TOLERABLE MOVEMENT CRITERIA FOR HIGHWAY BRIDGES

Research, Development, and Technology

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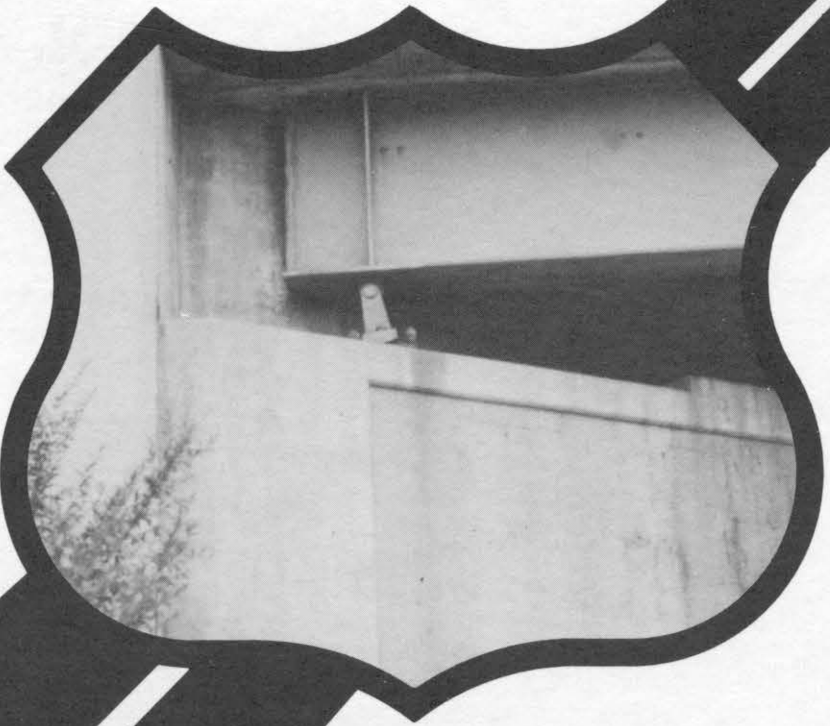
U.S. Department of Transportation

Federal Highway Administration

Report No.
FHWA-TS-85-228

Final Report
March 1986



FOREWORD

This technology sharing report summarizes a comprehensive research study on Tolerable Movement Criteria for Highway Bridges, which is presented in a three-volume interim report and in a one-volume final report.

This report will be of interest primarily to geotechnical and structural engineers involved in the design and construction of bridge foundations. It should be noted that the analytical studies and recommendations contained in the report pertaining to acceptable differential settlement of steel bridges represent only the conclusions of the researchers based on their research results. These conclusions should not be applied without careful evaluation of various conditions concerning the materials, structure, and foundation of the bridge, as well as the environmental conditions.

Sufficient copies of this report are being distributed to provide a minimum of two copies to each FHWA Regional and Division office and four copies to each State highway agency. Direct distribution is being made to the Region and Division offices.

R. J. Betsold
Director, Office of Implementation
Federal Highway Administration

NOTICE

Technical Report Documentation Page

1. Report No. FHWA-TS-85-228	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Tolerable Movement Criteria for Highway Bridges		5. Report Date February 1986
		6. Performing Organization Code HRT-10
7. Author(s) Lyle K. Moulton		8. Performing Organization Report No. CES-8401
9. Performing Organization Name and Address Department of Civil Engineering, West Virginia University, P.O. Box 6101, Morgantown, West Virginia 26506-6101		10. Work Unit No. (TRAIS) 35P1-072
		11. Contract or Grant No. DTFH61-82-P-30114
12. Sponsoring Agency Name and Address Federal Highway Administration, U.S. Department of Transportation, Washington, D.C. 20590		13. Type of Report and Period Covered Technology Sharing Report August 1, 1982 - March 31, 1984
		14. Sponsoring Agency Code

15. Supplementary Notes

FHWA Contract Officer's Technical Representative - Chien-Tan Chang, HRT-10
Technical Review by: Albert F. DiMillio, HNR-30

16. Abstract

This technology sharing report summarizes a comprehensive program of research on the tolerable movement of highway bridges that was begun in 1978 and completed in 1983. This investigation included (a) a state-of-the-art assessment of tolerable bridge movements based on a literature review, an appraisal of existing design specifications and practice, the collection and analysis of field data on foundation movements, structural damage and the tolerance to movements for a large number of bridges (314) in the United States and Canada, and an appraisal of the reliability of the methods currently used for settlement prediction; (b) a series of analytical studies to evaluate the effect of different magnitudes and rates of differential movement on the potential level of distress produced in a wide variety of steel and concrete bridge structures of different span lengths and stiffnesses; and (c) the development of a methodology for the design of bridges and their foundations that embodies a rational set of criteria for tolerable bridge movements.

17. Key Words Abutments, Bridges, Deformations, Design, Foundations, Piers, Settlement, Structural Damage, Tolerable Movements	18. Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22161

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	93	

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INTRODUCTION

Background

A great deal of data has been collected over the years relative to the effect of differential settlements on buildings and industrial structures. These data have been used to establish limits on the movements that are considered tolerable. These tolerable movement criteria are then used in conjunction with appropriate geotechnical and structural analyses to decide upon how the structure should be designed and founded in order to tolerate any anticipated movements safely and economically. Unfortunately, similar tolerable movement criteria and the acccompanying design methodology have not been available for highway bridges.

Because of the lack of well founded criteria for the tolerable movements of bridges, designers are commonly forced to rely on seemingly conservative rules of thumb or other guidelines contained in textbooks, building codes or specifications. One such rule of thumb requires that all continuous bridges be founded on rock or piles. The current AASHTO "Standard Specifications for Highway Bridges" states, "In general, piling shall be considered when footings cannot, at a reasonable expense be founded on rock or other solid material." The employment of these guidelines in practice has often led to the decision to use piling or other costly deep foundations without the consideration of other design alternatives that might have resulted in satisfactory performance at a lower overall cost. In fact, the majority of State highway agencies in the United States rely almost exclusively upon pile foundations to support their highway bridges, when these structures cannot be founded directly upon rock.

It was recognition of the need for the development of criteria for determining whether a proposed bridge can tolerate the estimated total and differential movements to which it may be subjected that led the Federal Highway Administration to sponsor the research described in this report. This comprehensive program of study was performed in the Department of Civil Engineering at West Virginia University. The research was initiated in 1978 and was completed in 1983.

Description of the Study

Although the research was divided into a substantial number of formal tasks and subtasks, basically, the work fell into three general study categories: (a) a state-of-the-art assessment of tolerable bridge movements based on a literature review, an appraisal of existing design specifications and practice, the collection and analysis of field data on foundation movements, structural damage and the tolerance to movements for a large number of bridges in the United States and Canada, and an appraisal of the reliability of the methods currently used for settlement prediction; (b) a series of analytical studies to evaluate the effect of different magnitudes and rates of differential movement on the potential level of distress produced in a wide variety of steel and concrete bridge structures of different span lengths and stiffnesses; and (c) the development of a methodology for the design of bridges and their foundations that would embody a rational set of criteria for tolerable bridge movements.

Project Research Reports

The results of the research described above have been presented in a three volume Interim Report, completed in 1981, and a Final Report completed in 1984. Volume I of the Interim Report (Report No. FHWA/RD-81/162) described the entire program of research and presented the results that had been obtained up to December of 1981. Volume II, Appendix A, (Report No. FHWA/RD-81/163) contains a description of the computerized "Bridge Data Storage and Retrieval System" that was developed in order to manage the large amount of data that was accumulated during the field studies. Volume III, Appendix B, (Report No. FHWA/RD-81/164) presents a very detailed "Analysis of Bridge Movements and Their Effects" that was conducted during the early stages of the field studies in order to identify those substructure and superstructure variables, or combinations of variables, that had the most important effect on bridge movements and their potential for producing structural damage. The Final Report (Report No. FHWA/RD-84/026) represents a comprehensive summary of the whole program of research and its results through its completion in December of 1983.

STATE-OF-THE-ART ASSESSMENT OF TOLERABLE BRIDGE MOVEMENTS

Literature Review

The rather comprehensive literature review performed during this study resulted in the collection of a substantial number of references dealing with the investigation of bridge approach embankments and bridge foundation movements. However, it was found that until relatively recently there was virtually nothing of a specific nature in the literature with respect to the tolerable movement of bridges. In 1978, the results of a 1975 survey of bridge movements and their effects, conducted by Transportation Research Board Committee A2K03 (Foundations of Bridges and Other Structures), were published in a series of four papers. Although three of these papers suggested criteria for tolerable vertical movements and two suggested criteria for tolerable horizontal movements, the suggested criteria were very general in nature and did not include consideration of the bridge type, width, span length and type of movement (i.e. total or differential). Thus, in spite of the pioneering efforts of Transportation Research Board Committee A2K03 and the large amount of data it collected on the influence of movements on the performance of bridge structures, no well-defined set of criteria for tolerable bridge movements was generally agreed upon.

Existing Design Specifications and Practice

In an effort to establish the extent to which existing design specifications and practice address the issue of tolerable bridge movements, a detailed review was made of the existing AASHTO "Standard Specifications for Highway Bridges", and current design practices were discussed with a number of State highway bridge engineers around the country, both by telephone and through personal interviews. It was found that the current AASHTO Specifications do not include any provisions or criteria for incorporating consideration of tolerance to foundation movements into the design of highway bridges. A proposal to include some limited consideration of differential settlement stresses, when they exceeded tolerable limits, was introduced at the four regional meetings of the AASHTO Subcommittee on Bridges and Structures during the spring of 1982. However, this proposal was not adopted, but was referred to the Technical Committee on Loads and Load Distribution for further consideration. The discussions with state highway bridge engineers revealed that, although a relatively small number of highway agencies do design their bridges to accomodate differential settlements, the majority employ pile foundations as a means of minimizing possible substructure movements. In general, the design practices of those States surveyed do not include the consideration of any tolerable movement criteria in the design of their bridges.

Field Studies

The process of collecting field data on bridge movements and their effects began with the acquisition of the 1967 and 1975 survey data in the files of Transportation Research Board Committee A2K03. In order to supplement and expand the scope of these data, various highway agencies were asked to supply additional information, including boring logs, settlement data, as-built plans and tolerance ratings for those bridges included in the original surveys, as well as complete information on any bridges that had experienced movements that were not included in the 1967 and 1975 survey responses. As a result, data were assembled for a total of 314 bridges distributed across 39 States, the District of Columbia and 4 Canadian provinces, including as-built plans for 115 of these structures. During this data collection process, field trips were made to the States of Connecticut, Ohio, Maine, Michigan, South Carolina, Utah and Washington. During these visits, bridge foundation design and performance were discussed with cognizant State officials, and selected bridges were visited and photographed in Connecticut, Maine, Utah and Washington.

The evaluation of the collected field data involved, in effect, three separate analyses, each with a somewhat different methodology. The first analysis involved the investigation of the influence on bridge abutment and pier movements of substructure variables such as (a) general soil conditions, (b) type of abutment (full height, perched or spill-through), (c) type of foundation (spread footings or piles), (d) height of approach embankment, and (e) abutment-embankment-pier geometry. It was also possible, as part of this analysis, to identify the most common causes of foundation movements for the bridges studied and to explore the influence of construction sequence and precompression on abutment movements. The second analysis involved the investigation of the influence of bridge foundation movements on the bridge structure in an effort to determine what types and magnitudes of movements most frequently result in detrimental structural damage. The variables considered in this analysis were: (a) type of movement (vertical only, horizontal only, or vertical and horizontal in combination), (b) magnitude of movements (maximum differential vertical movements between two successive abutments or piers and maximum horizontal movements), (c) span type, (d) type of structural material (steel or concrete), (e) number of spans and (f) abutment type. The third analysis involved the investigation of the tolerance of the various bridge structures to movements and considered such variables as (a) type of movement, (b) magnitude of movements, (c) span type, (d) type of structural material, (e) number of spans and (f) abutment type.

Movements of Foundation Elements

Abutments. There were a total of 580 abutments which had sufficient data to be included in the analysis. Over three-quarters of these (439) experienced some type of movement. A general summary of the movement data for these abutments is presented in table 1. The frequency of occurence

Table 1. General summary of abutment movements.

Movement Type	Frequency		Magnitude of Movements	
	Number of Abutments	Percent Moved	Range in Inches	Average in Inches
All Types	439	100.0		
Vertical	379	86.3	0.03 - 50.4	3.7
Horizontal	138	31.4	0.1 - 14.4	2.6
Vertical &	77	17.5	0.1 - 50.4	6.9
Horizontal			0.1 - 14.4	2.2

Note: 1 inch = 25.4 mm.

of the various ranges of vertical and horizontal abutment movements is illustrated in figure 1. These data show that the great majority of abutments that moved experienced vertical movement, less than one-third moved horizontally, and a substantial number moved both vertically and horizontally. The magnitudes of the vertical movements tended to be substantially greater than the horizontal movements, especially when vertical and horizontal movements occurred simultaneously. Although the majority of the abutments that experienced horizontal displacement moved inward, many becoming jammed against the beams or girders (See figures 2 and 3), a substantial number of abutments (a total of 39) moved outward away from the bridge superstructure and toward their approach embankments (See figures 4 and 5). These were almost invariably perched abutments founded on piles driven through approach fill placed over deep compressible soils.

Of those abutments with sufficient data to be included in the analysis, substantially more perched abutments were reported than either full height or spill-through abutments, as illustrated in figure 6. Although these data show that more full height abutments experienced movements than spill-through abutments, both the range and average magnitude of the movements of the spill-through abutments were greater than for the full height abutments. This was true with respect to both vertical and horizontal movements. The large number of perched abutments that moved suggests that greater attention needs to be directed to the design and construction of the foundation systems for this type of abutment. In this connection, it was also found that the construction sequence and/or the use of precompression exerted a significant influence on the movements of perched abutments founded on spread footings on fill. This is illustrated in table 2, which shows that the range and average magnitude of abutment movements were substantially lower when a preload and/or waiting period (from one to six months) was employed prior to construction of the abutments than when the abutments were constructed immediately following completion of the embankments.

In terms of foundation type, more abutments on spread footings were reported to have moved than abutments founded on piles. However, as shown in table 3, abutments founded on piles actually experienced larger ranges

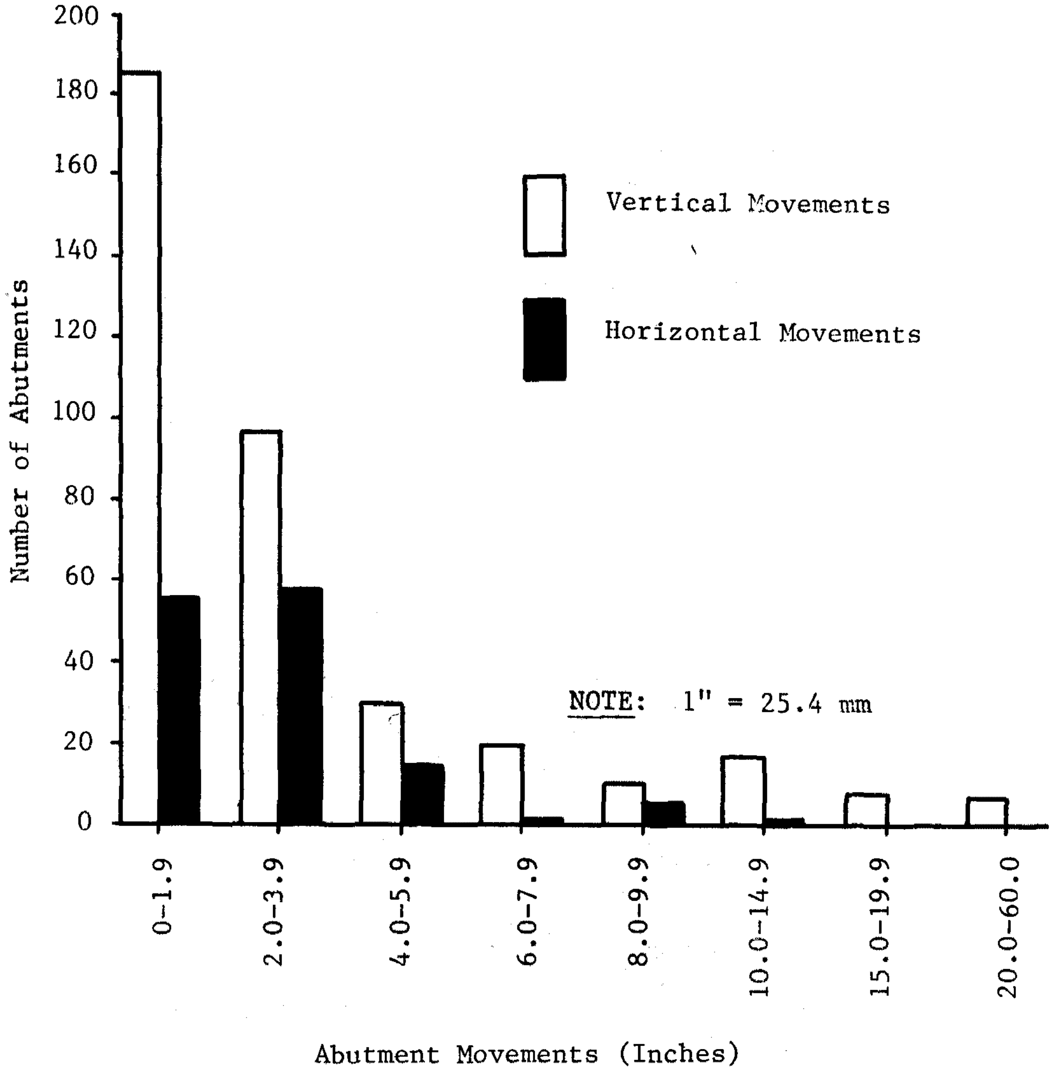


Figure 1. Frequency of occurrence of various ranges of vertical and horizontal abutment movements.

Figure 2. Illustration of inward horizontal displacement leading to abutment being jammed against beams.

Figure 3. Backwall of abutment jammed against beam as result of inward horizontal movement of abutment.

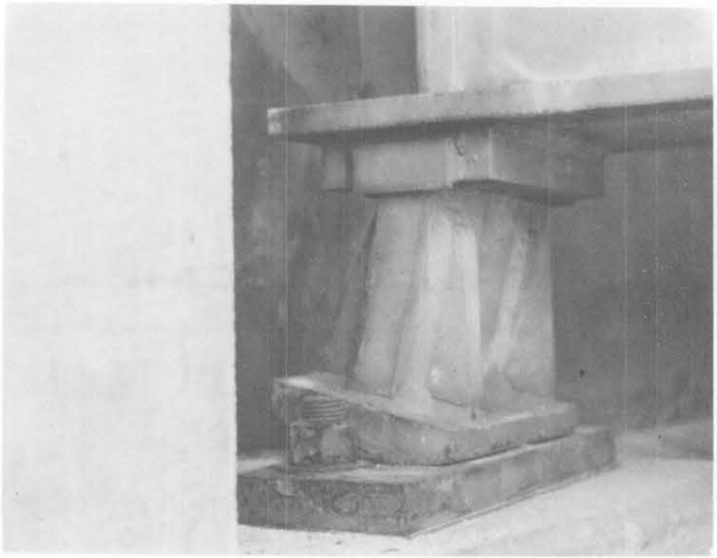


Figure 4. Tilted rocker caused by backward horizontal displacement of abutment.

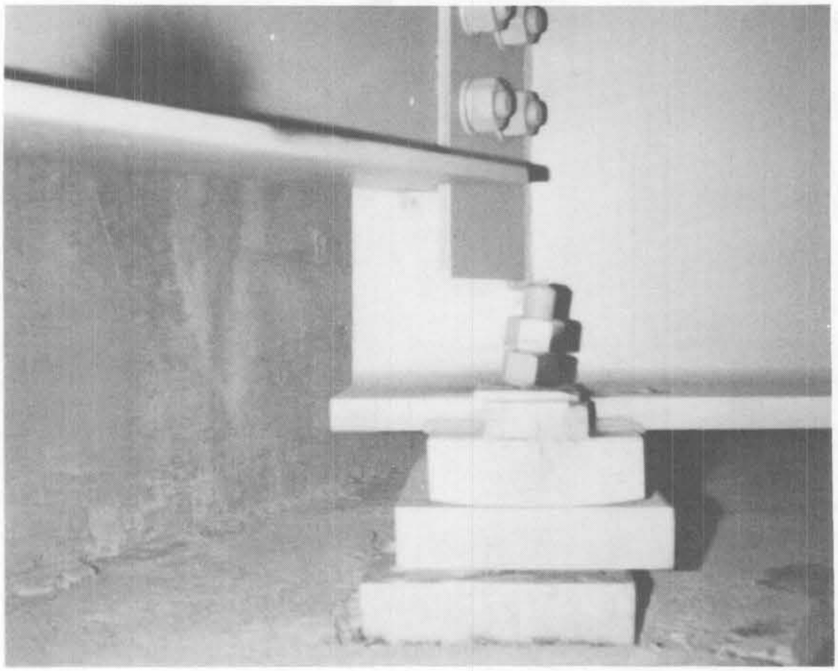


Figure 5. Displaced bearing and tilted anchor bolt caused by backward horizontal displacement of abutment.

