	ksi

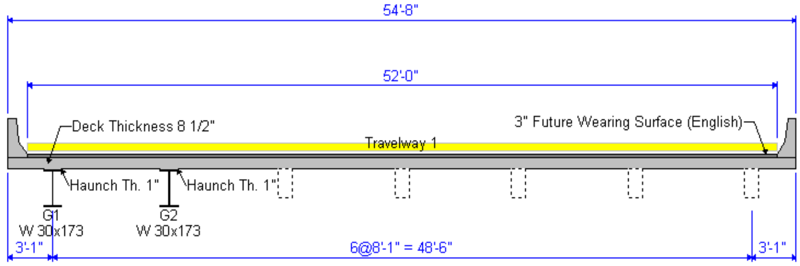


Bridge: 6114 (A7300)

Year:	2006	
Skew:	-1	

Material Properties:

fy (girder):	50	ksi
fc' (deck):	4	ksi
fy (deck r/f):		ksi

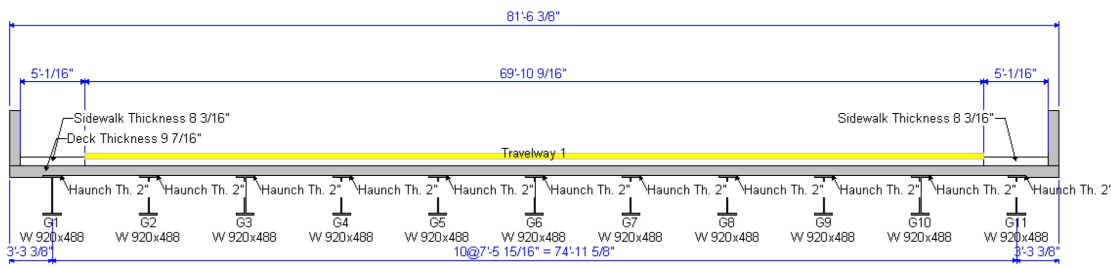


Bridge: 16464 (1044600)

Year:	2003		
Skew:	2.7		

Material Properties:

fy (girder):	50W	ksi	(345 MPA)
fc' (deck):	4	ksi	
fy (deck r/f):	60	ksi	



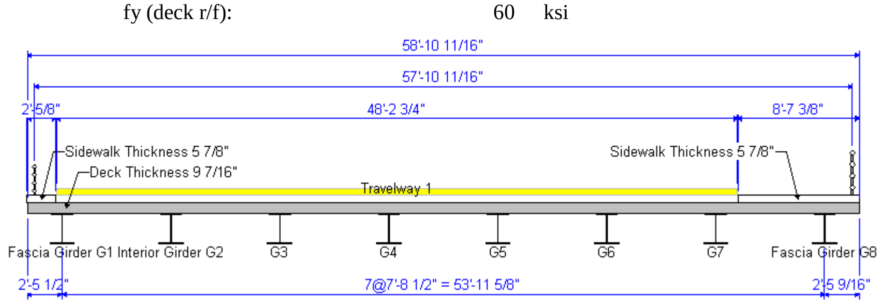
*W36 x 328 NOT W920x488

Bridge: 16830 (1060310)

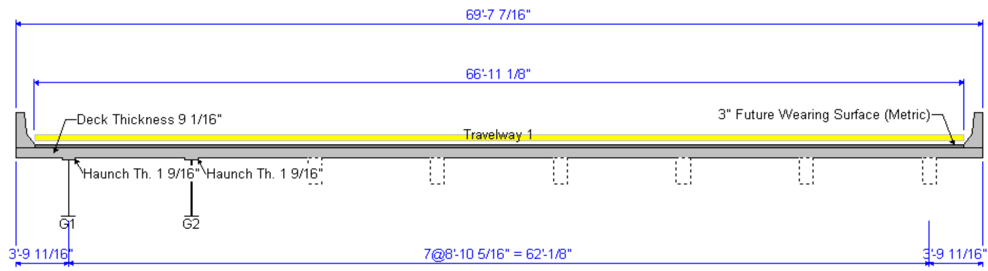
Year:	2000	
Skew:	0	

Material Properties:

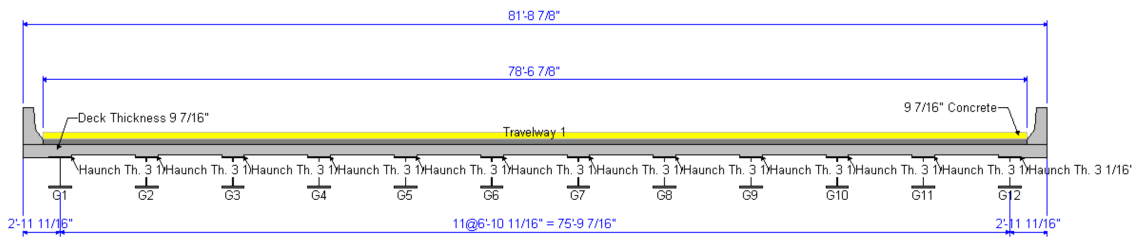
fy (girder):	50	ksi
fc' (deck):	3.3	ksi
fy (deck r/f):	60	ksi



Bridge:	7953 (A7189)			
	Year:	2007		
	Skew:	0		
Material Properties:				
	fy (girder):	50	ksi	(345 MPA)
	fc' (deck):	4	ksi	(28 MPA)
	fy (deck r/f):		ksi	

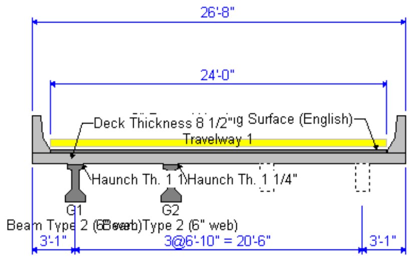


Bridge:	16266 (1052690)			
	Year:	2008		
	Skew:	-8		
Material Properties:				
	fy (girder):	50	ksi	(345 MPA)
	fc' (deck):	3	ksi	(21 MPA)
	fy (deck r/f):	60	ksi	(400 MPA)

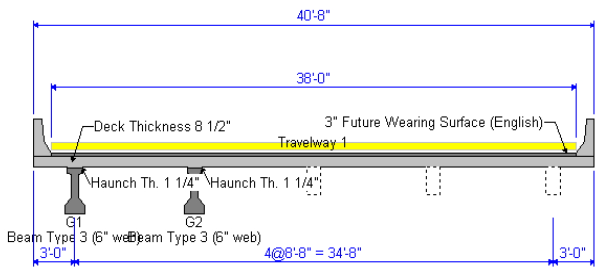


Three-Span Bridges

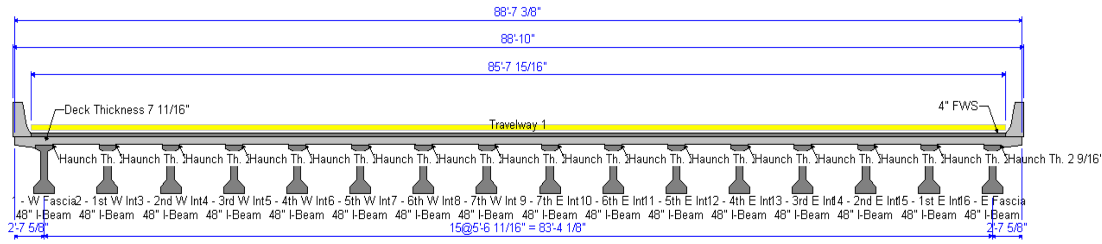
Bridge:	8787 (A6402)		
	Year:	2005	
	Skew:	0	
Material Properties:			
	fu (prestressing strand):	270	ksi
	fy (stirrups):	60	ksi
	fy (mild longitudinal):	60	ksi
	fc' (deck):	4	ksi
	fc' (beam):	6	ksi