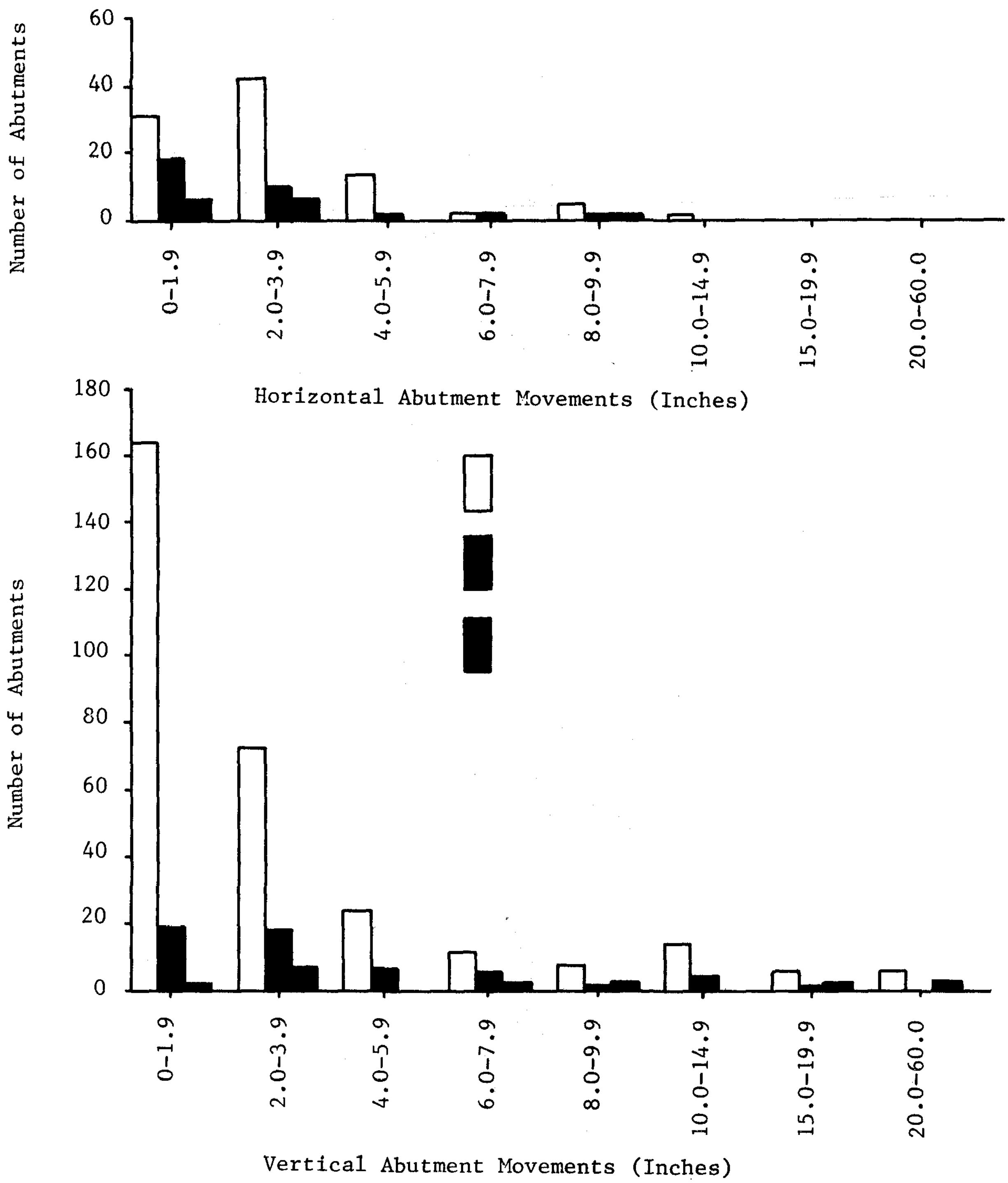


Figure 6. Frequency of occurrence of various ranges of vertical and horizontal abutment movements in terms of abutment type.

Table 2. Summary of movements of perched abutments on spread footings on fill in terms of construction sequence.

		Frequency		Magnitude of Movements	
Construction Sequence	Movement Type	Number of Abutments	Percent Moved	Range in Inches	Average in Inches
Preload and/or Waiting Period	All Types	81	100.0		
	Vertical	81	100.0	0.2-5.2	1.8
	Horizontal	2	2.5	0.3-0.3	0.3
	Vertical &	2	2.5	4.0-5.0	4.5
	Horizontal			0.3-0.3	0.3
No Preload or Waiting Period	All Types	63	100.0		
	Vertical	60	95.2	0.1-35.0	7.3
	Horizontal	13	20.6	0.3- 5.0	3.5
	Vertical &	10	15.0	0.1-35.0	18.2
	Horizontal			0.3- 5.0	3.7

Note: 1 inch = 25.4 mm.

Table 3. Summary of abutment movements in terms of foundation type.

		Frequency		Magnitude of Movements	
Foundation Type	Movement Type	Number of Abutments	Percent Moved	Range in Inches	Average in Inches
Spread Footings	All Types	266	100.0		
	Vertical	254	95.5	0.1-35.0	3.7
	Horizontal	40	15.0	0.1-8.8	2.4
	Vertical &	28	10.5	0.1-35.0	6.1
	Horizontal			0.1-8.0	2.2
Piles	All Types	173	100.0		
	Vertical	122	70.5	0.03-50.4	3.9
	Horizontal	99	57.2	0.3-14.4	2.7
	Vertical &	48	27.7	0.3-50.4	5.6
	Horizontal			0.3-14.4	2.3

Note: 1 inch = 25.4 mm.

and slightly larger average vertical and horizontal movements than did those founded on spread footings. These findings, coupled with the relatively large number of pile supported abutments that did move, tends to suggest that the mere use of pile foundations does not necessarily guarantee that abutment movements will be within tolerable limits, particularly for the case of perched abutments on fills.

With respect to foundation soil type, as might have been expected, the largest vertical and horizontal abutment movements were most commonly associated with soil profiles containing substantial quantities of fine grained soils.

Although there was a general trend toward increasing magnitudes of vertical movements of abutments with increase in the height of their approach embankments, the correlation was not particularly good, and little evidence of other meaningful trends was observed.

Piers. The results of the analysis of pier movements showed that piers moved less often than abutments, with only about twenty five percent of the 1068 piers considered in the analysis having experienced any movements. Moreover, the general summary of the pier movements, presented in table 4, shows that the average vertical pier movements tended to be smaller than for abutments. However, unlike the abutments, average horizontal pier movements tended to be larger than the vertical movements.

Table 4. General summary of pier movements.

	Frequency		Magnitude of Movements	
Movement Type	Number of Piers	Percent Moved	Range in Inches	Average in Inches
All Types	269	100.0		
Vertical	234	87.0	0.03-42.0	2.5
Horizontal	52	19.3	0.1 -20.0	3.3
Vertical &	17	6.3	0.3- 13.7	5.1
Horizontal			0.6- 20.0	2.7

Note: 1 inch = 25.4 mm.

In terms of foundation type, it was found that, like the abutments, the piers founded on piles experienced larger average vertical and horizontal movements than those founded on spread footings. However, there were substantially more spread footing foundations that moved than pile foundations, and the range of their vertical movements was greater.

Very few trends were observed with regard to pier movements in terms of soils and foundation conditions. As would be expected, the most frequent movements for both spread footings and pile foundations were associated with fine grained foundation soils.

Piers located in or near the toe of approach embankments experienced movement more than twice as frequently as piers that were located away from the embankment. Moreover, both the range and average magnitude of horizontal movements for piers located in or near the toe of approach embankments was much greater than for piers located away from the embankment, suggesting that more consideration needs to be given to the higher level of horizontal stresses that exist in these areas.

Causes of Foundation Movements. The investigation of the influence of substructure variables on bridge abutment and pier movements also resulted in the identification of the cause or causes of these movements. The primary causes of substructure movements usually fell into three general categories: (a) movements of approach embankments and/or their foundations; (b) unsatisfactory performance of pile foundations; and (c) inadequate resistance to lateral earth pressures, causing horizontal movements of abutments.

The movements of approach embankments were commonly caused by (a) consolidation settlements of compressible foundation soils underlying the embankments, (b) post construction settlements of the embankments themselves, or (c) sliding caused by slope or foundation instability. Among the most commonly identified conditions that led to slope or foundation instability were excessively steep slopes, low shear strength of embankment or underlying foundation soils and streambed scour at the toe of slope. The movements of perched and spill-through abutments, which were caused by movements of approach embankments, were not limited to those abutments founded on spread footings, but included a substantial number of pile supported abutments.

Except for those cases where the movements of perched and spill-through abutments were caused by embankment sliding, the available data were not generally sufficient to explain the unsatisfactory performance of many pile foundations or the inadequate resistance of some abutments to imposed lateral earth pressures.

Influence of Foundation Movements on Bridge Structures

Types of Damage. A general summary of the types of structural damage and the numbers of bridges that were reported to have experienced these is presented in table 5. Although most of the terms used to describe

Table 5. General Summary of Data on Structural Damage.

Type of Structural Damage	Number of Bridges
Damage to Abutments	69
Damage to Piers	18
Vertical Displacement	45
Horizontal Displacement	68
Distress in the Superstructure	117
Damage to Rails, Curbs, Sidewalks, Parapets	30
Damage to Bearings	34
Poor Riding Quality	12
Not Given/Corrected During Construction	10
None	81

structural damage in table 5 are self-explanatory, some explanation is required for the terms "vertical displacement", "horizontal displacement", "distress in the superstructure" and "damage to bearings". The term "vertical displacement", when applied to structural damage, includes the raising or lowering of the superstructure above or below planned grade or a sag or heave in the deck. This category of damage would also be applied to those structures that required shimming or jacking to restore them to a satisfactory grade. The term "horizontal displacement", when applied to structural damage, includes the misalignment of bearings and the superstructure or beams jammed against the abutments. Also included in this category of damage are cases where the superstructure extended beyond the abutment, where beams or girders required cutting, or where there was a horizontal movement of the floor system. "Distress in the superstructure" consists of cracks or other evidence of excessive stress in beams, girders, struts and diaphragms, as well as cracking and spalling of the deck. Other types of damage included in this category are the opening, closing or damage to deck joints and cases where the cutting of relief joints were required. "Damage to bearings" includes the tilting or jamming of rockers as well as cases where rockers have pulled off bearings, or where movement resulted in an improper fit between bearing shoes and rockers requiring repositioning. Also included in this category are deformed elastomeric bearing pads, sheared anchor bolts in the bearing shoes and the cracking of concrete at the bearings.

Influencing Factors and Their Effects. The influence of the type of movement on observed structural damage is illustrated in figure 7, which shows that most types of structural damage are associated either with horizontal movements or with horizontal and vertical movements occurring simultaneously. In contrast, those bridges for which only vertical movement was reported had the lowest frequency of damaging structural effects, with over forty percent having no damage at all. These same general trends were observed in terms of the magnitude of movements, in that even moderate differential vertical movements tended to produce a relatively low incidence of structural damage, while relatively small horizontal movements produced a high frequency of damaging structural effects. For example, of the 155 bridges with maximum differential settlements of less than 4 inches (101.5 mm), 79 (51 percent) experienced no damage whatsoever. The majority of the remaining 76 structures experienced primarily abutment damage, in the form of minor cracking, opening or closing of construction joints etc., and relatively minor distress in the superstructure (See figures 8 and 9). In contrast, for those bridges that experienced horizontal movements alone, movements of from 1.0 to 2.0 inches (25.4 to 50.8 mm) quite commonly caused distress in the superstructure, occurring in more than two-thirds of the cases. The bearings were also effected in more than a third of these structures. Although it was more difficult to correlate structural damage with

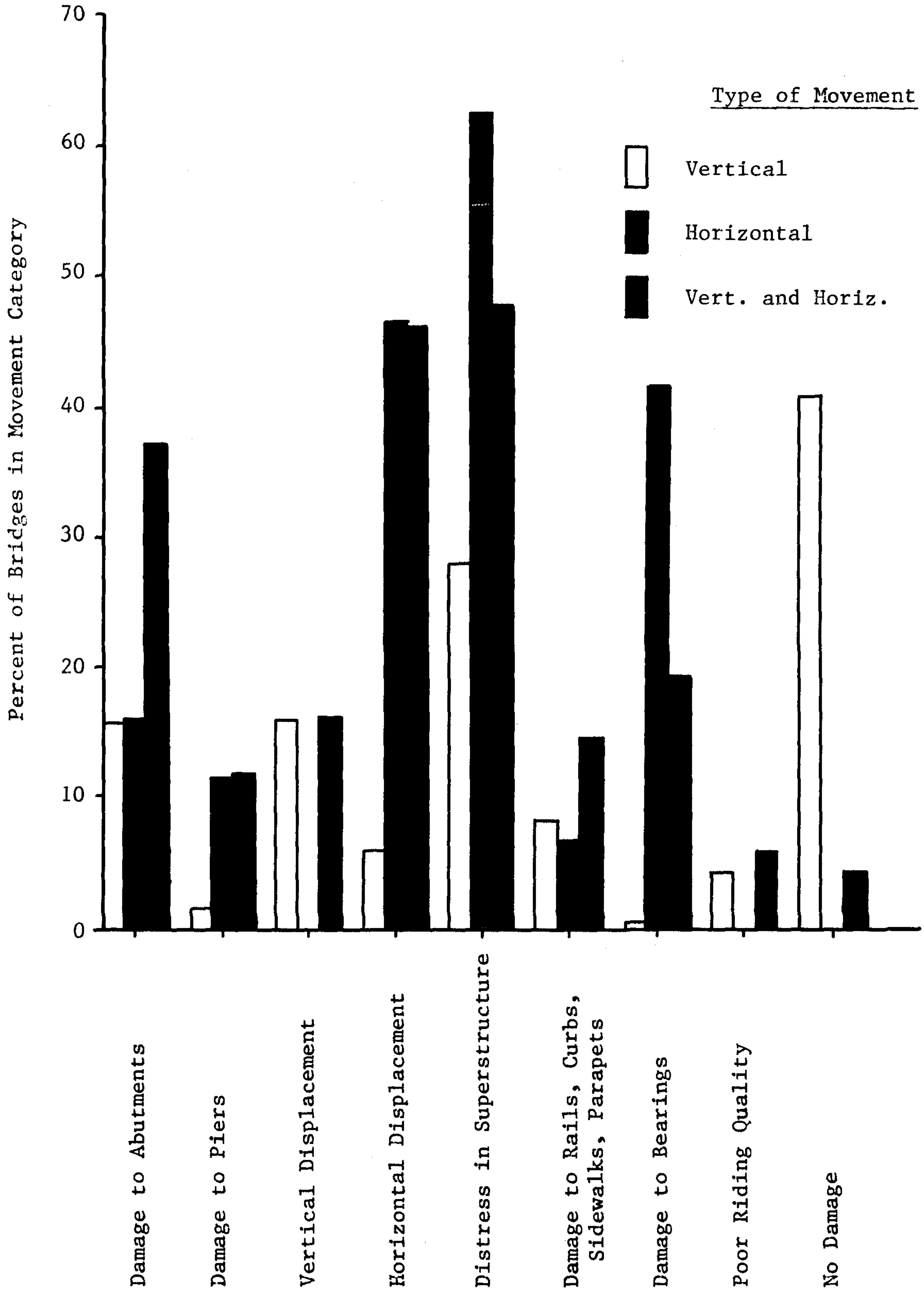


Figure 7. Types of structural damage associated with types of foundation movements.

Figure 8. Minor cracking in abutment caused by differential settlement.

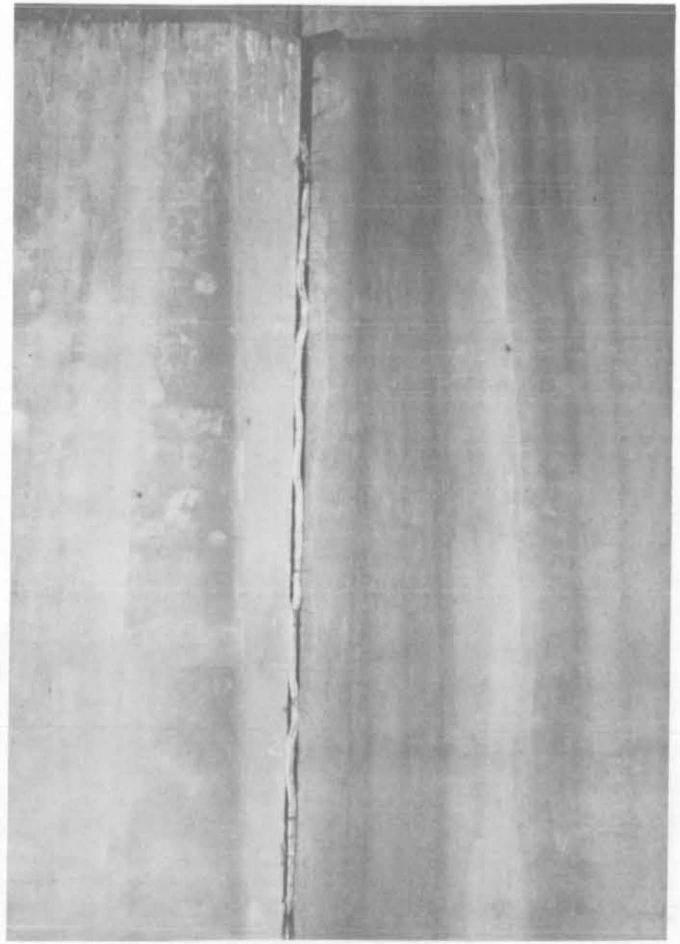


Figure 9. Opening of construction joint in abutment as result of differential settlement.

magnitudes of substructure movements for those cases where vertical and horizontal movements occurred simultaneously, a detailed review of the actual causes of the various types of distress in the bridges revealed that it was most commonly the horizontal component of the movement that was responsible for the reported damage.

In terms of span type, the data showed that distress in the superstructure was the most common structural effect reported for both continuous and simply supported bridges, although this type of distress was reported for 43.5 percent of the simply supported bridges and only 31.2 percent of the continuous bridges. The data also showed that abutment damage and horizontal displacement were the second most common effects for the simply supported bridges, occurring in 30.4 and 27.2 percent of the cases, respectively, while these types of damages were reported for only 14.2 and 18.8 percent, respectively, of the continuous bridges. Moreover, 37.0 percent of the continuous bridges experienced no damage while only 15.2 percent of the simply supported bridges were reported to be undamaged. Thus, contrary to what might have been expected, it appears that the continuous bridges were less susceptible to many types of structural damage as a result of substructure movements than were the simply supported bridges. For both types of spans, however, the most frequent and most serious type of structural distress seemed to be related to horizontal movements.

The data on the frequency of occurrence of the various types of structural damage in terms of type of construction material showed that, in general, the concrete bridges seemed to be more susceptible to distress in the superstructure caused by foundation movements than did the steel bridges. In fact, the data show that the steel bridges, with differential vertical movement alone, had a lower overall incidence and severity of structural damage than did the concrete bridges. Of the 117 steel bridges which experienced only vertical movements, only 16.2 percent experienced distress in the superstructure, while this type of damage was reported in 50.9 percent of the 57 concrete bridges with this same type of movement. In addition, there were substantially more steel bridges that were undamaged by vertical differential movements. Nevertheless, there were a substantial number of concrete bridges that were subjected to moderate differential settlements without experiencing any structural damage at all.

Two such bridges are shown in figures 10 through 14. Again it was found that even relatively small horizontal movements, on the order of 2 inches (50.8 mm), produced more frequent and more severe structural damage than did larger differential vertical movements, regardless of type of structural material.

Relatively few positive conclusions can be drawn with respect to the influence of number of bridge spans on the effects produced by foundation movements. However, the data do tend to indicate that multispan structures had a higher frequency of more severe structural effects produced by foundation movements than did single span bridges.

Although those bridges with perched abutments, in general, had the highest occurrence of the more serious types of structural damage, they also had, by far, the largest number that experienced no structural damage. A detailed examination of the data showed that it was primarily differential vertical movement in excess of 4 inches (101.6 mm) that caused damage to these bridges with perched abutments. However, the most damaging effects were produced by horizontal movements between one inch (25.4 mm) and 4 inches (101.6 mm) in magnitude, and these effects were most serious when these horizontal movements were accompanied by larger vertical differential movements, i.e. differential settlements in excess of 4 inches (101.6 mm).

Tolerance of Bridges to Foundation Movements

Although the term "tolerable" is subjective to some extent, in that foundation movements that might be considered to be tolerable by one engineer may be considered intolerable by another, for the purpose of this study it was necessary to adopt some consistent definition of tolerable movement in order to remove some of this subjectivity. Thus, the following definition used by Transportation Research Board Committee A2K03 was adopted: "Movement is not tolerable if damage requires costly maintenance and/or repairs and a more expensive construction to avoid this would have been preferable".

Tolerance in Terms of Structural Damage. Overall, of the 280 structures where data on tolerance to foundation movements were available

Figure 10. Continuous concrete box girder bridge-left abutment settled approximately 1.5 inches (38.1 mm) relative to the pier.

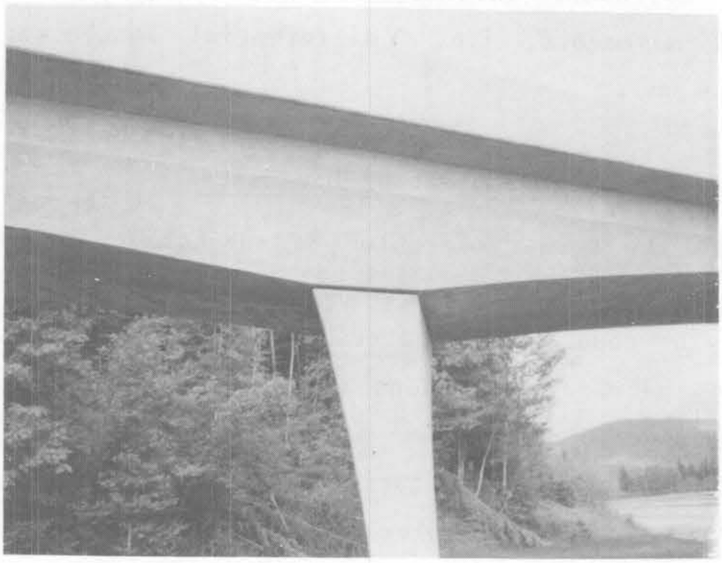
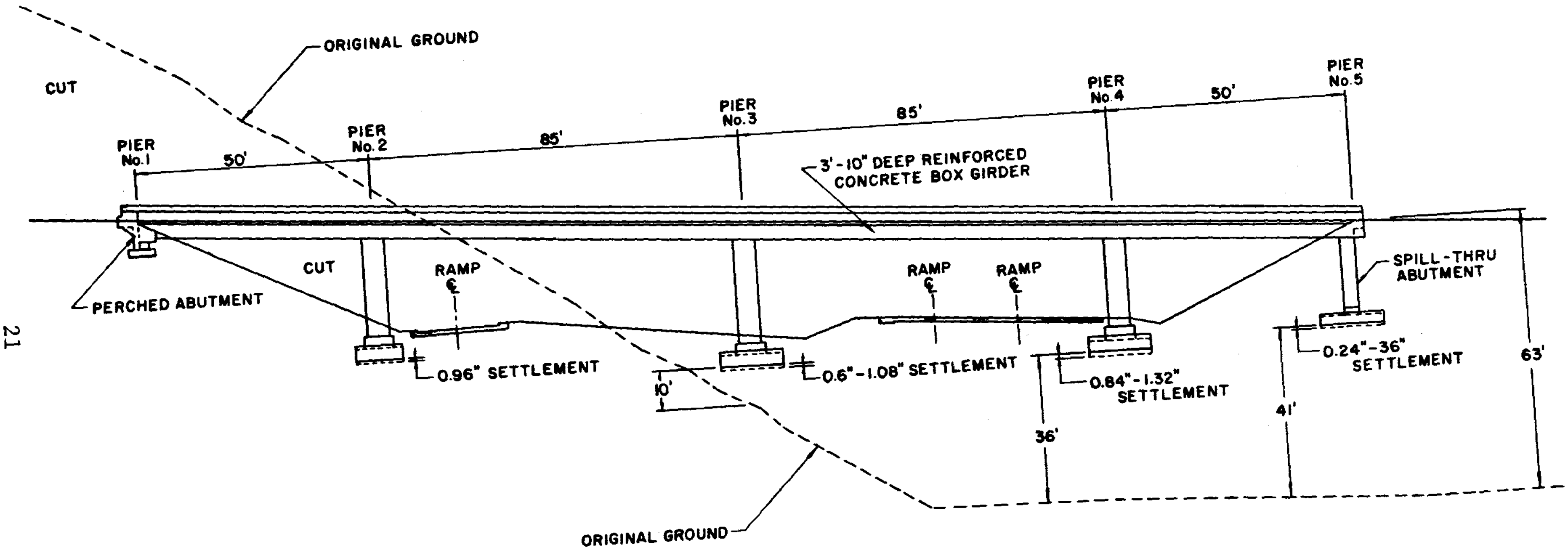


Figure 11. Closeup of bridge shown in figure 10 showing no signs of distress in spite of differential settlement of abutment.