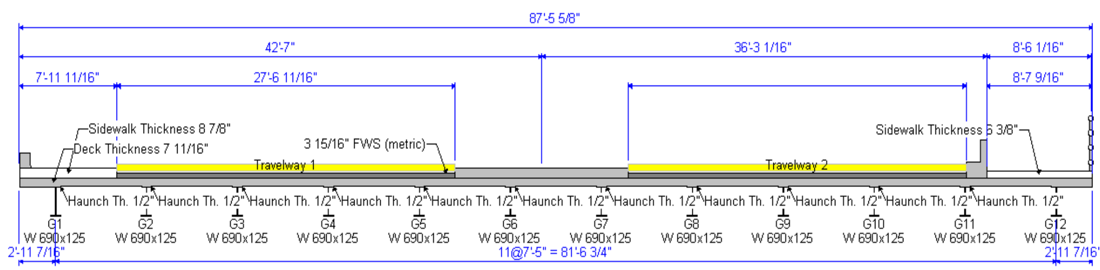
Bridge:	8442 (A7454)		
	Year:	2008	
	Skew:	-50	
Material Properties:			
	fu (prestressing strand):	270	ksi
	fy (stirrups):	60	ksi
	fy (mild longitudinal):	60	ksi
	fc' (deck):	4	ksi
	fc' (beam):	6	ksi



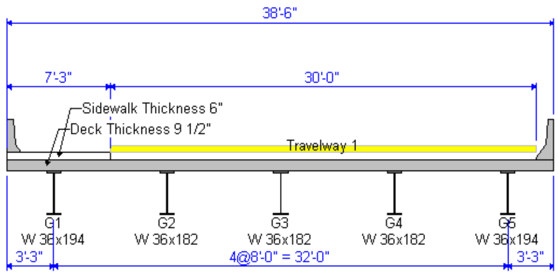
Bridge:	3015 (0162798)			
	Year:	2006		
	Skew:	25.5		
Material Properties:				
	fu (prestressing strand):	270	ksi	
	fy (stirrups):	58	ksi	(400 Mpa)
	fy (mild longitudinal):	58	ksi	(400 Mpa)
	fc' (deck):	3.48	ksi	(24 Mpa)
	fc' (beam):	6	ksi	(42 Mpa)



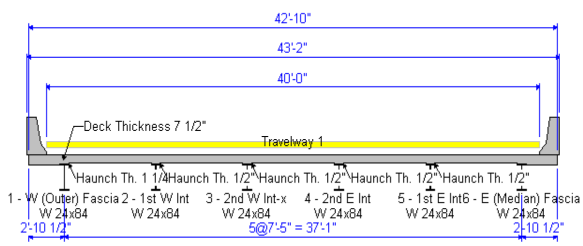
Bridge:	3191 (0490187)			
	Year:	2004		
	Skew:	0		
Material Properties:				
	fy (girder):	50	ksi	(345 Mpa)
	fc' (deck):	3.5	ksi	(24 MPa)
	fy (deck r/f):	58	ksi	(400 MPa)



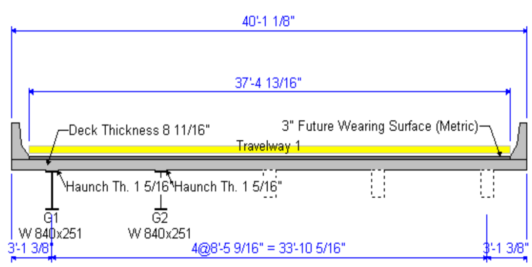
Bridge:	16548 (5524010_txo)			
	Year:	2008		
	Skew:	0		
Material Properties:				
	fy (girder):	50W	ksi	
	fc' (deck):	3	ksi	
	fy (deck r/f):	60	ksi	



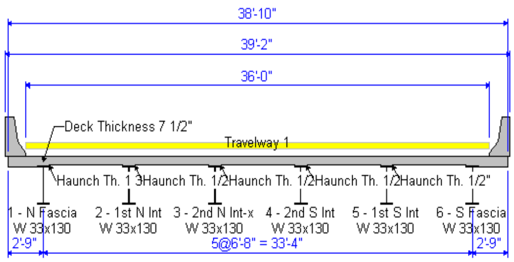
Bridge:	3766 (0990242)			
	Year:	2008		
	Skew:	0		
Material Properties:				
	fy (girder):	50	ksi	
	fc' (deck):	3.5	ksi	
	fy (deck r/f):	60	ksi	



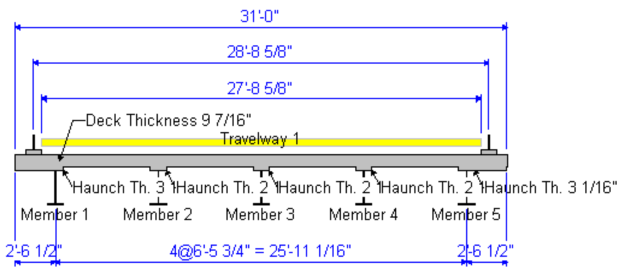
Bridge:	4789 (A6630)			
	Year:	2003		
	Skew:	22.5		
Material Properties:				
	fy (girder):	36	ksi	(250 MPa)
	fc' (deck):	4	ksi	(28 MPa)
	fy (deck r/f):	60.9	ksi	(420 Mpa)



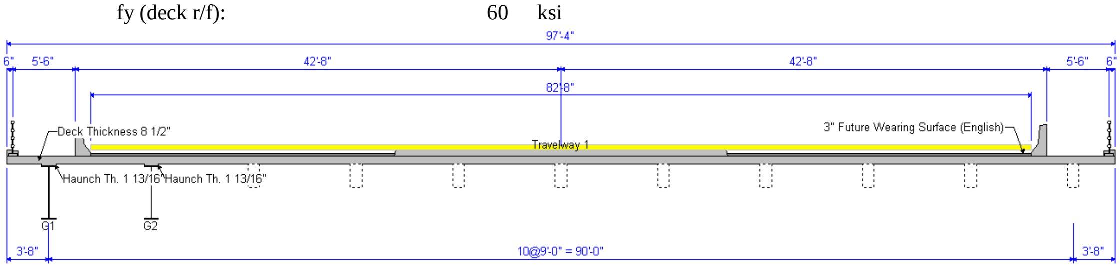
Bridge:	2178 (0410104)		
	Year:	2004	
	Skew:	0	
Material Properties:			
	fy (girder):	50	ksi
	fc' (deck):	3.5	ksi
	fy (deck r/f):	60	ksi



Bridge:	14035 (3334320)		
	Year:	2005	
	Skew:	14	
Material Properties:			
	fy (girder):	50	ksi
	fc' (deck):	3	ksi
	fy (deck r/f):	60	ksi



Bridge:	8873 (A7616)		
Year:		2008	
Skew:		8.5	
Material Properties:			
	fy (girder):	50W	ksi
	fc' (deck):	4	ksi
	fy (deck r/f):	60	ksi



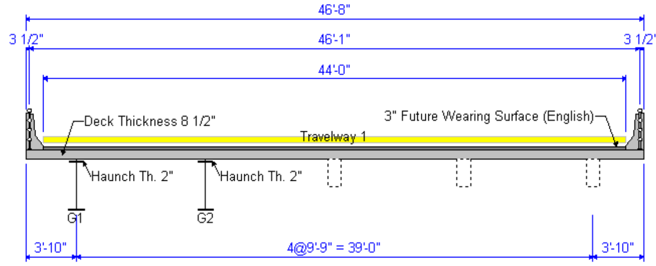
Bridge: 8850 (A7612)