NOTES: PIERS No. 1 & 2 FOUNDED ON SPREAD FOOTINGS ON NATURAL GROUND WITH MAXIMUM ALLOWABLE BEARING PRESSURE OF 5 TONS/SQ. FT.
PIERS No.3,4, AND 5 FOUNDED ON SPREAD FOOTINGS ON FILL WITH MAXIMUM ALLOWABLE BEARING PRESSURE OF 3 TONS/SQ. FT.
1 INCH = 25.4 mm., 1 FOOT = 0.305 METERS

Figure 12. Elevation view of curved concrete box girder bridge showing the settlements experienced following construction.

Figure 13. View of side of curved concrete box girder bridge shown in figure 12 showing no signs of distress in spite of the settlements that took place.

Figure 14. View of bottom of curved concrete box girder bridge shown in figures 12 and 13 showing no signs of distress in spite of the settlements that took place.

or could reasonably be assumed, the movements were reported to be tolerable for 180 bridges and intolerable for 100. The tolerance of bridges to structural damage is illustrated in figure 15. These data show that, in the category of tolerable structural effects produced by foundation movements, damage to abutments and distress in the superstructure appear most frequently. In most instances, the reported damage involved relatively minor cracking and/or opening or closing of construction joints in the abutments, as shown in figures 8 and 9, and cracking and spalling of concrete decks. Of course, as would be expected, the foundation movements associated with all 81 bridges that experienced no structural damage were reported to be tolerable.

For those 100 bridges with intolerable movements, figure 15 shows that distress in the superstructure, vertical displacement and horizontal displacement occurred most frequently. In addition, abutment damage and damage to bearings appeared in a substantial number of cases. A detailed study of these bridge damage data revealed that, in the majority of the cases, there was a direct interrelationship between these most frequently occurring categories of structural damage, and that most were related to horizontal movements or horizontal movements in combination with vertical movements. Although there were a variety of damaging incidents reported, by far the most frequently occurring sequence of events involved the inward

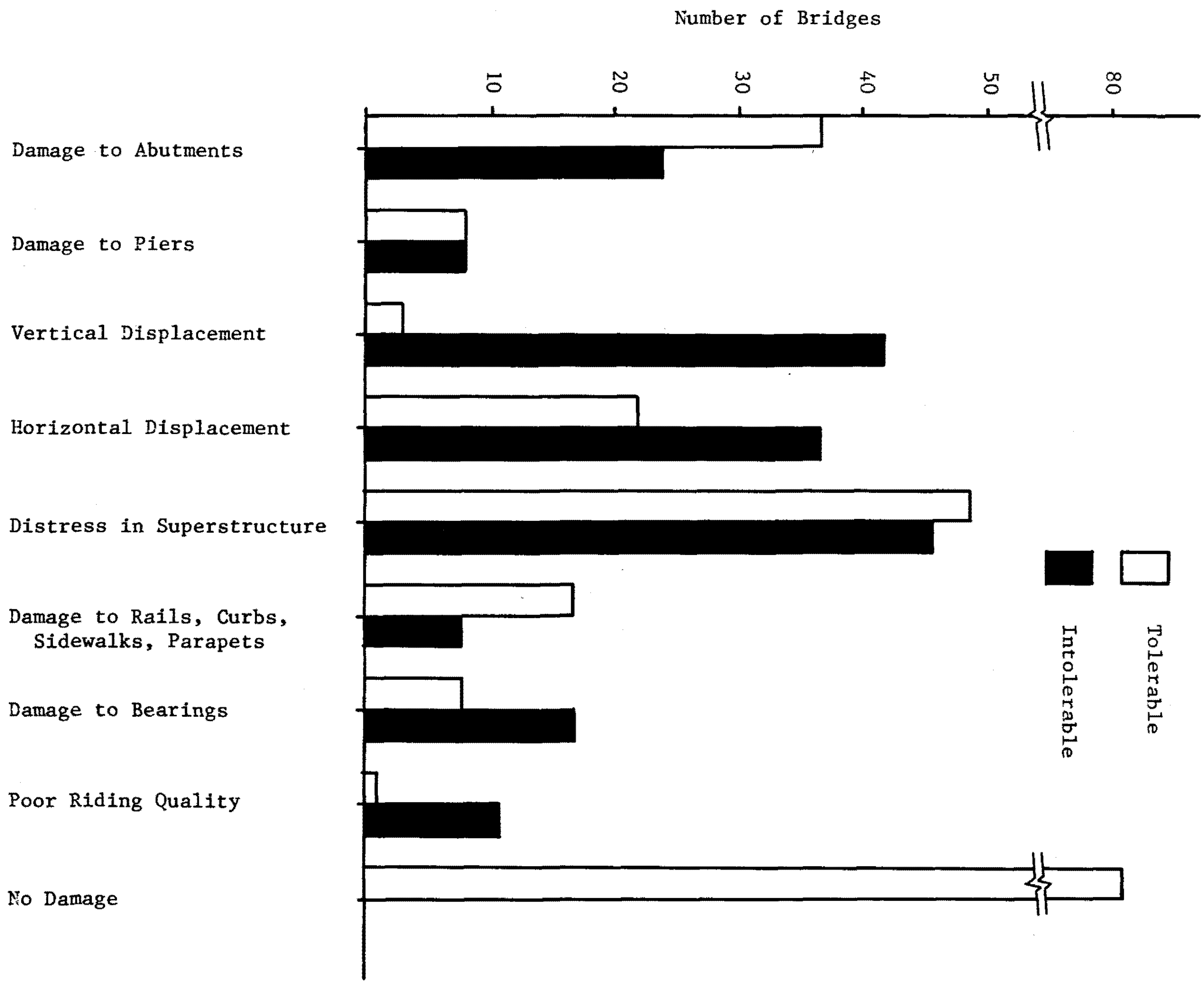


Figure 15. Tolerance of bridges to structural damage.

Table 6. Range of movement magnitudes considered tolerable or intolerable.

Interval[a] in Inches	Number of Bridges with the Given Type of Movement							
					Vertical and Horizontal Component			
	Vertical Only		Horizontal Only		Vertical Component		Horizontal Component	
	Tol.	Intol.	Tol.	Intol.	Tol.	Intol.	Tol.	Intol.
0.0 - 0.9	52	0	3	0	9	1	8	0
1.0 - 1.9	40	2	5	1	9	3	7	10
2.0 - 3.9	33	10	1	10	6	4	8	10
4.0 - 5.9	1	8	2	0	2	5	0	8
6.0 - 7.9	3	5	1	3	0	2	0	1
8.0 - 9.9	0	5	0	3	0	2	0	2
10.0 - 14.9	2	5	0	2	0	6	0	2
15.0 - 19.9	1	4	0	0	0	3	0	1
20.0 - 60.0	0	0	0	0	0	4	1	0
Total	132	39	12	19	26	30	24	34

[a]For vertical moments, magnitudes refer to maximum differential vertical movement. For horizontal movements, magnitudes refer to maximum horizontal movement of a single foundation element. Note: 1 inch = 25.4 mm.

vertical settlements less than 2.0 inches (50.8 mm) and 91.2 percent of those less than 4.0 inches (101.6 mm) were considered to be tolerable. However, although there were some larger differential vertical settlements that were considered tolerable, generally the tolerance to these settlements decreased significantly for values over 4 inches (101.6 mm). Thus, only 23.5 percent of the differential vertical settlements between 4 inches (101.6 mm) and 8.0 inches (203.2 mm) and 17.6 percent of those over 8 inches (203.2 mm) were reported as being tolerable.

In terms of horizontal movements alone, of those bridges with maximum movement less than 2.0 inches (50.8 mm), the movements were considered tolerable in 88.8 percent of the cases. However, a large majority (81.8 percent) of the maximum horizontal movements of 2.0 inches (50.8 mm) and greater were found to be intolerable. Furthermore, table 6 shows that even horizontal movements less than 2.0 inches (50.8 mm) were only reported as being tolerable in 60.0 percent of the cases, when accompanied by differential vertical movements. In fact, a more detailed analysis of the data revealed that for simultaneous horizontal and vertical movements of this type, the horizontal movements were only reported as being tolerable, in the great majority of the cases, when their magnitude approached one inch (29.4 mm) and less.

Tolerance in Terms of Span Length. In order to determine the influence of span length on the tolerance of bridges to foundation movements, the tolerance was evaluated in terms of maximum longitudinal angular distortion (differential vertical settlement divided by span length). There were 204 of the 280 bridges with tolerance data, where the data were sufficiently complete to permit this type of analysis. Of these 204 bridges, the movements were reported to be tolerable for 144 and intolerable for 60. A summary of the frequency of occurrence of the various ranges of magnitudes of angular distortion considered tolerable and intolerable for all of the bridges included in this portion of the study is presented in figure 16. The data in figure 16 suggest that an upper limit on the angular distortion that might be considered tolerable would be 0.004. In fact, the detailed data showed that 97.7 percent of the 44 angular distortions less than 0.001 and 94.4 percent of the 132 angular distortions less than 0.004 were considered to be tolerable. However, only

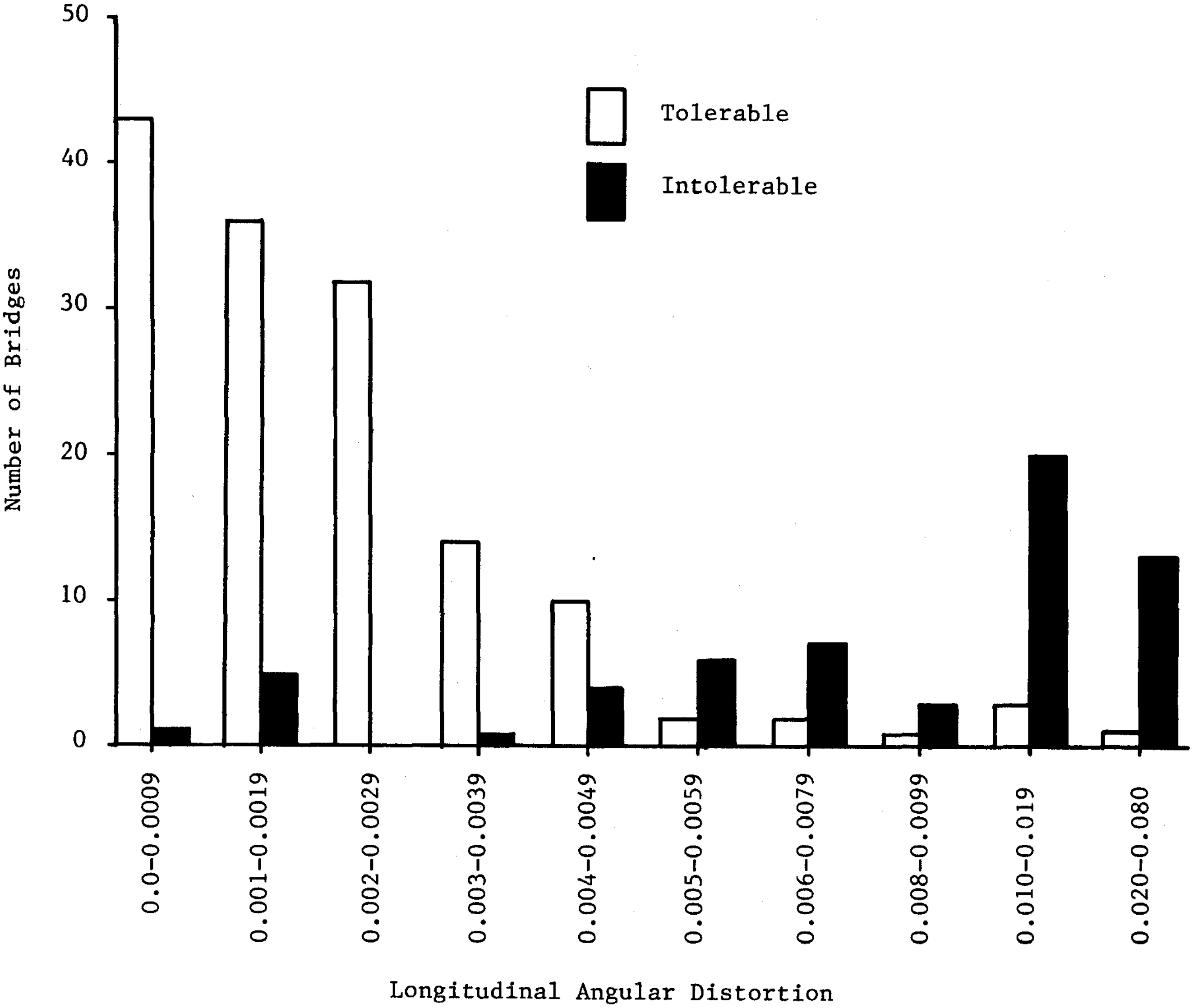


Figure 16. Ranges of magnitudes of longitudinal angular distortion considered tolerable or intolerable for all bridges.

42.9 percent of the values of angular distortion between 0.004 and 0.01, and 7.1 percent of those over 0.01, were considered to be tolerable.

Tolerance in Terms of Span Type. When the data in figure 16 were subdivided by span type, they showed that the simply supported bridges were less sensitive to angular distortion than the continuous bridges. While this result was expected, it was anticipated that there would be a more dramatic difference than actually occurred. For the continuous bridges, 93.7 percent of the angular distortions less than 0.004 were considered to be tolerable, while only 25.0 percent of those over 0.004 were considered to be tolerable. In contrast, for the simply supported bridges, 97.2 percent of the angular distortions less than 0.005 were reported as being tolerable, while only 20.0 percent of those over 0.005 were considered tolerable. Translated in terms of differential settlement, these data suggest that, for simply supported bridges, differential settlements of 3.0 inches (74.2 mm) and 6.0 inches (152.4 mm) would most probably be tolerable for spans of 50 feet (15.2 meters) and 100 feet (30.5 meters), respectively. However, for continuous bridges, it would appear that differential settlements of 2.4 inches (61.0 mm) and 4.8 inches (121.9 mm) would be more reasonable tolerable limits for spans of 50 and 100 feet (15.2 and 30.5 meters), respectively.

Tolerance in Terms of Material Type. The results of a breakdown of the data in figure 16 in terms of material type suggested that the concrete bridges might be slightly more tolerant to angular distortion than the steel bridges. For the concrete bridges, 97.4 of the angular distortions less than 0.005 were considered to be tolerable, while for the steel bridges, only 91.3 percent of the angular distortions less than 0.005 were reported to be tolerable. Thus, the reported trend for the concrete bridges to experience more frequent and more severe superstructure damage than the steel bridges as a result of foundation movements did not show up in the tolerance data. This implies that the frequently reported distress in the superstructure of concrete bridges was quite often judged to be tolerable. A detailed breakdown of the data in figure 15 in terms of material type provided verification for this observation.

Reliability of Settlement Predictions

One of the most common issues raised by the various bridge engineers, who were contacted throughout the course of this study, pertained to the reliability of the current methods used for predicting settlements. In an effort to address this issue, a detailed review of the literature was made to determine the state-of-the-art of settlement prediction for both granular and cohesive soils. A search was then made of the settlement records and soil properties data collected during the field studies, in an effort to select some case histories of bridge foundation movements that would permit a comparison to be made between measured and predicted settlements.

For granular soils, it was found that there are a wide variety of methods currently in use for settlement prediction. For the most part, these methods are either entirely empirical or they contain some elements of empiricism. It appears that the most popular of these methods fall in two general categories: (a) empirical methods based on the Terzaghi and Peck approach, with modifications by Teng, Meyerhof, Peck and Bazaraa, and other authors; and (b) semi-empirical methods, which are based on the theory of elasticity and use standard penetration test results, or the results of cone penetrometer tests, to estimate the elastic constants for the foundation soils. Although some very good agreement is reported in the literature between predicted and measured settlements of sands, efforts to compare the various settlement prediction methods for the same case history appearing in the literature were not particularly productive, either because of a lack of soil property data, loading data or both. However, overall the data extracted from the literature did indicate that the settlement of sands could usually be predicted within 50 percent of the measured value.

A review of the data collected for all 314 of the bridges included in the field studies revealed that there were no bridge foundations on granular soils where the data was sufficiently complete to permit a comparison between measured and predicted settlements. While this finding was disappointing, it should be pointed out that, from a practical standpoint, the reliability of prediction of the settlements of granular soils is substantially less important than that of cohesive soils as far as bridge foundations are concerned. This is because the settlement of granular soils occurs very rapidly, so that at each stage of loading during the process of bridge construction, the settlement is essentially completed before the next stage of loading is applied. Thus, adjustments in grade can be made during construction, and there are no post-construction settlements of significance to contend with.

For cohesive soils, it was found that, although there are some fairly sophisticated methods of settlement prediction available, most commonly these predictions are made with the Terzaghi theory of one-dimensional consolidation, using the Taylor modification for gradual rate of loading. In this method, the stress increases in the foundation soils, caused by the loads applied at the foundation level, are estimated using the theory of elasticity. The data extracted from the literature and that collected during the field studies for bridges founded on cohesive soils were sufficiently complete in a number of cases to permit the comparison of measured and predicted settlements. The results of two such comparisons are presented in figures 17 and 18.

Figure 17 shows the comparison between measured and calculated settlements beneath the center of the north abutment of the Main Street Connector bridge over Route 2 in East Hartford, Connecticut, for the first seven months following the start of construction. This bridge is a two-span simply supported structure founded on 13 feet (4.0 meters) of fine to medium sand underlain by 86 feet (26.2 meters) of varved clay. The final calculated north abutment settlement of 3.1 inches (7.9 cm) compared quite favorably with the final observed abutment settlement, which varied from 3.0 to 3.5 inches (7.6 to 8.9 cm).

Figure 18 shows the comparison between measured and calculated settlements beneath the center of the north abutment of the U.S. Route 1 bridge over the Boston and Maine Railroad at Wells, Maine, for the first 23 months following the start of construction. This bridge is a single span structure whose abutments are founded on approach embankments supported by reinforced earth, as shown in figure 19. The foundation soil consists of 30 feet (9.1 meters) of loose to medium dense sand overlying 50 feet (15.2 meters) of sensitive silty clay. The reinforced earth supported embankment

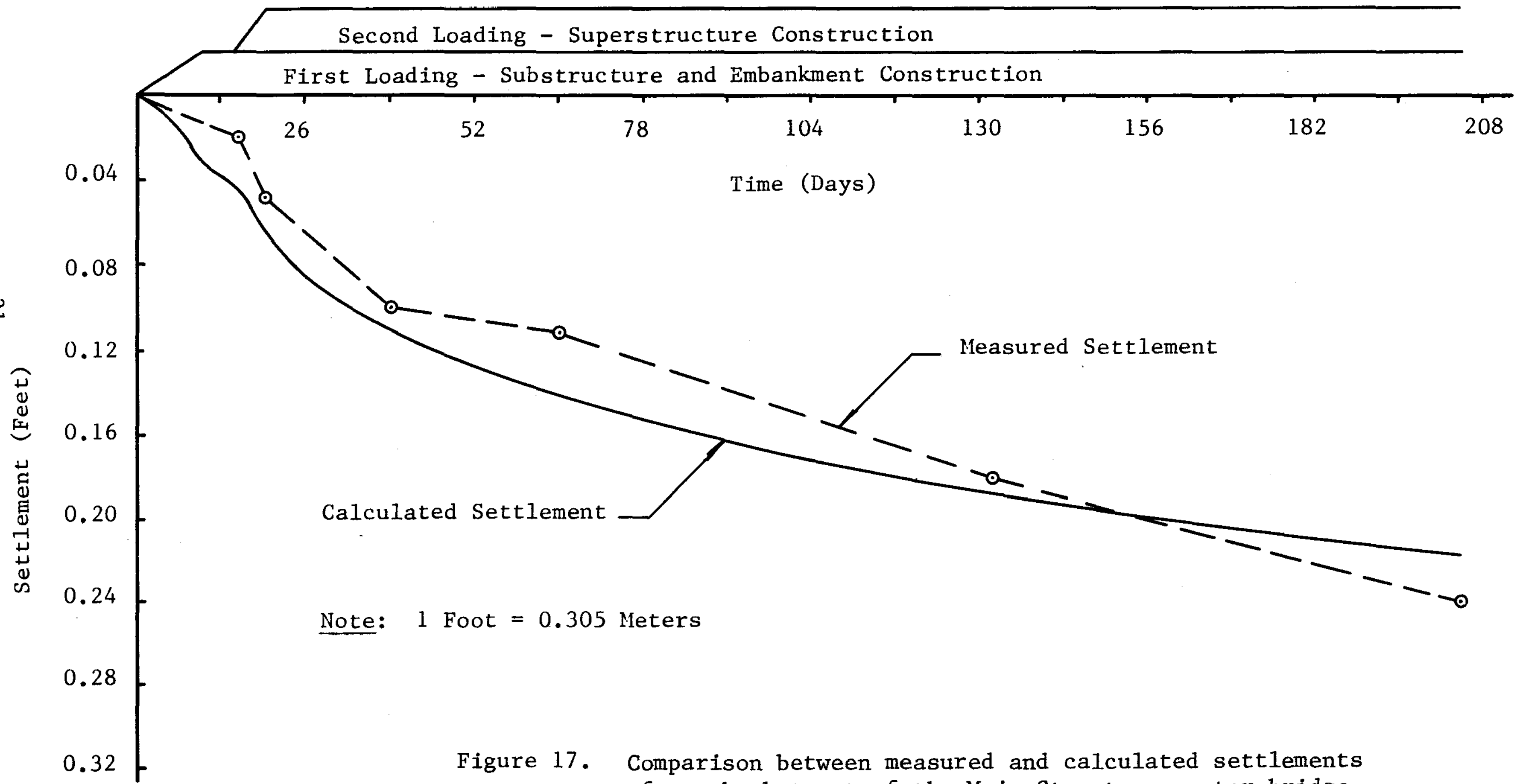


Figure 17. Comparison between measured and calculated settlements of north abutment of the Main Street connector bridge over Route 2 in East Hartford, Connecticut.

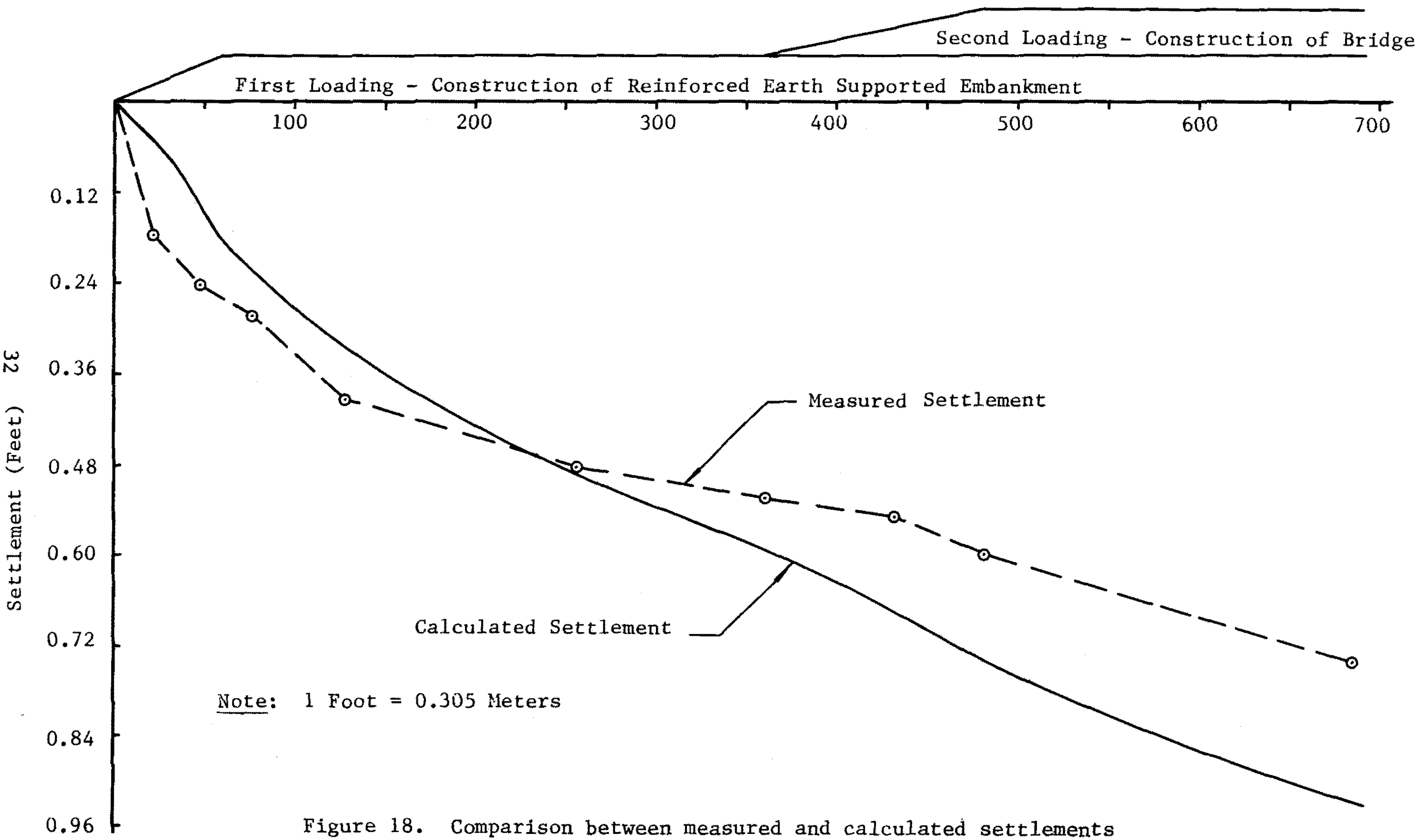


Figure 18. Comparison between measured and calculated settlements of north abutment of U.S. Route 1 Bridge over Boston and Maine Railroad at Wells, Maine.

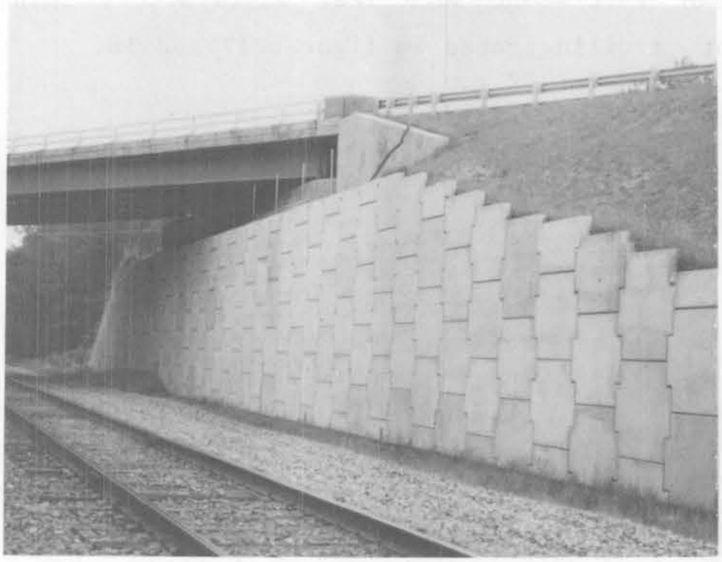


Figure 19. Reinforced earth supported embankment that serves as foundation for abutments of railroad bridge at Wells, Maine.

was constructed first as a preload and was allowed to settle for about a year, as shown in figure 18, before the bridge was constructed. The final calculated settlement at the north abutment is 31.0 inches (78.8 cm). However, a comparison with the final measured settlement is not possible at this time because the settlement is incomplete.

Overall, the results of the comparisons between predicted and measured settlements for cohesive soils showed that reasonably reliable predictions of the ultimate foundation settlement can be made, usually within 25 percent of the measured value, as long as good subsurface information and consolidation test data are available. However, in general, predictions of the time rate of settlement were less satisfactory than predictions of final settlement, as illustrated in figures 17 and 18.

ANALYTICAL STUDIES

The primary objective of the analytical studies was to evaluate the effects of differential vertical movements of various magnitudes on two-span and four-span continuous bridges of steel and concrete for a wide variety of span lengths. The tolerance of the bridge superstructures to the settlement of their foundations was investigated as a function of span length, stiffness and other problem parameters. For the most part, static loading conditions were used in the analysis, although for the steel bridges a limited investigation of the effect of dynamic loading was conducted. The results of the analyses were presented in graphical and/or tabular form showing the increases in stresses caused by differential settlements. In addition, a mathematical model for the behavior of multispan continuous steel slab/stringer systems was developed and used to prepare a series of design aids that could be used to estimate the stress increases resulting from the differential settlement of abutments or piers. Only a limited discussion of these analyses, their results and observations are presented here, and the reader is referred to the Interim and Final Reports for the details of the analyses and their results.

Steel Bridges

Continuous Slab/Stringer Systems

Static Loading. The analysis of the effect of support settlement for static loading was accomplished with the aid of the ICES-STRUDL-II computer package. The bridge superstructures were designed according to the "Standard Specifications for Highway Bridges" of the American Association of State Highway and Transportation Officials (AASHTO) for both dead and live loads. The live loading consisted of the AASHTO HS-20-44 wheel loading or its equivalent lane loading, depending on span length. Generally, three loading conditions were investigated: (a) dead load; (b) live load and dead load, with live load positioned to produce maximum negative moment; and (c) live load and dead load, with the live load positioned to produce maximum positive moment.

The settlements of the bridge supports were varied from zero up to three inches (76.2 mm) in increments of one-half inch (12.7 mm) or one inch (25.4 mm), depending on bridge type and span length. For the two-span bridges, two settlement cases were studied: (a) settlement of the exterior support (abutment) and (b) settlement of the center support (pier). For the four-span bridges, three settlement cases were studied: (a) settlement of the exterior support; (b) settlement of the interior support immediately adjacent to the exterior support; and (c) settlement of the center support.

The bridges investigated included continuous two-span and four-span slab/stringer systems consisting of rolled beam spans up to 60 feet (18.3 meters) in length, rolled beams with cover plates up to 150 feet (45.7 meters) in length, and plate girder spans up to 250 feet (76.2 meters) in length. A variety of stringer sizes and spacings were investigated. All slab/stringer systems utilized an 8 inch (203.2 mm) concrete deck, and composite action was assumed between the slab and the stringers. In each individual bridge, equal span lengths were used in order to reduce the number of variables considered.

The computer aided analyses resulted in graphical representations of the effects of support settlements on the moment and displacement diagrams for each structure, as illustrated for typical bridges in figures 20, 21 and 22. From moment diagrams, such as those shown in figures 20 and 22, the effect of differential settlement on the member stresses was determined. The results of these analyses showed that two settlement conditions were critical. For the two-span bridges, the maximum negative stress occurred at the center support, with settlement of the exterior support, under conditions of loading that would produce maximum negative stress. The maximum positive stress occurred near the mid-point of the first span of the structure, with settlement of the center support, under conditions of loading that produce maximum positive stress. For the four-span bridges, the maximum negative stress occurred at the center support, with settlement of the first interior support, under conditions of loading to produce maximum negative moment. The maximum positive stress occurred at approximately the mid-point of the second span, with settlement of the

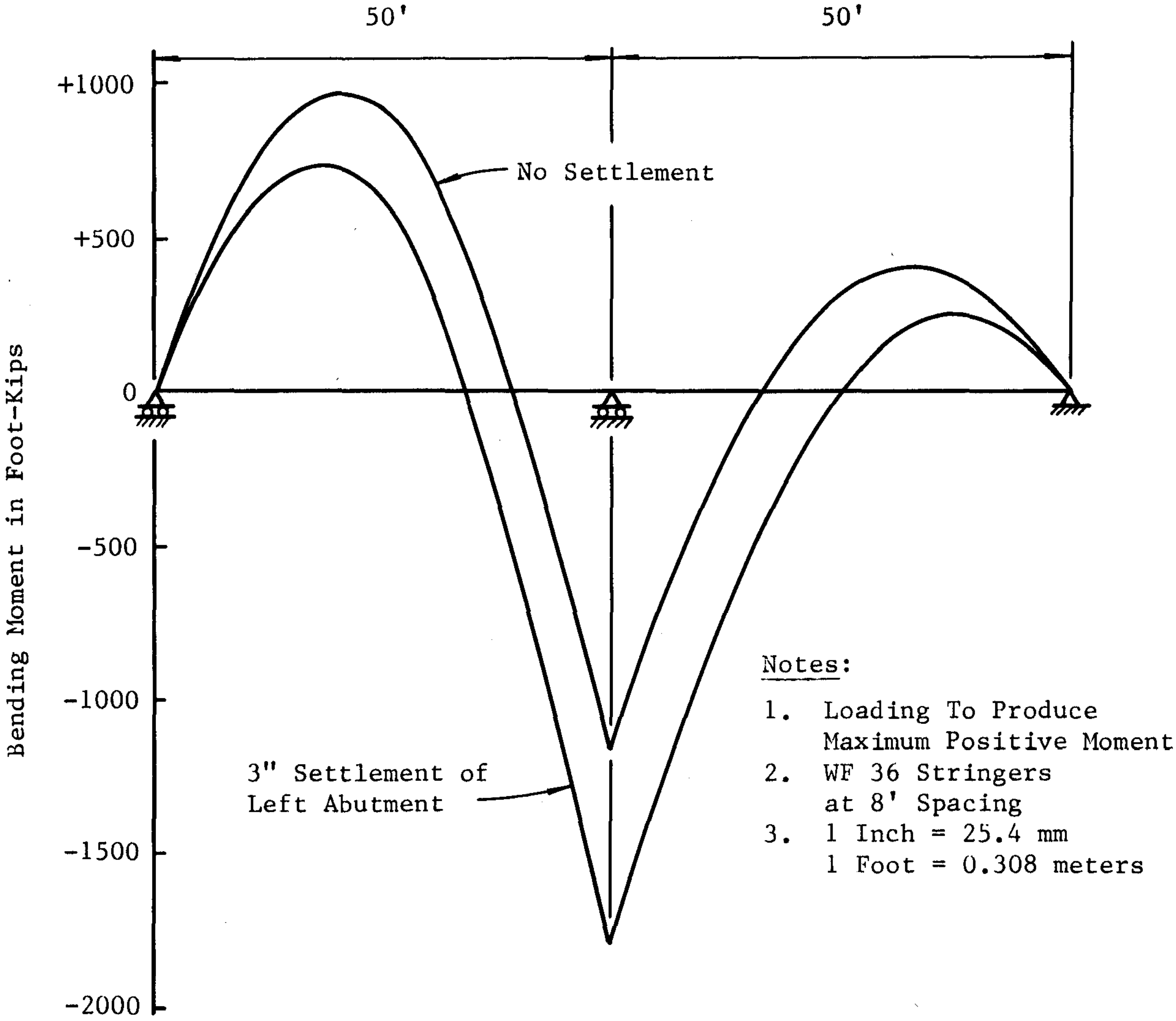


Figure 20. Typical moment diagram for two-span continuous bridge loaded with dead load, live load and settlement of left abutment.

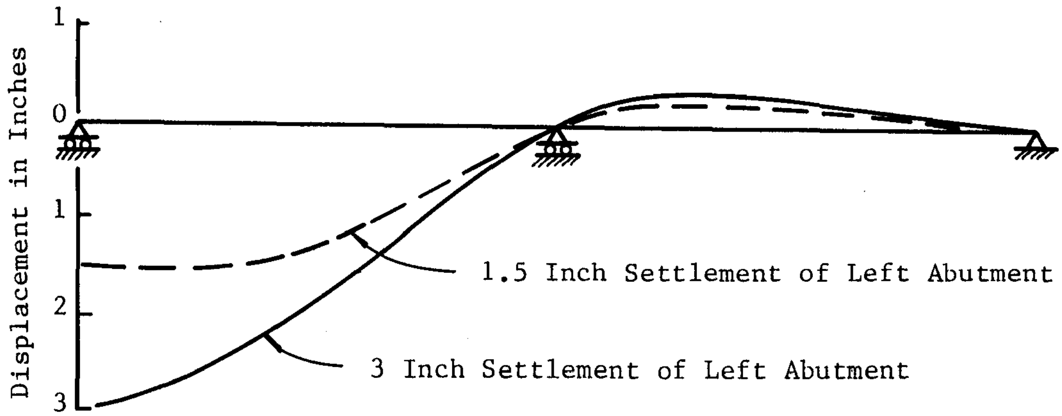


Figure 21. Typical displacement diagrams for two-span continuous bridge loaded with dead load, live load and settlement of left abutment.

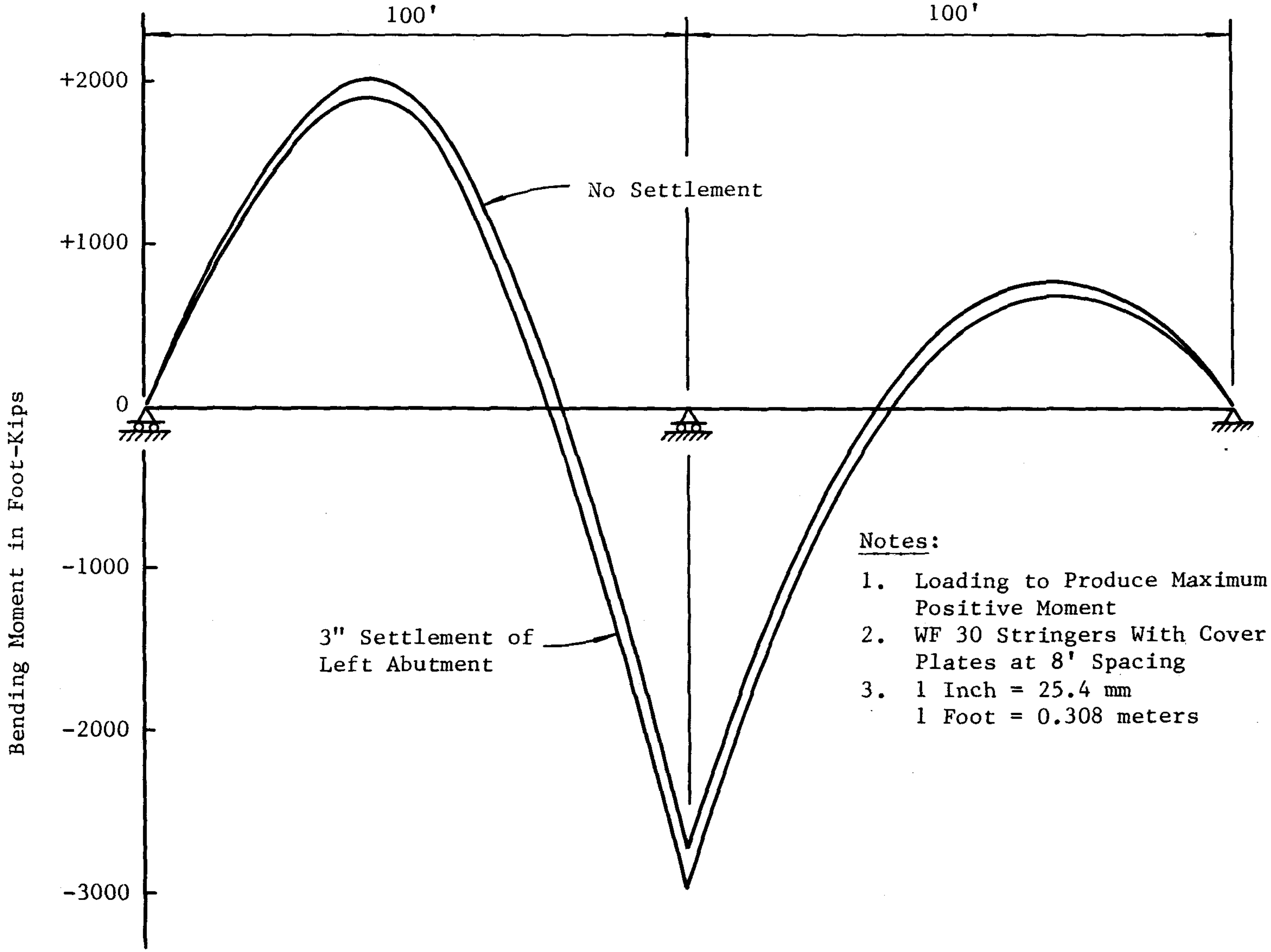


Figure 22. Typical moment diagram for two-span continuous bridge, with 100 foot spans, loaded with dead load, live load and settlement of left abutment.

center support, under conditions of loading to produce maximum positive moment in that span.

A study of the data resulting from the analyses of the two-span and four-span bridges showed that the effect of altering the stringer spacing was negligible. Although reducing the stringer spacing reduced the load on each stringer and thus reduced the moments, the effect of the differential settlement of the supports on the moments was very nearly the same for the stringer spacings investigated. However, the data show that support settlements of up to three inches (76.2 mm) can have a very important effect upon the stresses, depending upon the span length and rigidity (EI) of the slab/stringer system. This effect is particularly significant for short span bridges, up to 60 feet (18.3 meters) in length, as illustrated in figures 23 and 24, which show the effects of changing span length on the percentage increase in stresses in two-span continuous bridges for the two critical settlement conditions described above. It should be recognized that these are theoretical stress increases, calculated on the basis of assumed elastic behavior, and that yielding would occur before the higher theoretical stress levels (shown dashed in figures 23 and 24) are reached.

Similar data for four-span bridges showed that, for a given span length, the theoretical percentage increase in stress caused by differential settlement was substantially greater than for the two-span bridges. This is because the continuity of these structures increases their effective stiffness. However, as the span lengths increase, the stresses caused by differential settlements decrease substantially, as illustrated in figures 23 and 24 and by a comparison of the typical moment diagrams given in figures 20 and 22. This is further illustrated by the typical results of the analyses given in table 7, where the calculated maximum levels of the stresses produced by differential settlements up to three inches (76.2 mm) are compared to the design stresses for the zero settlement case. The low stresses for the zero settlement case for the 30 foot (9.1 meters) span are, in part, the result of the overdesign produced by using W36 stringers for this short span. The data in table 7 show that for longer spans, i.e. spans in excess of 100 feet, the calculated increases in stress caused by differential settlements up to three inches (76.2 mm) were virtually negligible.

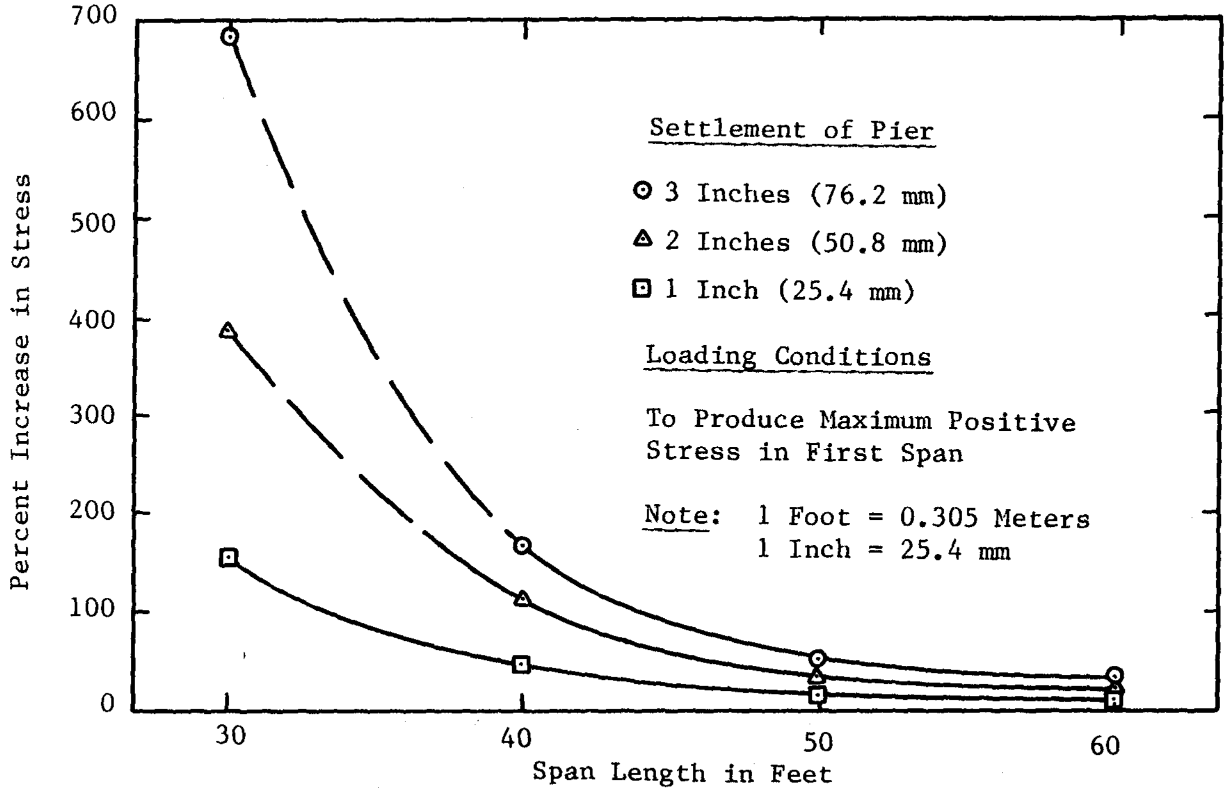


Figure 23. Theoretical percent increase in positive stress at mid-point of first span vs. span length for two-span continuous bridge (W36 composite).