Year: 2008

Skew: 0

Material Properties:

fy (girder):	50W	ksi
fc' (deck):	4	ksi
fy (deck r/f):		ksi



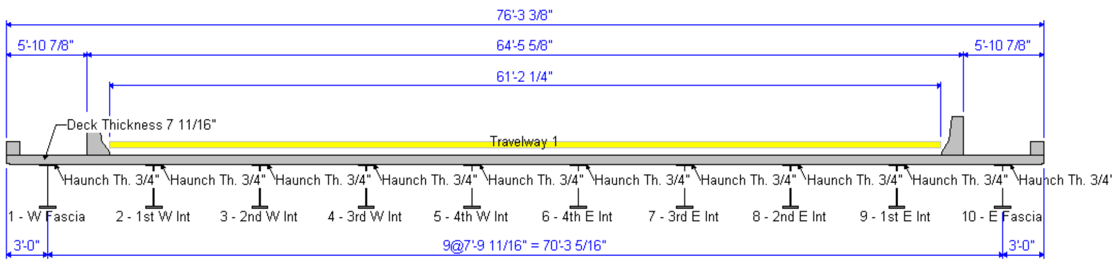
Bridge: 1620 (0460045)

Year: 2002

Skew: 20

Material Properties:

fy (girder):	50	ksi	
fc' (deck):	3.5	ksi	(24 Mpa)
fy (deck r/f):	58	ksi	(400 Mpa)



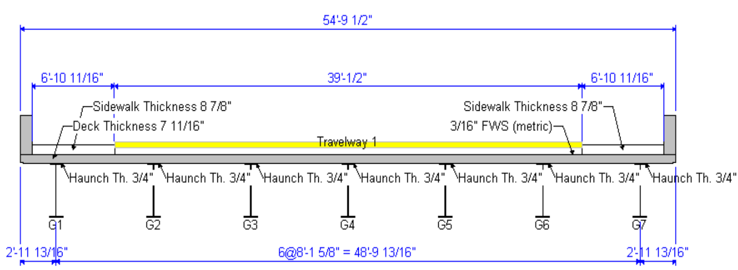
Bridge: 2735 (0162028)

Year: 2002

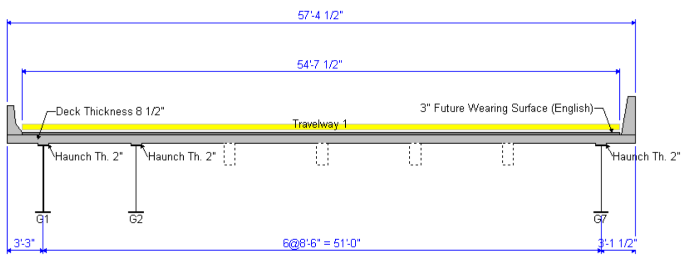
Skew: -22

Material Properties:

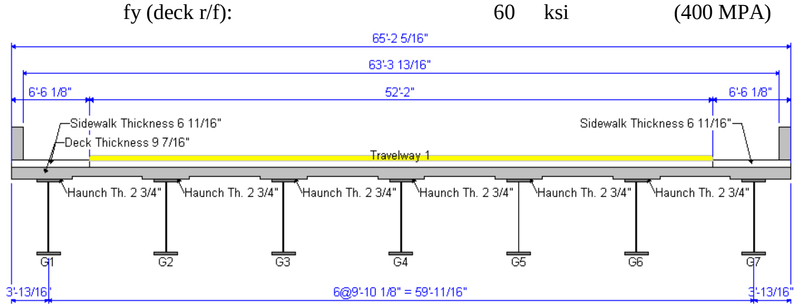
fy (girder):	50	ksi	(345 Mpa)
fc' (deck):	3.5	ksi	(24 Mpa)
fy (deck r/f):	58	ksi	(400 Mpa)



Bridge:	8894 (A7327)			
	Year:	2008		
	Skew:	0		
Material Properties:				
	fy (girder):			
	Web:	50	ksi	
	Top Flange:	50	ksi	0-70 ft
		70W	ksi	70-187 ft
		50	ksi	187-303 ft
		70W	ksi	303-420 ft
		50	ksi	420-490 ft
	Bottom Flange:	50	ksi	0-70 ft
		70W	ksi	70-187 ft
		70W	ksi	187-303 ft
		70W	ksi	303-420 ft
		50	ksi	420-490 ft
	fc' (deck):	4	ksi	
	fy (deck r/f):		ksi	