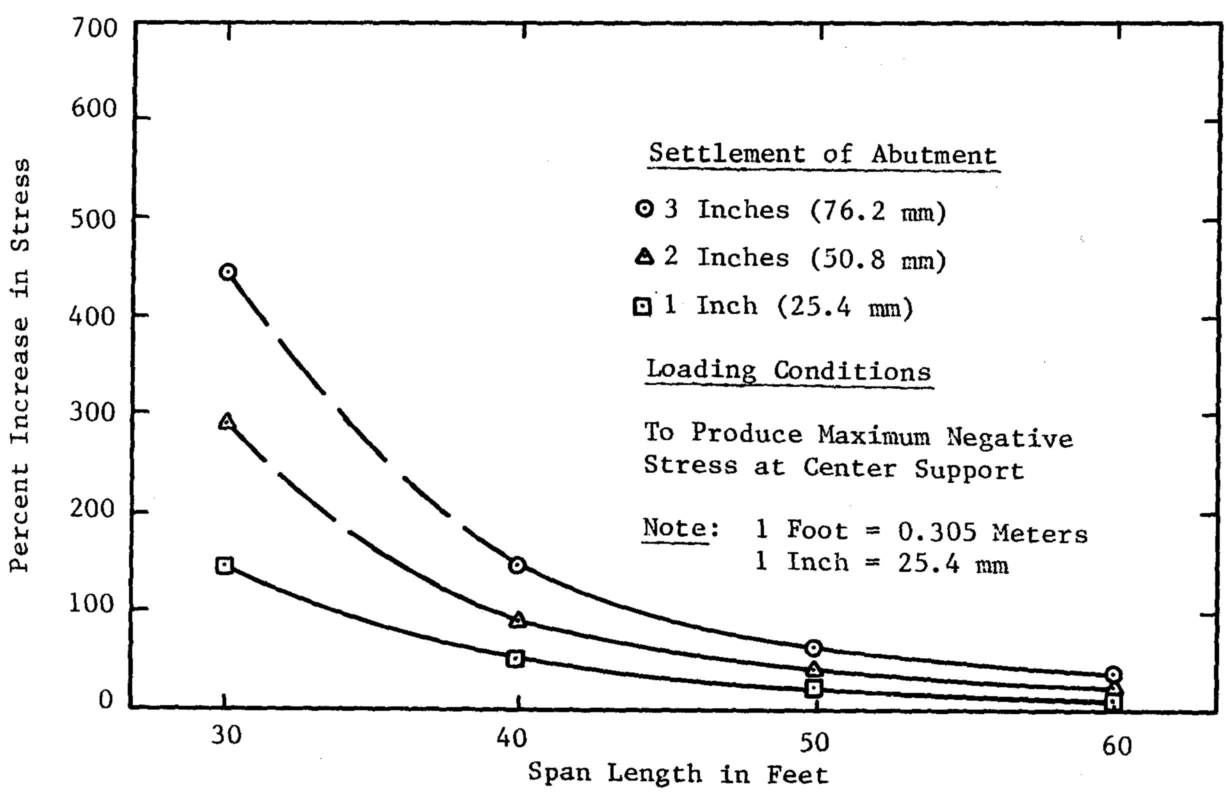


Figure 24. Theoretical percent increase in negative stress at center support vs. span length for two-span continuous bridge (W36 composite).

Table 7. Typical values of maximum negative stresses at the center support of two-span and four-span continuous steel bridges caused by differential settlements.

Span Length in Feet[a]	Settlement in Inches	Maximum Calculated Stresses(ksi): Two-Span Bridges With Settlement of Exterior Support	Maximum Calculated Stresses(ksi): Four-Span Bridges With Settlement of First Interior Support
30	0	14.6	11.0
	1	18.8	21.0
	2	28.2	36.5
	3	38.4	50.5
50	0	18.0	17.0
	1	22.5	23.2
	2	26.5	29.0
	3	30.0	35.0
100	0	18.8	18.4
	3	21.2	23.0
150	0	18.9	19.8
	3	21.8	21.5
200	0	20.0	19.0
	3	21.0	21.5
250	0	19.8	20.0
	3	21.2	21.3

[a]The 30 and 50 foot spans were designed with W36 stringers, the 100 foot span was designed with W36 sections and cover plates, and the 150 to 250 foot spans consist of plate girders.
Note: 1 inch = 25.4 mm, 1 foot = 0.305 meters and 1 ksi = 6.9 MPa.

The influence of the rigidity of the slab/stringer systems on their response to differential settlements was quite apparent when the data contained in figures 23 and 24 for the W36 - composite design were compared with similar data developed for designs using W33 and W30 stringers. These data showed that the lower rigidity of the W33 and W30 stringers led to a significantly lower level of stress increase as a result of differential settlement. However, the combined influence of span length and rigidity (stiffness) is best illustrated by comparing the theoretical stress increase, caused by differential settlement, with the ratio of the moment of inertia, I, to the span length, ℓ, as shown in figures 25 and 26 for the two-span bridges. These data show that, for stiff structures with short spans, the stress increase caused by differential settlement is much greater than for more flexible structures with long spans. Again, similar data for the four-span bridges showed greater percentage increases in stress levels than for the two-span structures. Overall, however, the results of the analysis showed that, for differential settlements up to three inches (76.2 mm), the stress increases would most likely be quite modest, as long as the ratio of moment of inertia to span length (I/ℓ) was 20 in^3. (327,741 mm^3) or less for both two-span and four-span bridges.

Dynamic Loading. The vibrations induced by traffic are generated by fluctuations of wheel contact loads as vehicles travel over bridge deck irregularities. These irregularities can be the result of (a) bridge deck deterioration and/or general roughness caused by poor construction control, or (b) a "bump" or "ramp" caused by the differential vertical movement of abutments or piers. The dynamic effects of both types of irregularities on two-span continuous steel bridges, with spans of from 30 to 250 feet (9.1 to 76.2 meters) were investigated in an effort to establish tolerable limits on frequencies, amplitudes, and human response levels. The analysis of each structure considered the effect of the weight of the load, the stiffness of the structures, the velocity of the moving load, and the truck axle spacing, as described in the Interim Report. Computer methods were utilized to perform these analyses.

The results of the analysis of slab/stringer systems under dynamic loading indicated that excessive dynamic deflection and frequency increases

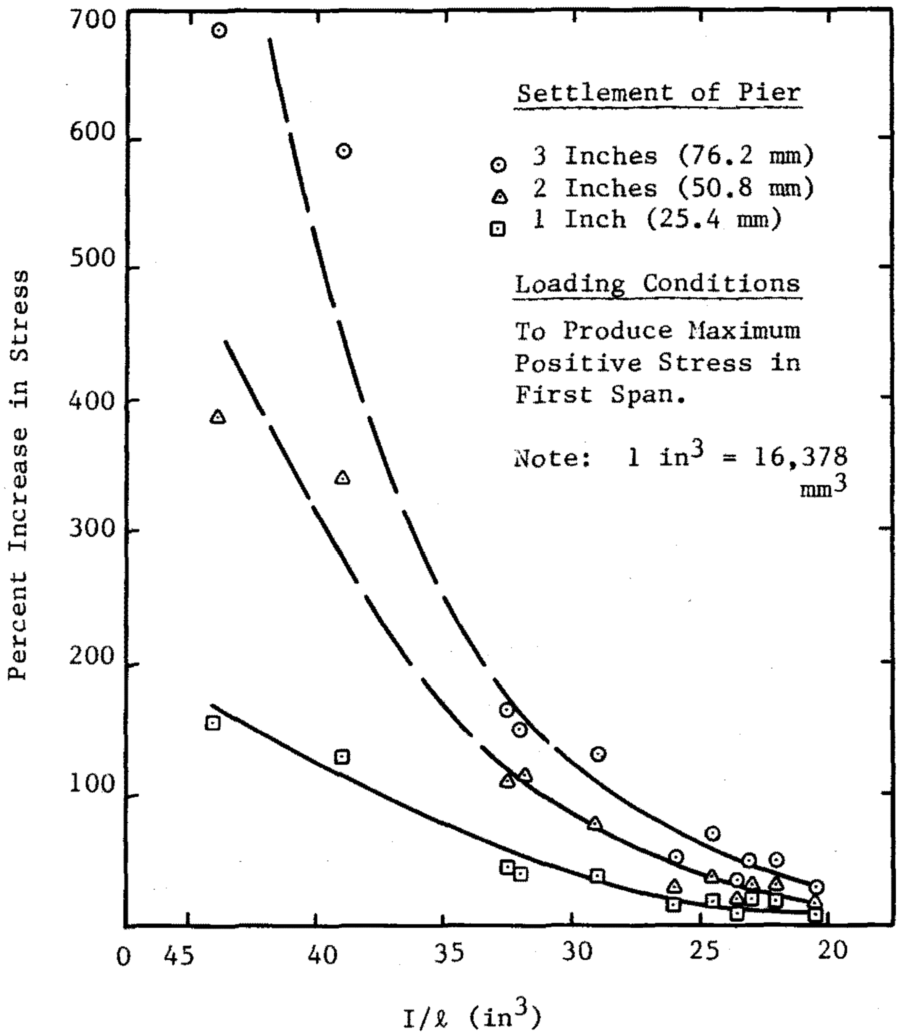


Figure 25. Theoretical percent increase in positive stress at mid-point of first span vs. I/ℓ for 2-span continuous bridges.

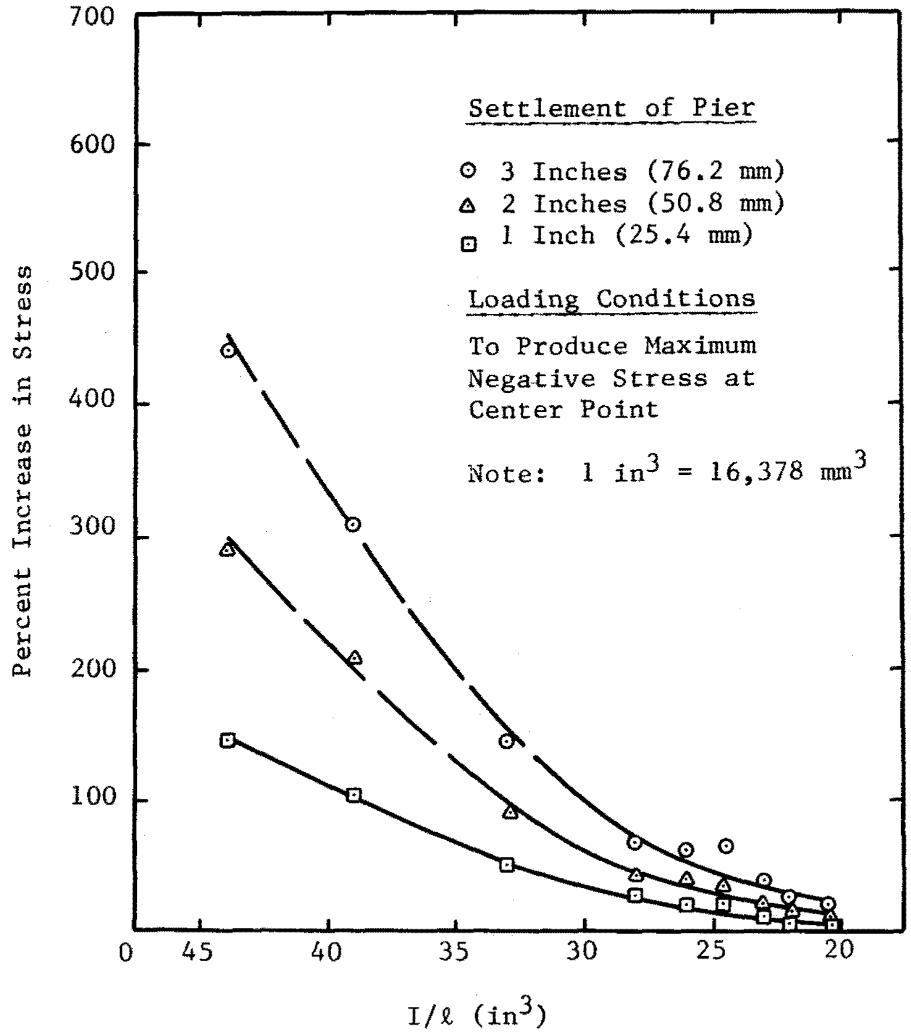


Figure 26. Theoretical percent increase in negative stress at center support vs. I/ℓ for 2-span continuous bridges.

might occur as the "resonance factor", i.e. the ratio of forced (ω_f) to the natural (ω_n) frequencies, approaches one. This information was used to establish a criterion that can be used by the designer to determine if a proposed bridge structure has sufficient mass and stiffness to prevent excessive dynamic deflection. The reader is referred to the Interim Report for further details.

Mathematical Model for the Behavior of Slab/Stringer Systems. Although the results produced by the analysis of the various steel bridge systems, as illustrated in figures 23 through 26, were very informative with respect to the influence of support settlements on stress increases, they are not particularly useful from a design standpoint. In an effort to remedy this situation, a mathematical model for the behavior of multispan continuous steel bridges was developed, using the macro flexibility approach, as described in the Interim Report. The expressions that were produced were simplified for computational ease and put in a form that would permit relatively simple checks to be made on the maximum stress increase produced by the settlement of any bridge support (either abutment or piers). The resulting equations were then used to develop a series of six design aids that would permit the estimation of the maximum positive and negative stresses in steel bridges resulting from differential settlement of abutments or piers. These design aids provide solutions for continuous steel bridges with up to five spans and with span lengths up to 250 feet (76.2 meters). Typical design aids for estimating the stresses produced by differential settlements of abutments and piers are presented in figures 27 and 28, respectively. The complete set of design aids has been published in the Interim Report.

In practice, the designer would enter the appropriate design aid with the span length, ℓ , and the number of spans, n, and pick off the values of $\Delta_o c/f_o(+)$ and $\Delta_o \bar{c}/f_o(-)$, for the case of abutment settlements, or values of $\Delta_\alpha c/f_\alpha(+)$ and $\Delta_\alpha \bar{c}/f_\alpha(-)$, for the case of pier settlement. These values could then be used with the anticipated abutment settlement, Δ_o, or pier settlement, Δ_α, and the estimated distances from the neutral axis to the outer fiber, c or $\bar{c}$, to calculate the maximum positive settlement stresses, $f_o(+)$ or $f_\alpha(+)$, or the maximum negative settlement stresses, $f_o(-)$ or $f_\alpha(-)$.

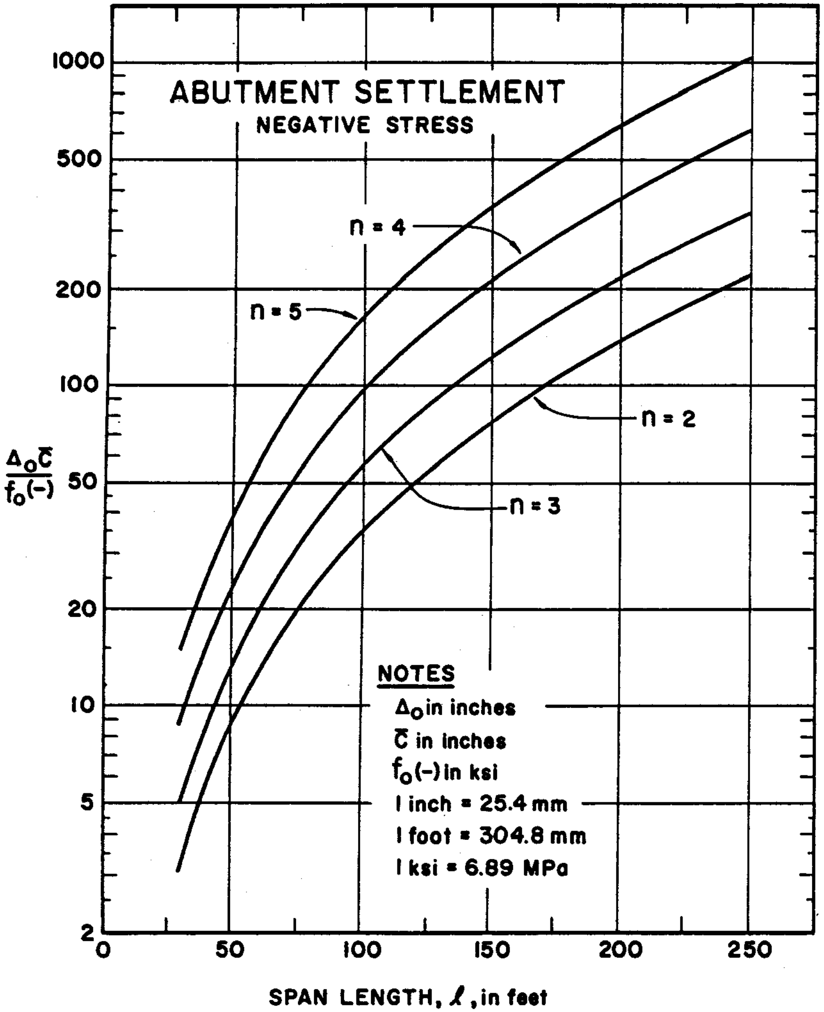


Figure 27. Design aid for determining the maximum negative stress increase caused by differential settlement of abutment.

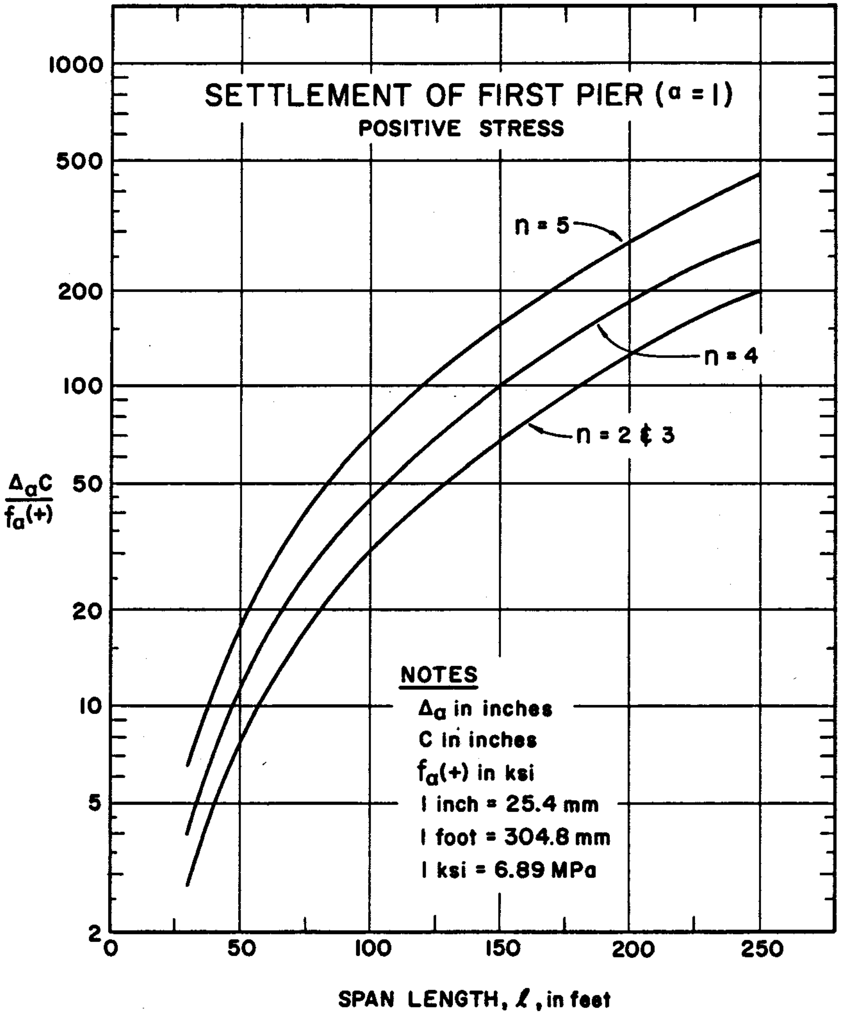


Figure 28. Design aid for determining the maximum positive stress increase caused by differential settlement of first interior support.

Continuous Truss Systems

In addition to the investigation of the effect of differential abutment and pier settlements on continuous two and four-span slab/stringer systems, two-span continuous parallel chord truss systems, with spans up to 680 feet (207.3 meters), and two-span continuous non-parallel chord truss systems, with spans up to 880 feet (268.2 inches), were also investigated.

For the two-span parallel chord trusses, span lengths of 480, 600 and 680 feet (146.3, 182.9 and 207.3 meters), with panel depths of 50, 60 and 70 feet (15.2, 18.3 and 21.3 meters), respectively, were investigated. A constant panel width of 40 feet (12.2 meters) was used in all cases, and the chord dimensions were kept constant for all spans in order to reduce the number of variables considered. For the nonparallel chord trusses, span lengths of 720, 800 and 880 feet (219.5, 243.8 and 268.2 meters) were analyzed. Again, the panel width was held constant at 40 feet (12.2 meters), but the depth of each truss varied from a maximum of 80 feet (24.4 meters) at the center support to a minimum of 40 feet (12.2 meters) at each quarter point. As the span length increased, the size of the chords was increased to increase the capacity of the structure. For both types of truss systems, the loads were applied at the panel points on the assumption that the floor beams would transfer the lane loadings to the trusses at these points. All trusses were analyzed as frames in order to account for any "secondary" stresses that might develop.

The results of the analysis of the two-span continuous truss systems showed that differential settlements up to three inches (76.2 mm) of either pier or abutment do not significantly affect the internal member stresses for long span trusses. For the parallel chord trusses, a maximum stress increase of about 9 percent was produced by a three inch (76.2 mm) settlement of the pier of the 70 foot (21.3 meter) deep truss with spans of 480 feet (146.3 meters), and the stress increases for the longer spans and smaller panel depths were substantially lower. The stress increases caused by a three inch (76.2 mm) differential settlement of the abutment were also very low. For the nonparallel chord trusses, a maximum stress increase of a little over three percent was produced by a three inch (76.2 mm) settlement of the abutment of the stiffest truss with spans of 720 feet, and, again, the stress increases for the longer spans and lower stiffnesses were substantially less. The stress increases caused by a three inch (76.2 mm) differential settlement of the pier were virtually negligible.

Concrete Bridges

The analysis of concrete highway bridges for the effects of support movement is an extremely complex problem. During the course of the investigation reported herein, the nature of these complexities was more fully appreciated, and, as the work progressed, it became apparent that the research originally proposed in this study could provide only a partial and fragmented answer to the question of what support movements may be tolerable for concrete highway bridges. The complexities of the problem lie in several primary areas: material properties, especially the creep behavior of concrete; structural configuration; sequence of construction; and analytical methods and simplifications. Each of these considerations leads to problems not encountered in the analysis of steel bridges.

The creep behavior of concrete materials is influenced by properties and proportions of the concrete mix constituents, as well as environmental factors associated with curing conditions.

Considerations of structural configuration are, in part, similar to those of steel bridges with comparable span lengths. However, some significant differences occur in the case of bridges constructed with precast, prestressed concrete I-type girders. For steel beams, the designer may make a refined choice of cross section by incrementing the overall height of the section and increasing the size of the flanges. In concrete, the choice may be reduced to selecting one of two standard sections, and providing an appropriate prestressing force. For example, in the case of a composite bridge with two equal spans of 100 feet (30.5 meters), made continuous for live loads, the designer might choose either an AASHTO-PCI standard Type IV or a Type V I-girder. The moment of inertia of the Type V section is about twice that of the Type IV, yet the section is only 17 percent deeper. Accordingly, the required prestressing force will be less for the Type V section, and the influence of creep due to a combination of dead load and prestressing force will be smaller. However, the settlement-induced stresses will be larger for the deeper Type V section. Thus, the overall comparison of the two sections shows that the

Type V section would be subjected to greater stresses due to settlement, but the effects of creep (and possibly creep relief of settlement-related stresses) will be less. This is but one example of the interactions of structural design parameters which complicate the analysis for conditions of support settlement. These parameters include number of spans, span length, girder type, prestress level, and profile of the prestressing strand.

The sequence of construction is particularly important in the analysis of bridges constructed of precast elements, made continuous to resist live loads, and acting composite with a cast-in-place deck. The creep behavior of precast elements, subsequently made continuous, is significantly different than that of a beam initially made continuous. Three events can be identified as significant with respect to the construction sequence: (a) the first loading of the concrete, (b) the time at which continuity is imposed, and (c) the time when settlement occurs. The order in which these last two events occur is also important, particularly where a gradual settlement is considered. Each of these aspects of construction is important in determining the significance of creep effects, and also the possibility of creep relief of settlement-induced stresses.

Each of these considerations, i.e. creep properties of the concrete, structural configuration and the sequence of construction, can be accounted for by using a sophisticated time-incremental solution employing computer methods. This procedure is very expensive to implement, because of the large amount of computer time required to analyze any particular case. It rapidly becomes infeasible when the number of cases for a meaningful parametric study is large. However, other, less sophisticated, methods are available for analysis either manually or on the computer, but they are, of course, more approximate in nature. Both types of solutions were employed for the studies reported herein, and a detailed description of these methods is included in the Interim Report.

The bridges investigated included composite and non-composite two-span continuous AASHTO-PCI standard I-girders, Types III, IV and VI, for spans of 75, 100 and 125 feet (22.9, 30.5 and 38.1 meters), respectively. These same girders and spans were also investigated for the non-composite case, where the beams were made continuous by means of a cast-in-place joint over the center support. In addition, two-span continuous cast-in-place box girder bridges with spans of 100 and 200 feet (30.5 and 61.0 meters), and a four-span continuous post tensioned box girder bridge with spans of 200 feet (61.0 meters) were also studied. For the two-span bridges, the effect of sudden and gradual settlements of the center support were considered, while for the four-span bridge, sudden and gradual settlements of the first interior support were considered. The differential settlements of the supports were varied between one inch (25.4 mm) and three inches (76.2 mm).

AASHTO-PCI Standard I-Girder Bridges

Continuous I-Girder Bridges. The analysis of a two-span continuous I-type girder provided a useful starting point for the discussion of bridges with spans of 75 to 125 feet (22.9 to 38.1 meters). Although this is not a practical type of construction, it is a convenient way to isolate effects of settlement. Using material properties corresponding to 5000 psi (34.5 MPa) concrete, the effect of a 3 inch (76.2 mm) settlement at the central support was considered. Girder types II, IV and VI were used for spans of 75, 100 and 125 feet (22.9, 30.5 and 38.1 meters), respectively. Comparing these I-sections, the approximate relative moments of inertia for the 75, 100 and 125 foot (22.9, 30.5 and 38.1 meter) spans increase as 1:2:6 and the relative section depths as 1:1.2:1.6.

Table 8 presents time-dependent moments and stresses in these continuous I-girder bridges for both sudden and gradual settlement. For the shortest span, a sudden 3 inch (76.2 mm) settlement produces bending moments significantly larger than dead load only. Even a settlement of only 1 inch (25.4 mm) would produce an effect on the order of 44 percent of the dead load moments.

In studying these results, it is important to remember that the cross section and span length are varying at the same time. An increase in span length, when other parameters are held constant, results in a more flexible structure and lower effects of settlement, since settlement moments are proportional to $3EI/\ell^2$, where E is the modulus of elasticity, I is the moment of inertia of the cross-section, and ℓ is the span length. However,

Table 8. Time-dependent moments and stresses in two-span I-Girder bridges caused by 3 inch settlement of center support.

Span Length in Feet (Girder Type)	Location of Moments and Stresses	Settlement Rate	Bending Moments in Foot-kips at Given Elapsed Time			Stresses in ksi at Given Elapsed Time at Given Location (Top or Bottom of Girder)					
						Zero Days		180 Days		1800 Days	
			Zero Days	180 Days	1800 Days	Top	Bottom	Top	Bottom	Top	Bottom
75 (III)	At Midspan	Sudden	+459	+281	+262	-1.00	+0.89	-0.66	+0.54	-0.62	+0.50
		Gradual	+198	+271	+282	-0.46	+0.38	-0.64	+0.52	-0.66	+0.54
	At Pier	Sudden	-125	-229	-268	-0.30	+0.24	+0.54	-0.44	+0.63	-0.52
		Gradual	-396	-249	-227	+0.93	-0.76	+0.59	-0.48	+0.53	-0.44
100 (IV)	At Midspan	Sudden	+805	+597	+574	-1.08	+0.90	-0.80	+0.68	-0.70	+0.60
		Gradual	+500	+585	+598	-0.67	+0.57	-0.78	+0.66	-0.80	+0.50
	At Pier	Sudden	-389	-806	-851	+0.52	-0.44	+1.08	-0.90	+1.10	-0.96
		Gradual	-1000	-839	-803	+1.30	-1.10	+1.10	-0.94	+1.08	-0.90
125 (VI)	At Midspan	Sudden	+1624	+1249	+1208	-0.94	+0.96	-0.72	+0.74	-0.70	+0.71
		Gradual	+1074	+1228	+1251	-0.62	+0.64	-0.71	+0.73	-0.72	+0.74
	At Pier	Sudden	-1048	-1798	-1879	+0.61	-0.62	+1.04	-1.07	+1.09	-1.10
		Gradual	-2148	-1840	-1794	+1.20	-1.27	+1.07	-1.09	+1.04	-1.06

Positive moment causes positive stress (tension) in bottom fibers.
Note: 1 ksi = 6.9 MPa, 1 kip-foot = 1.37 kN - m, 1 inch = 25.4 mm, 1 foot = 0.305 meters

longer spans also have greater effects of dead and live load, so a larger cross section is required.

For the 75, 100 and 125 foot (22.9, 30.5 and 38.1 meter) I-girders considered, the factor I/ℓ^2 and, hence, the settlement moments, increase with increasing span, as 1:1.2:2.1. However, the ratio of settlement stresses to dead load stresses varies as I/ℓ^4, since dead load moments increase as the square of the span length. For these I-girders and spans, the term I/ℓ^4 varies as 1:0.66:0.75. Thus, the relative effect of settlement drops off and then increases again as span lengths increase, a result of the particular choice of girder section.

Precast Girders Made Continuous With a Field Joint. A similar analysis to that of the previous section was performed for two-span continuous structures made from two precast beams with a cast-in-place field joint. Spans and girder sizes are the same as before, and the results are shown in table 9. For this type of structure, stresses follow the I/ℓ^2 relationship described previously. In all cases, cracking may result at the central support due to the effects of sudden settlement. The effects of sudden settlement are reduced with time due to creep relief of the settlement moment in conjunction with the creep redistribution of dead load moments. In the case of gradual settlement, moments induced by settlement, and those resulting from moment redistribution, offset one another.

Because of redistribution of dead load movements due to creep, the stresses resulting from settlement in a continuous structure made continuous by a cast-in-place joint are considerably lower than for a cast-in-place continuous bridge.

Girder Composite With Cast-in-Place Deck. In the analyses reported in this section, composite action was introduced by casting a concrete deck over cast-in-place I-type girders. The material properties assumed in analysis are typical of 5000 psi (34.5 MPa) concrete in the girder, and 4000 psi (27.6 MPa) concrete in the deck. A maximum sudden settlement of 3 inches (76.2 mm) at the central support of the resulting two-span continuous composite beam was assumed. Girder sections and spans were the

Table 9. Time-dependent moments and stresses in two-span bridges made continuous with a field joint, caused by 3 inch settlement of center support.

Span Length in Feet (Girder Type)	Location of Moments and Stresses	Settlement Rate	Bending Moments in Foot-kips at Given Elapsed Time			Stresses in ksi at Given Elapsed Time at Given Location (Top or Bottom of Girder)					
						Zero Days		180 Days		1800 Days	
			Zero Days	180 Days	1800 Days	Top	Bottom	Top	Bottom	Top	Bottom
75 (III)	At Midspan	Sudden	+657	+344	+310	-1.55	+1.27	-0.81	+0.66	-0.73	+0.60
		Gradual	+396	+334	+330	-0.93	+0.76	-0.79	+0.64	-0.78	+0.64
	At Pier	Sudden	+522	-103	-171	-1.23	+1.01	+0.24	-0.20	+0.40	-0.33
		Gradual	0	-124	-131	0	0	+0.29	-0.24	+0.31	-0.25
100 (IV)	At Midspan	Sudden	+1305	+756	+696	-1.75	+1.48	-1.01	+0.86	-0.93	+0.79
		Gradual	+1000	+744	+720	-1.34	+1.13	-1.00	+0.84	-0.87	+0.82
	At Pier	Sudden	+611	-488	-607	-0.82	+0.69	+0.65	+0.68	+0.81	-0.69
		Gradual	0	-511	-599	0	0	-0.55	-0.58	+0.80	-0.68
125 (VI)	At Midspan	Sudden	+2684	+1760	+1710	-1.56	+1.59	-1.02	+1.04	-0.99	+1.01
		Gradual	+2134	+1637	+1524	-1.24	+1.27	-0.95	+0.97	-0.88	+0.90
	At Pier	Sudden	+1100	-748	-847	-0.64	+0.65	+0.43	-0.44	+0.49	-0.50
		Gradual	0	-992	-1218	0	0	+0.57	-0.59	+0.70	-0.72

Positive moment causes positive stress (tension) in bottom fibers.
Note: 1 ksi = 6.9 MPa, 1 kip-foot = 1.37 kN - m, 1 inch = 25.4 mm, 1 foot = 0.305 meters

same as in previous examples. Settlement was assumed to occur when the girder age was 28 days and the slab was one day old.

Results for the three span lengths are shown in table 10. A comparison is provided for composite action, both accounting for and ignoring the effects of shrinkage and creep. Deck stresses change only slightly due to settlement, since the settlement occurs when the deck concrete is very weak and has low stiffness. Consequently, girder stresses are comparable to those of cast-in-place bridges. Creep and shrinkage tend to reduce the effects of settlement, as illustrated in figure 29, which shows the time-dependent variation of stresses at midspan of the 100 foot (30.5 meter) span bridge resulting from a 3 inch (76.2 mm) sudden settlement of the center support.

To contrast the effects of sudden and gradual settlements, the same 100 foot (30.5 meter) span bridge was analyzed for a total settlement of 3 inches (76.2 mm), assuming a time-dependent variation of the settlement. Equal increments of 1 inch (25.4 mm) settlement were applied at 93 days, 453 days and 1553 days. Time-dependent stresses for the gradual settlement are shown in figures 30 and 31 for midspan and the central support, respectively. In this case, a gradual settlement results in eventual higher stresses at the central support than does sudden settlement. Maximum stresses occur during the application of the second increment of deflection at 453 days. Thus, a slow gradual application of settlement does not create high initial stresses, but the lack of creep relief causes the stresses to eventually become higher than those caused by sudden settlement.

<u>Composite Section with Prestressing</u>. To supplement the studies described above, a series of analyses were conducted for two-span precast prestressed I-girders, made continuous for live loads by a cast-in-place joint, acting composite with a cast-in-place deck. The prestressing force was chosen to exactly balance the tensile stress at midspan for the loading condition which produces maximum positive moments. A parabolic strand profile was assumed, so the effects of prestressing can be accounted for by means of an equivalent distributed load. In the analysis, it was assumed that girder and deck had identical properties and that the settlement occurred just after continuity was imposed.

Table 10. Long-term stresses in two-span continuous cast-in-place composite bridges caused by dead load and settlement.

Span Length in Feet (Girder Type)	Assumed Settlement of Central Support	Assumed Behavior with Respect to Creep and Shrinkage	Stresses in ksi in Given Member at Given Location							
			At Central Support				At Mid Span			
			Slab		Girder		Slab		Girder	
			Top	Bottom	Top	Bottom	Top	Bottom	Top	Bottom
75 (III)	3 Inch Sudden	Included	+0.43	+0.27	-0.01	-0.84	-0.12	-0.05	-0.79	+0.83
		None	-0.38	-0.25	-0.65	+1.30	-0.39	-0.26	-1.60	+2.10
	None	Included	+0.43	+0.28	+0.22	-1.00	-0.11	-0.04	-0.67	+0.75
		None	0.00	0.00	0.00	0.00	-0.20	-0.13	-1.30	+1.50
100 (IV)	3 Inch Sudden	Included	+0.60	+0.43	+0.24	-1.20	-0.16	-0.09	-1.00	+1.10
		None	-0.15	-0.11	-0.29	+0.48	-0.41	-0.29	-2.10	+2.40
	None	Included	+0.60	+0.44	+0.45	-1.30	-0.16	-0.08	-1.00	+1.10
		None	0.00	0.00	0.00	0.00	-0.26	-0.18	-1.80	+1.90
125 (VI)	3 Inch Sudden	Included	+0.64	+0.50	+0.32	-1.20	-0.16	-0.09	-1.00	+1.20
		None	-0.16	-0.12	-0.32	+0.54	-0.39	-0.30	-2.00	+2.50
	None	Included	+0.64	+0.50	+0.55	-1.30	-0.16	-0.09	-0.96	+1.10
		None	0.00	0.00	0.00	0.00	-0.23	-0.17	-1.70	+2.00

Note: 1 ksi = 6.9 MPa, 1 inch = 25.4 mm, 1 foot = 0.305 meters.

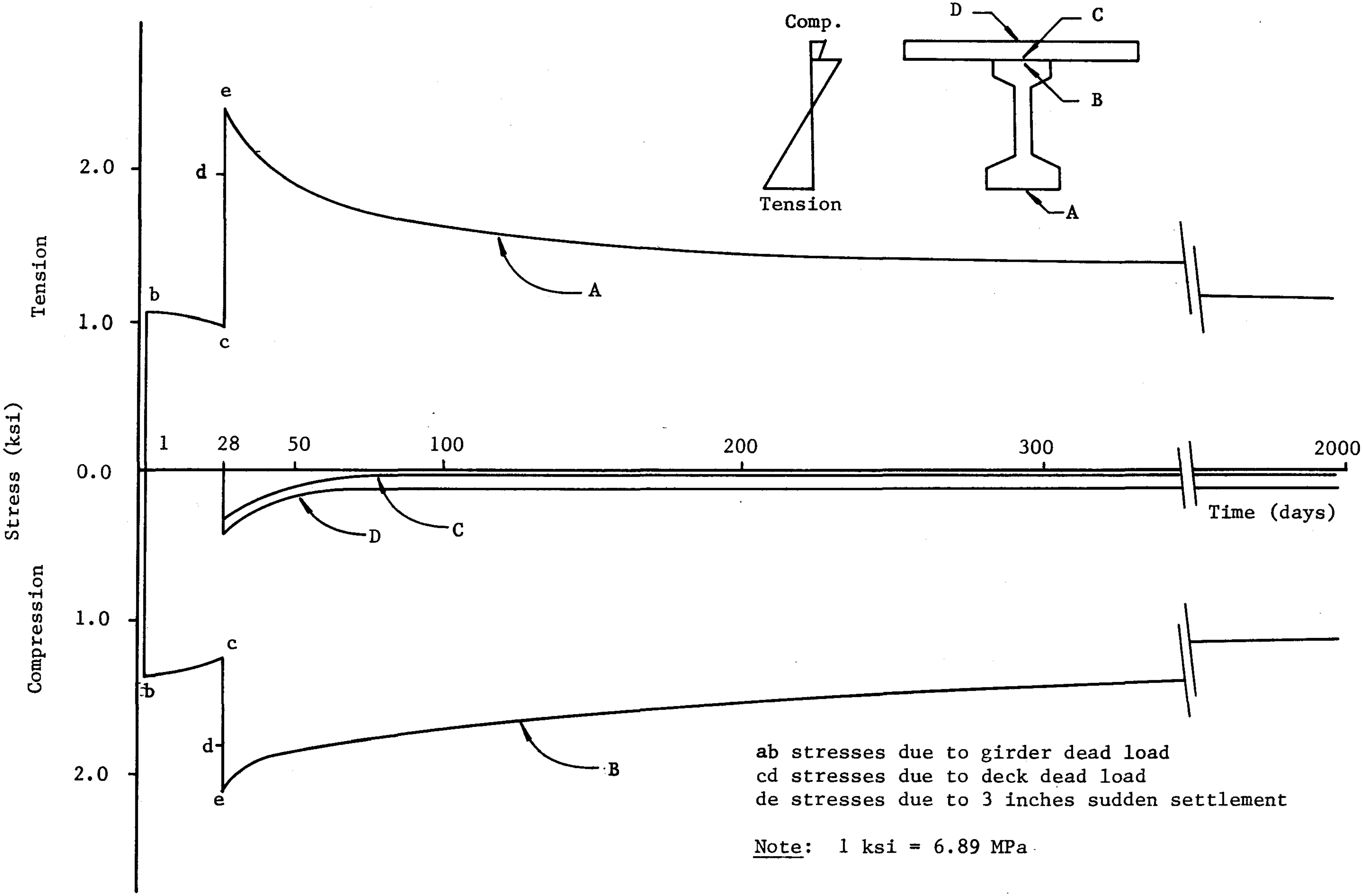


Figure 29. Time-dependent variation of stresses at midspan for two-span continuous concrete bridge with 100 foot spans - sudden settlement of center support.

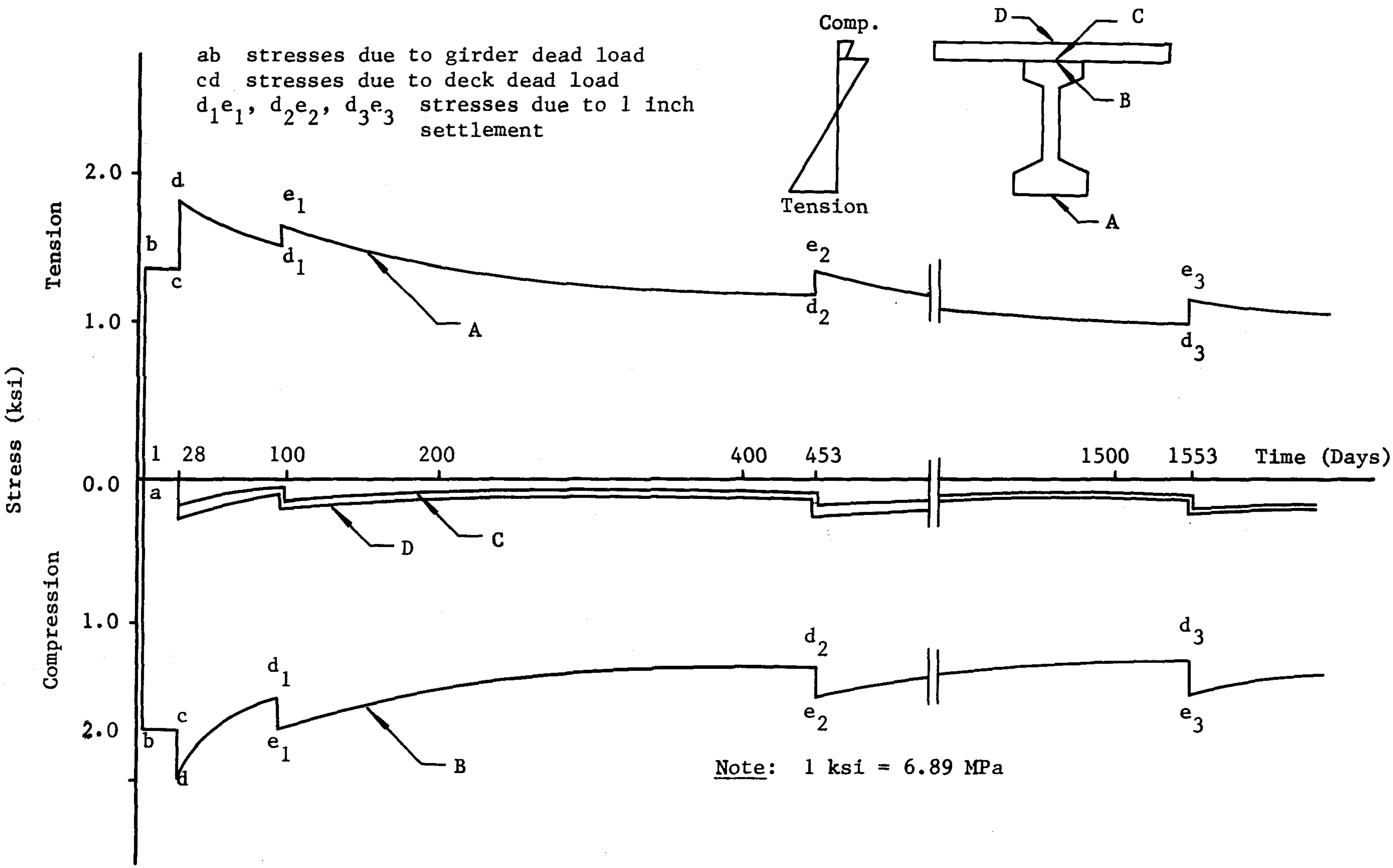
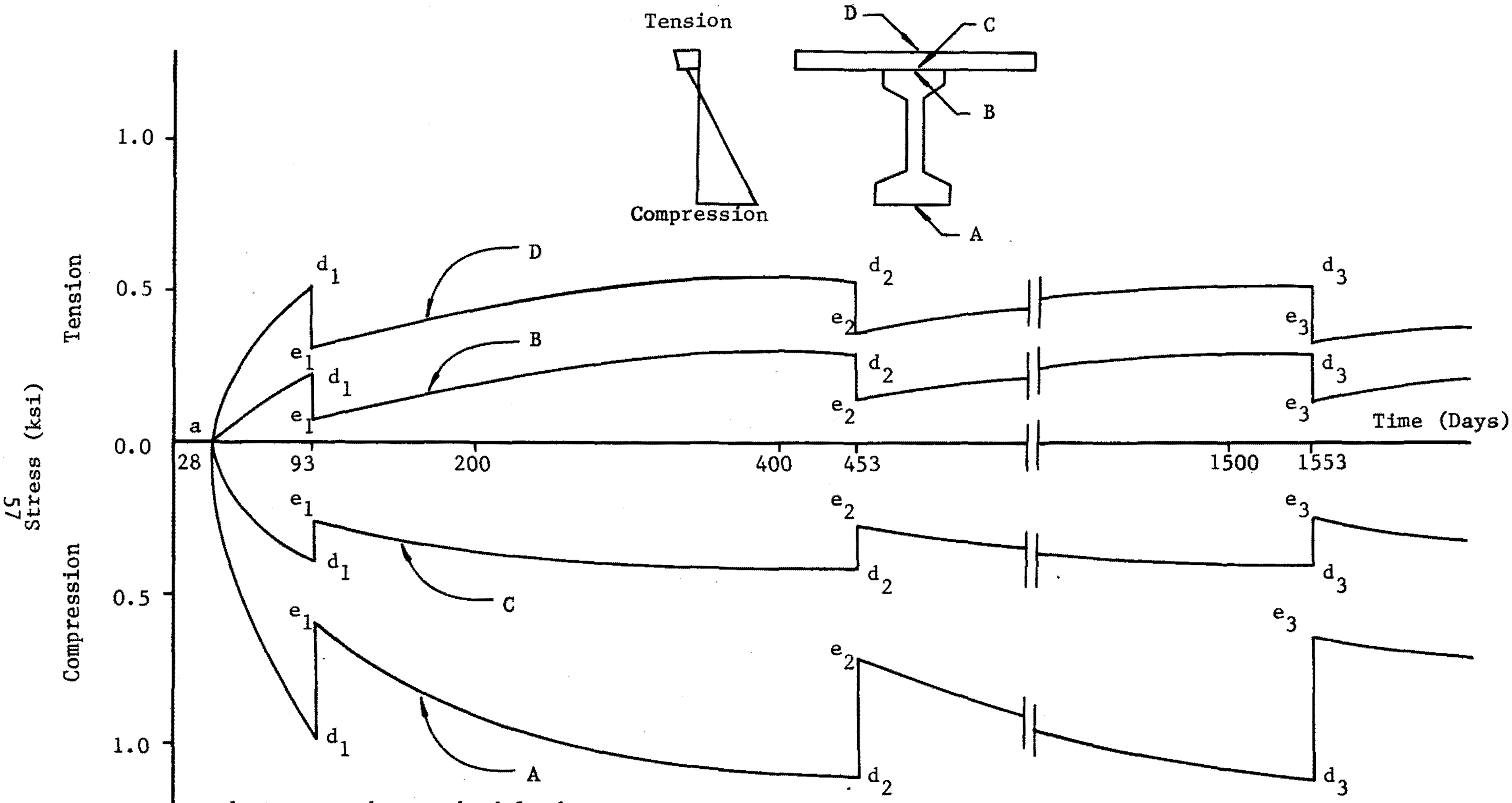


Figure 30. **Time-dependent variation** of stresses at midspan for two-span continuous concrete bridge with 100 foot spans - gradual settlement of center support.



ad stresses due to dead loads
d_1e_1, d_2e_2, d_3e_3 stresses due to 1 inch settlement

Note: 1 ksi = 6.89 MPa

Figure 31. **Time-dependent variation of stresses at center support for continuous concrete bridge with 100-foot spans - gradual** settlement of center support.