Bridge:	14096 (1011160)			
	Year:	2007		
	Skew:	20		
Material Properties:				
	fy (girder):	50W	ksi	(345 MPA)
	fc' (deck):	3	ksi	
	fy (deck r/f):	60	ksi	(400 MPA)



Appendix B

RELATIONSHIP BETWEEN STIFFNESS AND SETTLEMENT-INDUCED MOMENTS FOR FOUR-, FIVE-, AND SIX-SPAN BRIDGES

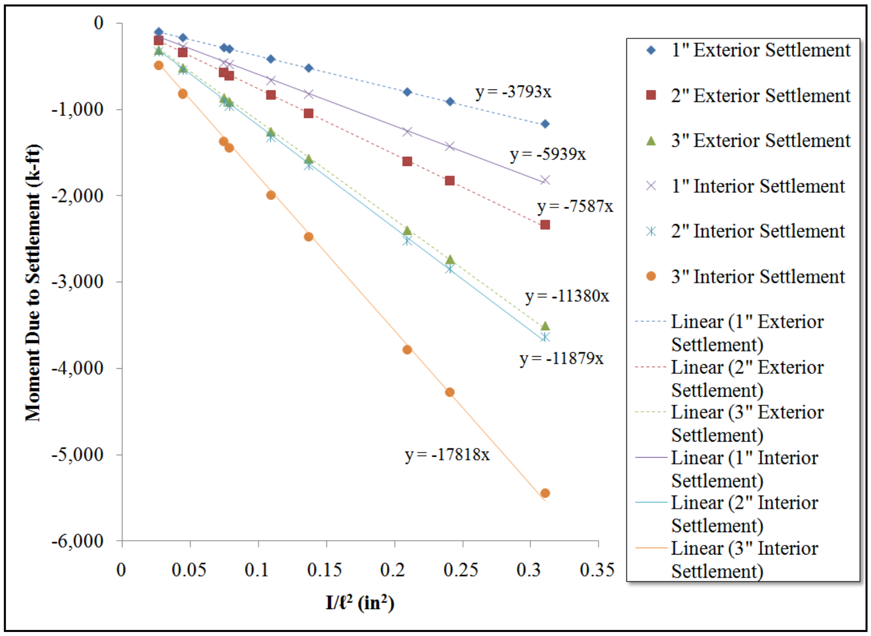


Figure B1: Negative Moments Caused by Differential Settlement of Four-Span Continuous Bridges

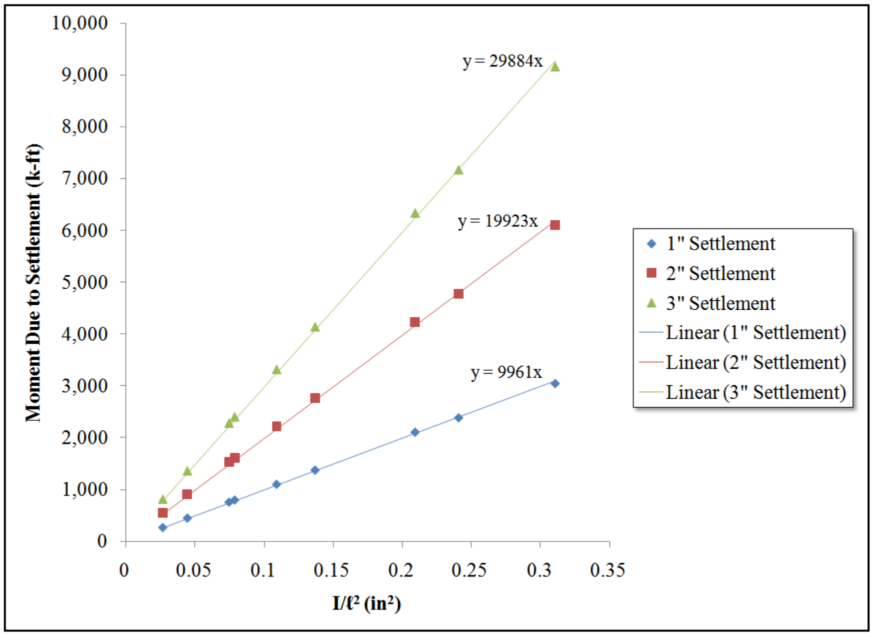


Figure B2: Positive Moments Caused by Differential Settlement of Four-Span Continuous Bridges

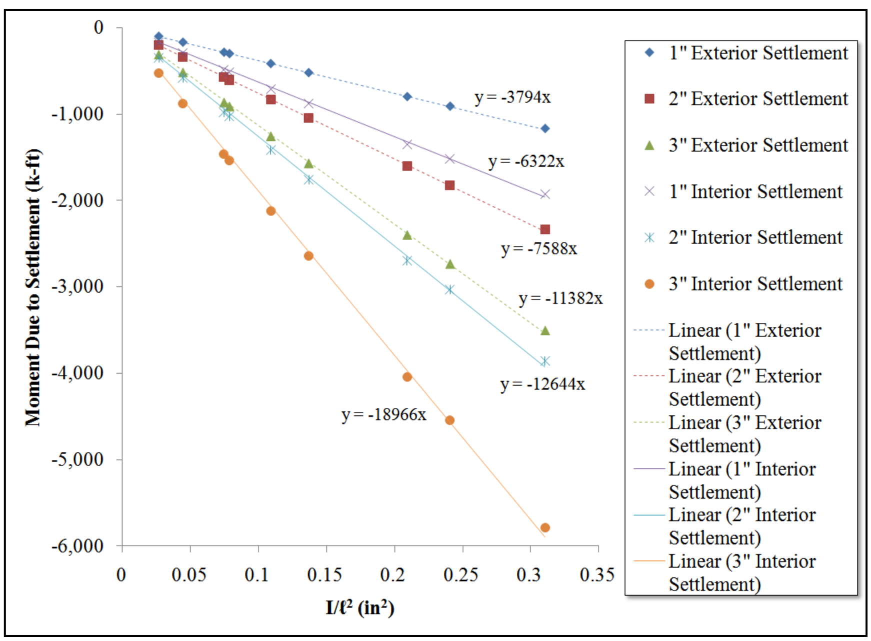


Figure B3: Negative Moments Caused by Differential Settlement of Five-Span Continuous Bridges

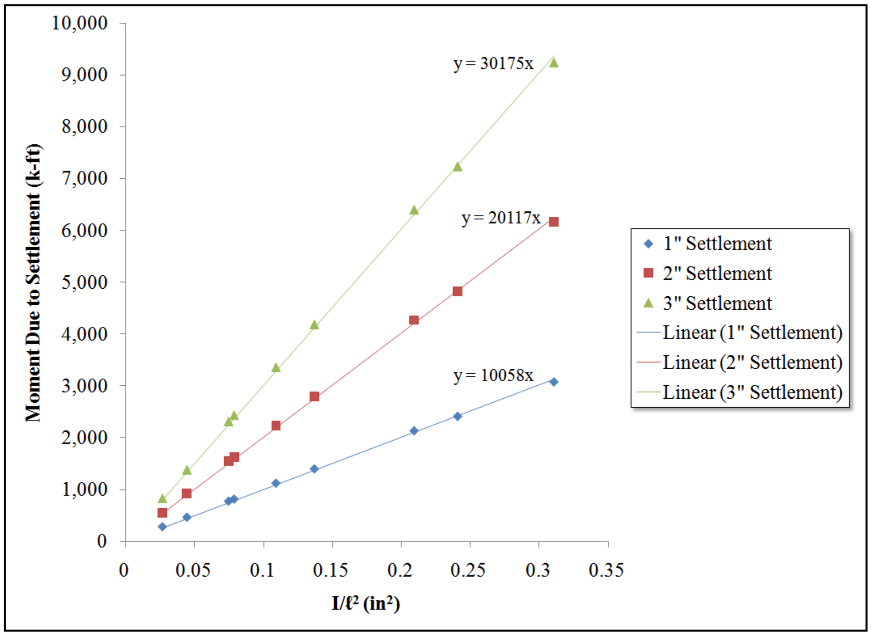


Figure B4: Positive Moments Caused by Differential Settlement of Five-Span Continuous Bridges