The results of these analyses for spans of 75 and 125 feet (22.9 and 38.1 meters), with Type III and Type VI girders, respectively, are shown in table 11. These results show the same general trends as for composite sections where prestressing was neglected, with the stresses merely shifted by the effect of prestress. As before, the total effects of settlement are reduced to about one-third of the instantaneous value due to the effects of creep. Analysis shows the stresses to remain within the allowable range for dead load settlement and prestresses, but live load will cause the allowable compressive stress to be exceeded.

Summary. The analyses described above have considered the combined effects of settlement and creep for various structural configurations with AASHTO-PCI standard I-girders. It was found that stresses resulting from sudden settlement are proportional to the settlement itself, the modulus of elasticity of the concrete when loaded, and the depth of the cross section, and inversely proportional to the span length. The overall ratio of settlement stresses to those caused by dead loads varies as the term I/ℓ^4. Therefore, a designer faced with a choice of possible cross sections should choose the section with a lower ratio of I/ℓ^4 to minimize the relative effects of settlement.

The effects of settlement and creep are in opposing senses in the case of precast elements made continuous for live loads. This does not, however, eliminate the need to investigate settlement-related stresses in these structures. Generally, for these structures, the effects of a 3 inch (76.2 mm) sudden settlement are unacceptably high when span lengths are on the order of 100 feet (30.5 meters) or less. The effects do drop off with increasing span length, and with 125 feet (38.1 meters) spans, stresses may be controlled by additional reinforcement.

Limited investigation of the effects of prestressing shows a need to study additional effects of span profile, age at loading, and gradual loading.

Box Girder Bridges

The research originally planned involved the study of the effects of sudden and gradual settlements of up to 3 inches (76.2 mm) for bridges

Table 11. Time-dependent stresses for two-span precast prestressed I-Girders made continuous for live loads by cast-in-place joint, acting composite with cast-in-place deck.

Span Length in Feet (Girder Type)	Location of Stresses	Settlement of Central Support in Inches	Stresses[a] in ksi at the Given Location for the Given Loading Condition and Elapsed Time					
			Dead Load+Prestress Zero Days		Dead Load + Prestress +Settlement, Zero Days		Dead Load + Prestress +Settlement, 10,000 Days	
			Top	Bottom	Top	Bottom	Top	Bottom
75 (III)	At Midspan	0	-1.53	-1.76	-1.53	-1.76	-1.61	-1.45
		3	-1.53	-1.76	-1.74	-0.96	-1.61	-1.48
	At Pier	0	-1.58	-1.58	-1.58	-1.58	-1.55	-1.69
		3	-1.58	-1.58	-2.00	0.00	-1.73	-1.02
100 (IV)	At Midspan	0	-1.40	-1.37	-1.40	-1.37	-1.39	-1.38
		3	-1.40	-1.37	-1.57	-0.97	-1.44	-1.27
	At Pier	0	-1.39	-1.39	-1.39	-1.39	-1.39	-1.40
		3	-1.39	-1.39	-1.73	-0.58	-1.47	-1.18

[a]Negative stresses are compression.
Note: 1 ksi = 6.9 MPa, 1 foot = 0.305 meters, 1 inch = 25.4 mm.

constructed of precast box sections for spans of 100, 125 and 150 feet (30.5, 38.1 and 45.8 meters), and cast-in-place box girders for span lengths from 100 to 300 feet (30.5 to 91.5 meters) in increments of 25 feet (7.6 meters). However, upon evaluating the pilot study accomplished as a part of this investigation, it was felt that the additional studies of precast box sections in the span range of 100 to 150 feet (30.5 to 45.8 meters) would be redundant in the light of the results of the analysis of the AASHTO-PCI standard I-girders, so additional analyses were not conducted.

The original intent for the many span length combinations to be analyzed for the cast-in-place box girders was to consider the possibility of tuning the superstructure; that is, adjusting the post-tensioning force over a period of time to keep total stresses within some acceptable range. After some preliminary analysis of two- and four-span continuous box girders, addtional efforts did not seem prudent. The analyses were quite expensive, and additional parameters other than span length should have been considered for completeness. The balance of this section will report the preliminary analysis made for two- and four-span box girders with span lengths of 100 and 200 feet (30.5 and 61.0 meters).

Two-Span Continuous Box Girders. The effects of sudden settlement were investigated for symmetrical two-span, continuous, cast-in-place box girder bridges with span lengths of 100 and 200 feet (30.5 and 61.0 meters). These structures were analyzed, as described in the Interim Report, using an in-house computer program. The box girders had an overall deck width of 27 feet 4 inches (8.3 meters), and a cell width of 13 feet (4.0 meters) at the bottom. Deck thickness was 7 inches (177.8 mm), the webs were 12 inches (305.2 mm) thick, and the bottom of the cell was 8 inches (203.2 mm) thick. Overall depth of the box section was 90 inches (2.4 meters). Concrete material properties assumed for purposes of analysis included a compressive strength of 5000 psi (35 MPa), a modulus of elasticity of 4500 ksi (31.5 GPa), a normal creep coefficient, ν, of 1.9 and an ultimate shrinkage of 210 micro strains.

For simplicity, several assumptions are necessary regarding the sequences of construction and loading. First, all concrete in the box girder was assumed to be placed at the same time, so elastic and time-dependent material properties would be the same throughout. Second, the girder was assumed to be shored until the concrete had reached an age of 28 days, when shoring was removed. At that time, the girder must support its own weight, and the concrete begins to creep. Finally, a sudden settlement of 3 inches (76.2 mm) at the central support was assumed to occur just after the shoring was removed.

Results of the analyses are shown in figures 32 and 33 for the bridge with 100 foot (30.5 meter) spans, and figures 34 and 35 for the bridge with the 200 foot (61 meters) spans. In each of these figures, the combined effects of dead load, settlement, shrinkage and creep are shown by a solid line, while the combined effects of dead load and settlement acting without creep relief are shown by a dashed line.

At the mid-span section, stresses due to settlement have the same sense as stresses due to dead loads. In doubling the span length it can be seen that dead load stresses increase by a factor of four, while the settlement stresses are decreased by a factor of four. Thus, the ratio of settlement to dead load stresses is inversely proportional to the fourth power of span length. For both span lengths, the effect of creep is to reduce the settlement-related stresses to about one-third of the instantaneous value.

For stresses at the center support, the conclusions are similar, with one important difference. At this section, the sense of stresses induced by the effects of dead load and settlement are opposite. For example, at the bottom flange, compressive stresses result from the effects of dead load, while tension effects are induced by settlement. This is shown to be quite significant for the shorter span, as shown in figure 33. In this case, a stress reversal occurs at the central support, leaving a significant net tension in the bottom flange. Since all of the analysis has assumed an uncracked elastic section, this figure likely overestimates the actual value of the tensile stress. However, a significant amount of cracking is certain to occur in the vicinity of the support. This stress is mitigated by the effects of creep and shrinkage, and a compressive stress is eventually restored.

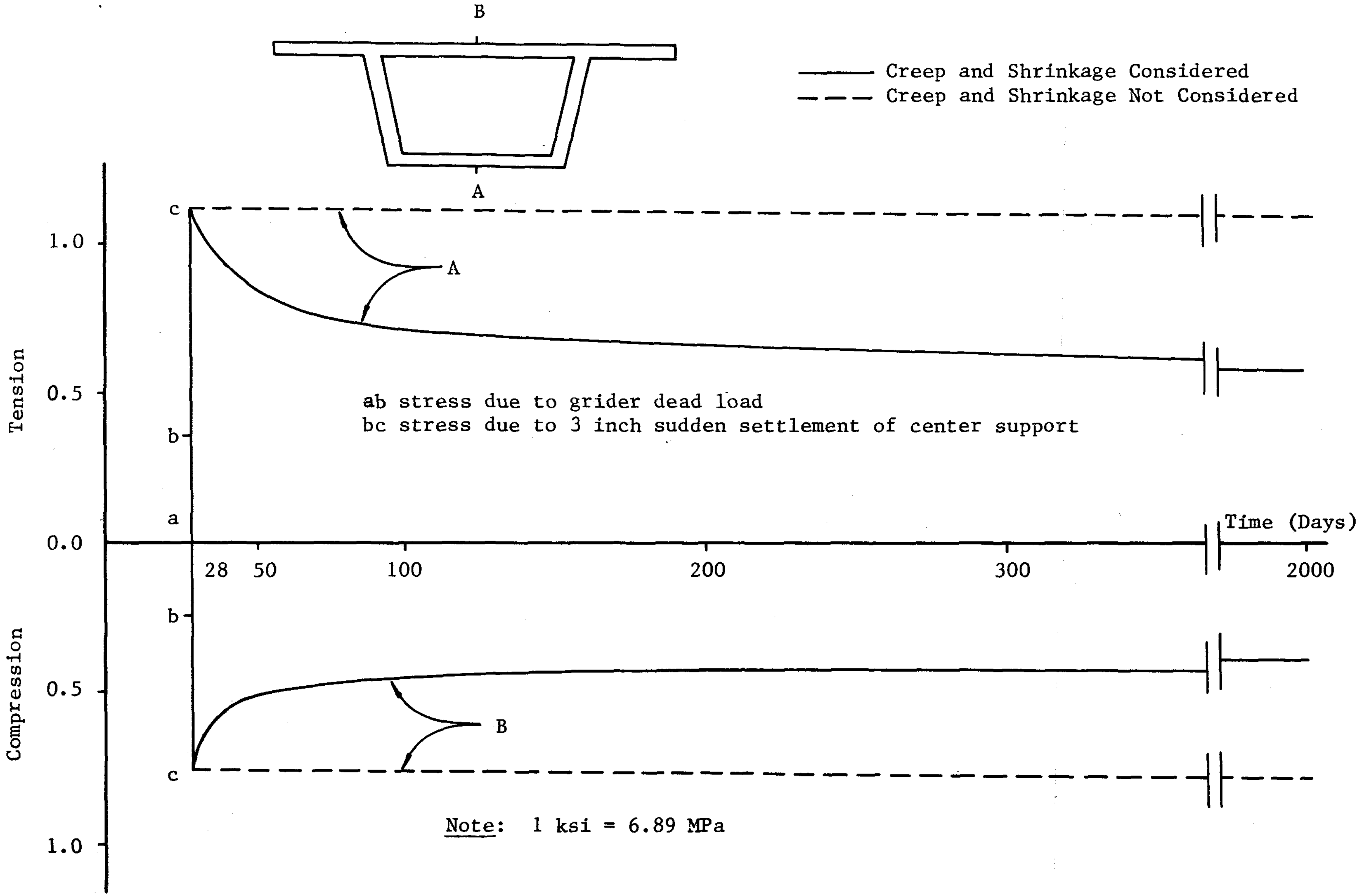


Figure 32. Time-dependent stresses at midspan for two-span continuous concrete box girder with 100-foot spans - sudden settlement of center support.

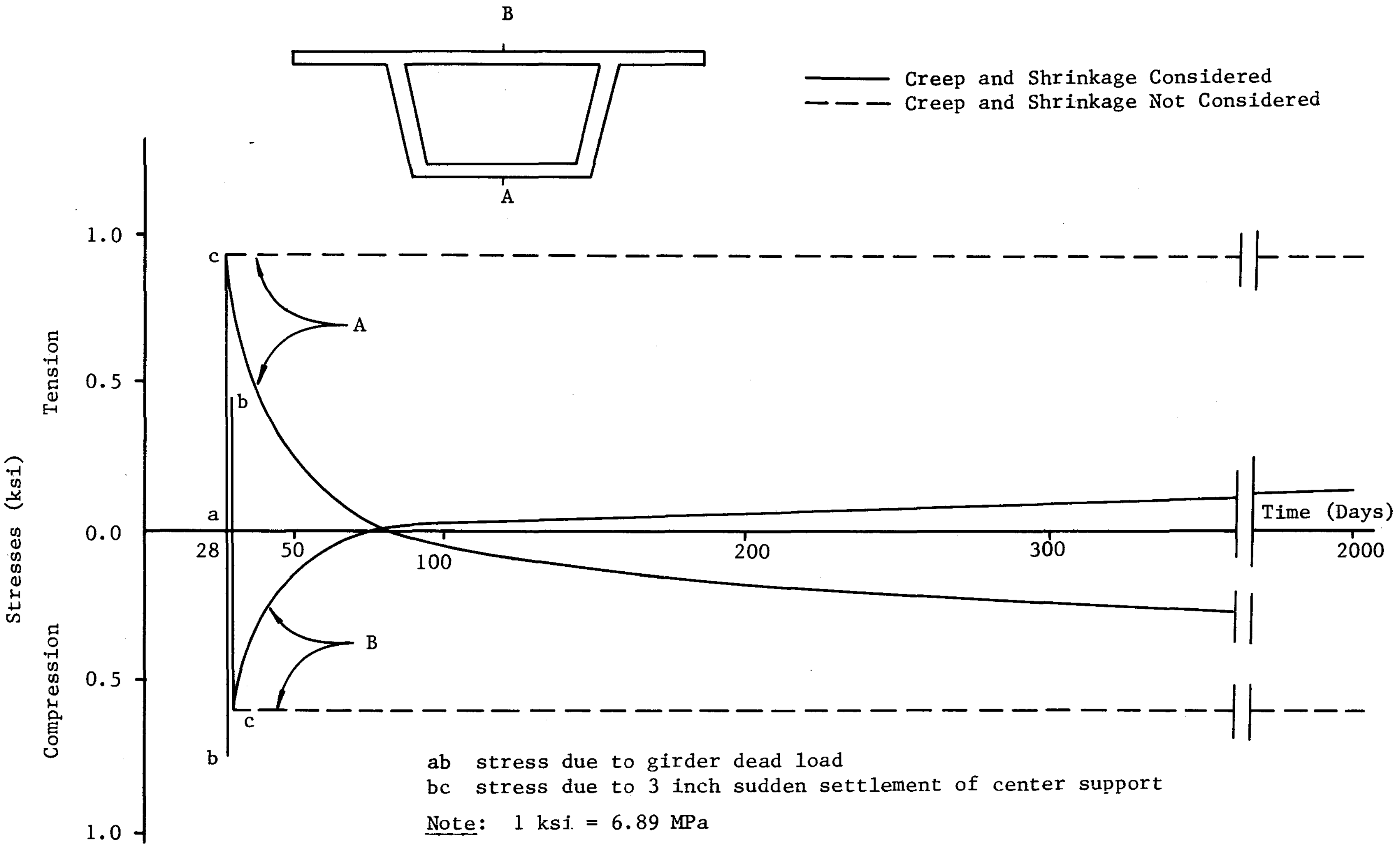


Figure 33. Time-dependent stresses at center support for two-span continuous concrete box girder bridge with 100-foot spans - sudden settlement of center support.

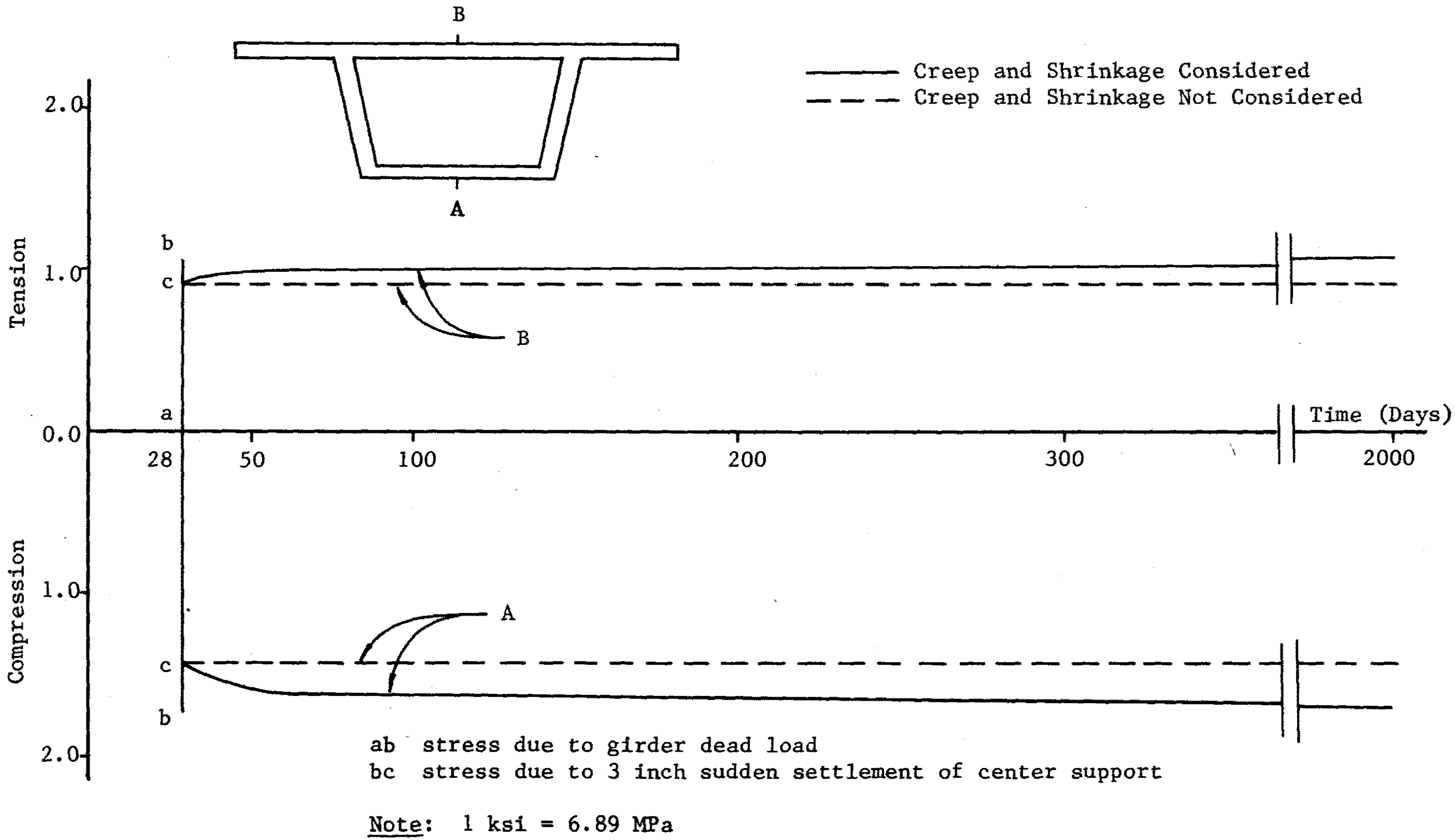


Figure 34. Time-dependent stresses at center support for two-span concrete box girder bridge with 200-foot spans - sudden settlement of center support.