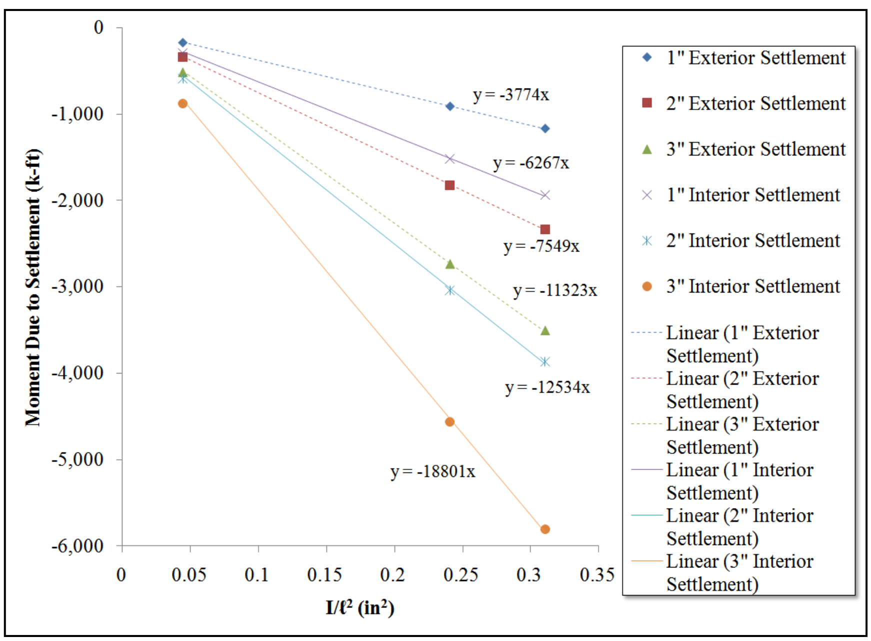


Figure B5: Negative Moments Caused by Differential Settlement of Six-Span Continuous Bridges

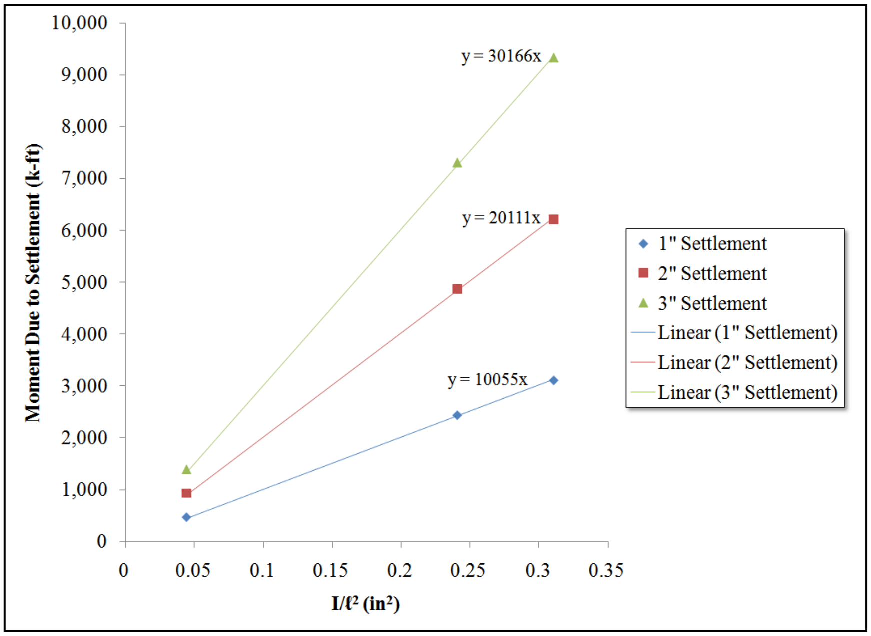


Figure B6: Positive Moments Caused by Differential Settlement of Six-Span Continuous Bridges