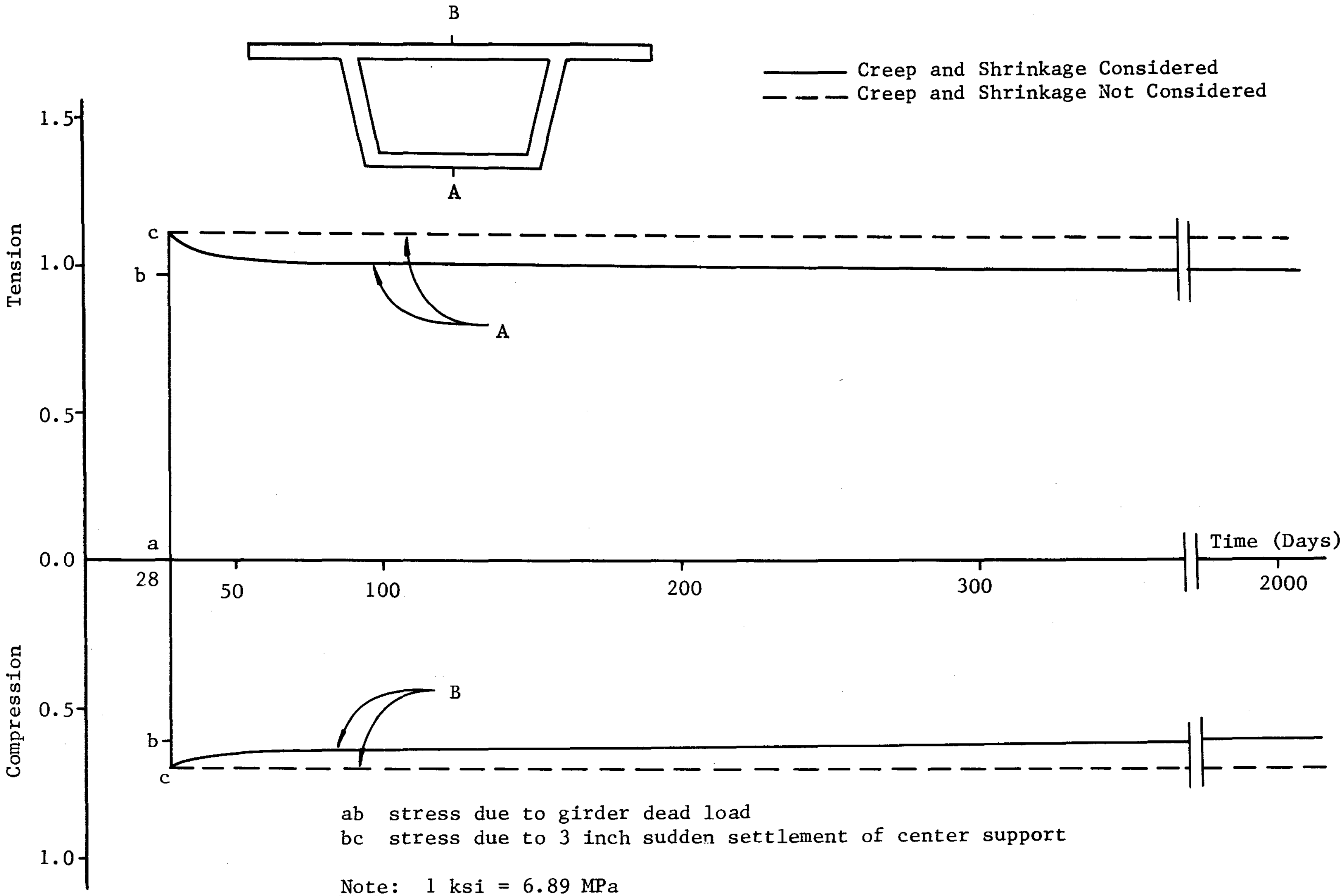


Figure 35. Time-dependent stresses at midspan for two-span concrete box girder bridge with 200-foot spans - sudden settlement of center support.

In the case of the center support stress in the longer span case, the effects of settlement are less dramatic. Immediately after the settlement occurs, the immediate effect is a stress relief. With time, the effects of creep restore the stresses to approximately those due to dead load alone.

Four-Span Post Tensioned Box Girder. As an example of the effects of span length on settlement-induced stresses, a post-tensioned box girder bridge was analyzed for the effects of sudden settlement. This structure assumed the same box section as used in the previous example, with four continuous spans of 200 feet (61.0 meters). For this analysis, dead load, prestressing force and settlement were assumed to act on the structure when the concrete reached an age of 28 days. Draped strands provided a prestressing force to balance approximately 75 percent of the dead load effect.

For this structure, the maximum effects of settlement are produced by settlement at the first interior support. By considering various loading patterns for live loads, it was determined that the maximum overall stresses occur at the second interior support. In figure 36, stresses at the second interior support are shown for a 3-inch (76.2 mm) sudden settlement at the first interior support. A "spike" on the curves shows the maximum live load effect at this section.

The four-span structure is inherently stiffer than the two-span structure, so the resulting settlement stresses are somewhat higher for bridges with the same span length. However, for this 200-foot (61.0 meters) span, the overall magnitude of settlement stresses is still relatively small.

Summary. For two- and four-span continuous box girders with 200 feet (61.0 meters) spans, the effects of a sudden support settlement of up to 3 inches (76.2 mm) are very small, and may be ignored for practical purposes. For spans of 100 feet (61.0 meters), the ratio of settlement to dead load stresses is significantly higher. In this case, midspan stresses are more than doubled just after the settlement occurs, and a stress increase of almost 70 percent remains after stresses are relieved by creep. A significant amount of tension cracking may be expected at midspan. At the

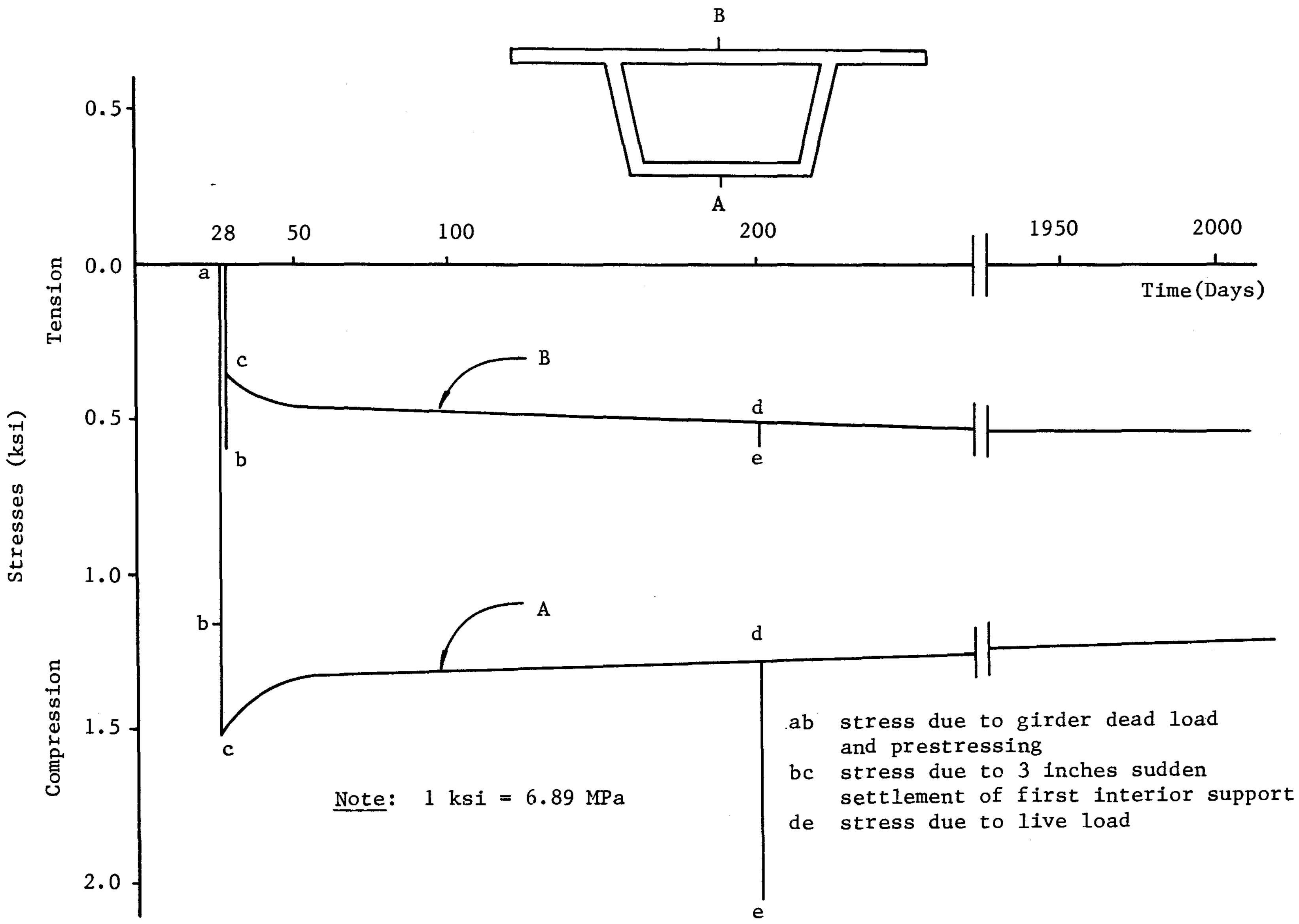


Figure 36. Time-dependent stresses at center support of four-span continuous post-tensioned concrete box girder bridge with 200-foot spans - sudden settlement of first interior support.

center suport, a 3 inch (76.2 mm) suddenly applied settlement results in a stress reversal, producing a high tension stress and tension cracking in the bottom flange of the box section. Since the ratio of settlement to dead load stresses varies inversely as the fourth power of span length, this stress reversal might be expected in similar two-span continuous box girders with spans less than about 125 feet (38.1 meters).

DEVELOPMENT OF DESIGN METHODOLOGY

The results of the field studies and analytical studies described above were used in the consideration of a number of possible methodologies for the design of highway bridges and their foundations that would embody a rational set of criteria for tolerable bridge movements. This resulted in the selection of a methodology that 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. This design methodology and some of the tolerable movement criteria that have been considered for use with this procedure are outlined below. The reader is referred to the Final Report for a detailed description of the procedure and recommendations for its implementation.

Basic Design Procedure

The methodology for the design of bridge systems that evolved from this research is presented schematically in figure 37. It is envisioned that in practice a trial structure type or types would be selected and a preliminary design or designs of the superstructure would be prepared, based upon geometric constraints and a preliminary assessment of subsurface conditions, as illustrated in figure 37. A detailed program of subsurface exploration, sampling and testing would then be undertaken, and, based upon the results of these studies, a trial foundation system or systems would be selected. At this stage, it appears reasonable that spread footing foundations should be considered as one viable alternative, pending further analysis, unless there is some compelling reason for the exclusive use of deep foundations, such as, for example, the possibility of streambed scour or the presence of compressible foundation soils that could lead to very large differential settlements.

Appropriate geotechnical analyses would then be conducted, as indicated in figure 37. In the case of spread footings, these analyses should include an evaluation of bearing capacity, estimates of long term

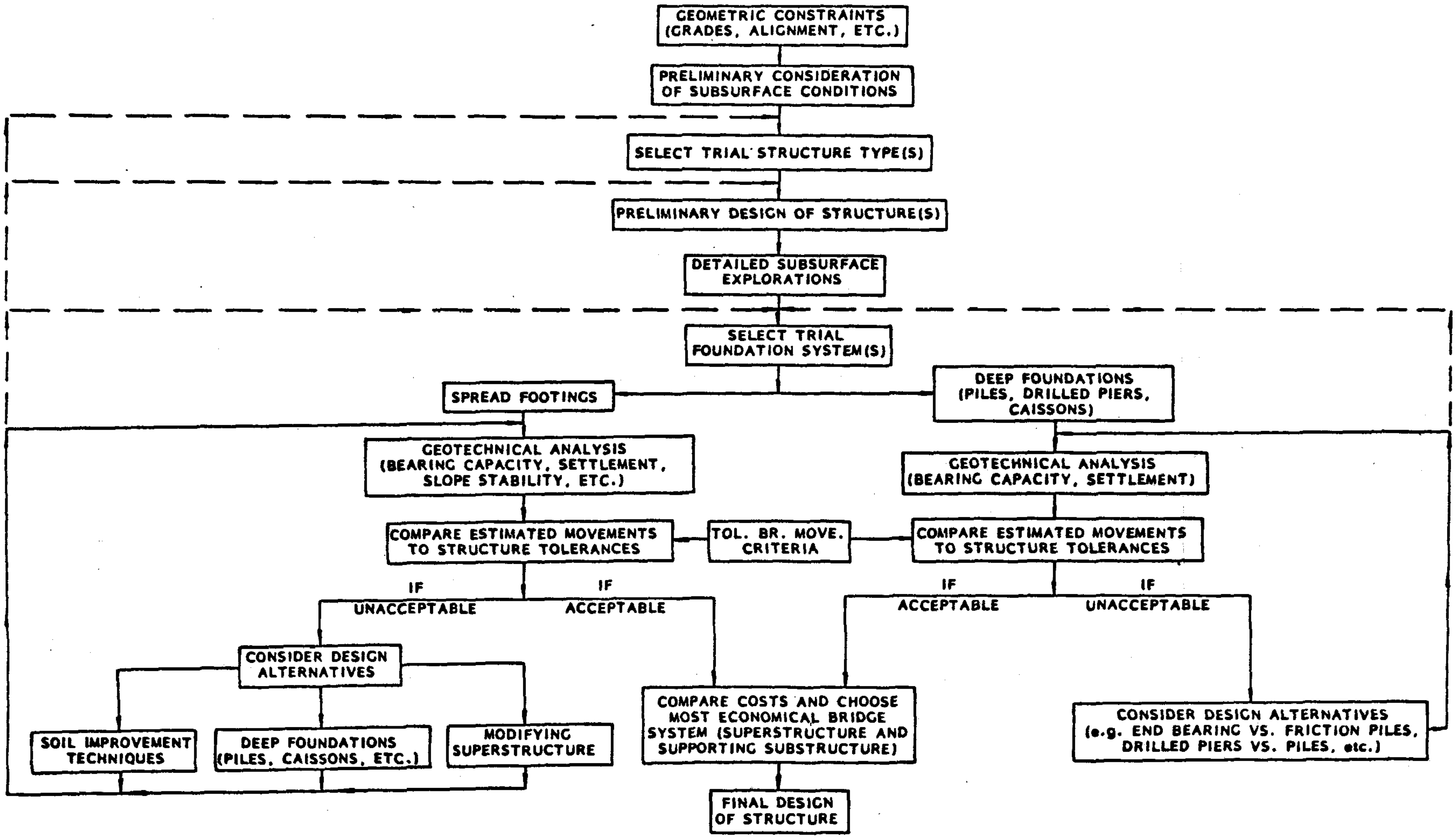


Figure 37. Schematic representation of proposed methodology for the design of highway bridge systems.

total and differential settlements and some appraisal of the potential for horizontal movements, including an evaluation of lateral earth pressures and the stability of approach embankments. Similar analyses should be conducted in the case of deep foundations. At this point in the design procedure, it is envisioned that the tolerance of the bridge superstructure(s) to the estimated foundation movements would be evaluated using tolerable bridge movement criteria such as those described below.

If it is determined that the original superstructure design(s) could tolerate the anticipated foundation movements, then the designer would proceed to perform appropriate cost comparisons and select the most economical bridge system (superstructure and supporting foundation). On the other hand, if it is found that the original superstructure design(s) could not tolerate the anticipated foundation movements, then the designer could consider a variety of design alternatives, as shown in figure 37. In the case of spread footing foundations, these could include (a) the use of piles or other deep foundations; (b) the use of a number of available soil and site improvement techniques, in an effort to minimize post construction movements; (c) the modification of the superstructure design to one that could better tolerate the anticipated foundation movements; or (d) some combination of these methods. This procedure will often lead to one or more new or revised designs, or an alteration of the subsurface conditions, requiring a return to an intermediate step in the design and analysis process, as indicated in figure 37. In the case of deep foundations, the consideration of design alternatives is somewhat limited. Nevertheless, the designer could consider alternate types of pile foundations, e.g. steel H-piles rather than cast-in-place concrete piles, or alternate types of deep foundations, such as drilled piers or caissons rather than some type of driven pile foundation. This procedure could also lead to a new or revised design requiring a return to an intermediate step in the design and analysis process. Ultimately, it is anticipated that this process will lead to two or more designs that can be expected to provide satisfactory long-term performance, thus permitting a selection of the final design based on cost effectiveness.

Tolerable Movement Criteria

As a result of both field and analytical studies, it became clear that the criteria for tolerable bridge movements should include consideration of both strength and serviceability. The strength criteria must insure that any stress increases in a bridge system caused by the predicted foundation movements do not adversely affect the long term load carrying capacity of the structure. The serviceability criteria, on the other hand, must insure rider comfort and the control of functional distress. The fact that the predicted foundation movements do not immediately jeopardize the load carrying capacity of the bridge does not necessarily insure the long term usefulness and safety of the structure. If the foundation movements significantly reduce the ability of a bridge to serve its intended function, then these movements may be intolerable, even though the load carrying capacity of the bridge is not seriously impaired. For example, movements that could lead to poor riding quality, reduced clearance at overpasses, deck cracking, bearing damage, and other kinds of functional distress that require costly maintenance must be controlled properly for satisfactory long term bridge performance. This control can be provided by adopting appropriate tolerable movement criteria based on serviceability.

In the following discussion of tolerable movement criteria, the emphasis has been placed, for the time being, on steel bridges, and only limited consideration of tolerable movement criteria for concrete bridges has been included until some of the complexities associated with the time-dependent behavior of these structures can be resolved.

Strength Criteria

From a strength standpoint, consideration of differential settlements will not require any change in the current design procedure for simply supported steel bridges with rectangular deck shapes. This is because of the fact that no significant internal stresses will develop in simply supported bridge members as a result of differential settlements. However, for continuous bridges, the superstructure design must embody some consideration of the possible increase in stress that could result from differential movement of the foundation elements.

Based on Allowable Overstress. Both field and analytical studies have shown that, depending upon span length and stiffness, many continuous bridges may experience relatively modest increases in stress because of foundation movements. These findings suggested that one basis for the establishment of strength criteria might be to define limits of overstress that would be acceptable for various bridge systems without risking serious damage. There are ample precedents for such criteria in existing American Association of State Highway and Transportation Officials (AASHTO) standards for design and maintenance and in other building codes and design specifications. However, these criteria generally involve temporary or transient overloads. For continuous bridges that experience differential settlements, the induced stresses might be permanent, unless remedial jacking operations are undertaken to relieve the overstress. Moreover, the increased stress levels could conceivably reduce the overall safety of the structure with respect to its ultimate load carrying capacity, and the risk of damage from fatigue could increase. Nevertheless, the design on the basis of a relatively small overstress might constitute an attractive alternative to the use of costly deep foundations to prevent differential movements.

In order to explore this alternative, an extensive literature search was conducted in an effort to find published accounts of research dealing with the measured behavior of bridges under load. It was found that there was a substantial body of literature describing measurements of the strains in a wide variety of steel highway bridges in the United States and Canada under actual highway loading or simulated highway loading using test trucks. In fact, measurements were available on over seventy such bridges. In general, the interpretation of these measured strains in terms of stress history showed that, under typical highway loading conditions, the peak live load stresses occurred relatively infrequently, and their magnitude was usually below the level that would have been expected based on current design criteria.

However, in order to investigate this general finding in greater detail, six of these case histories that were particularly well documented were selected for further study. These included five three-span continuous steel bridges and one four-span continuous steel structure. A specially prepared computer program was then used to compute the live load stresses in the test bridges under AASHTO HS20-44 truck loading at the same locations at which the strain measurements had been recorded in the field. The results of these computations permitted detailed comparisons to be made between live load stresses based on field measurements and computed stresses based on AASHTO HS20-44 truck loading. A typical comparison of this type is shown in table 12. It is clear from these data that the maximum computed live load stresses, based on current design criteria, are substantially higher than the stresses based on field measurements. The reader is referred to the Final Report for the complete results of this study.

For the bridges included in this study, it appears that a modest increase in stress level, as a result of differential settlement, could be tolerated without resulting in the structure being seriously overstressed. Although, in terms of the existing design specifications, these additional differential settlement stresses would theoretically constitute an overstress, the data suggest that the actual stresses could be kept at tolerable levels by setting appropriate limits on this theoretical overstress. The establishment of such limits, of course, will require further study.

Based Upon Working Stress Design For Service Loads. A more conservative approach to the establishment of a tolerable movement criterion based upon strength would be to adopt a design procedure that insures that the structure can accomodate the anticipated foundation movements without exceeding the allowable stresses provided by existing AASHTO specifications. Although, in the context of the research described herein, this approach establishes one type of tolerable movement criteria based upon strength, it also constitutes one of the design alternatives (modifying superstructure) in the design procedure illustrated in figure 37. As such, it should probably be considered in competition with other possible design alternatives in terms of effectiveness and economy.

One method of implementing this approach for both steel and concrete bridges would be simply to design the bridge to accomodate the anticipated settlements. For concrete bridges, these designs should include

Table 12. Comparison between measured and computed live load stresses for a typical three-span[a] continuous steel bridge.

Location of Point at Which Measurement was Made	Live Load Stresses in ksi		
	Measured		
	Stress Range	Frequencey in Percent	Computed Using HS20-44 Loading
Span No. 1	0 - 2.0	73.0	8.00
	2 - 2.5	9.0	
	2.5 - 3.0	10.5	
	3.0 - 3.5	5.2	
	3.5 - 4.0	1.8	
	> 4.0	<<1.0	
Span No. 2	0 - 2.0	67.0	9.63
	2 - 2.5	12.0	
	2.5 - 3.0	10.0	
	3.0 - 3.5	7.0	
	3.5 - 4.0	3.2	
	> 4.0[b]	<<1.0	
Edge of Cover Plate	0 - 2.0	93.5	6.24
	2.0 - 2.5	5.7	
	> 2.5	<<1.0	
At Piers No. 1 and No. 2	0 - 2.0	98.5	6.6
	2.0 - 2.5	1.0	
	> 2.5	<<1.0	

[a]Spans lengths are 44, 63 and 44 feet (13.4, 19.2 and 13.4 meters).
[b]Maximum measured stress level in Span No. 2 was 4.5 to 5.0 ksi at a 0.2 percent frequency of occurrence.
Note: 1 ksi = 6.9 MPa.

consideration of creep and shrinkage, and, in the case of spans in excess of 200 feet (61.0 meters), differential settlements up to three inches (76.2 mm) can be safely ignored.

Another method of implementing this approach for steel bridges would be to adopt a design procedure based on working stress design for service loads, reducing the allowable stress by a value equivalent to the stress increase caused by the predicted differential settlements. This design procedure would involve three basic steps: (a) the design of the bridge under the assumption that no movement will take place using the AASHTO working stress design procedures, but using reduced allowable stresses in the top and bottom fibers to adjust for anticipated settlement; (b) the comparison of the predicted movements with tolerable movements established on the basis of serviceability criteria; and (c) the modification of the original design in order to satisfy minimum strength and serviceability criteria. Of course, the third step might not be necessary if the comparisons embodied in step (b) show that the original design can safely tolerate the anticipated movements. It should be noted that the use of the procedure contained in step (a) will produce the same results as if the bridge were designed from the beginning to accommodate the anticipated settlements, although the availability of design aids such as those given in figure 27 and 28 make the former method somewhat easier. In practice, the designer could use the appropriate design aids, along with predicted values of foundation settlements, to solve for maximum positive and negative settlement stresses. The resulting values could then be subtracted from the AASHTO limit of 0.55 f_y in order to obtain allowable stresses for use in design. The primary advantage that this method has over alternate procedures is that it provides a uniform method of design that is applicable regardless of whether or not any foundation movement is anticipated. However, this procedure will lead to somewhat heavier sections than the design based on an allowable overstress as discussed above.

Based on Load Factor for Settlement Stresses. In an effort to overcome some of the limitations of the approaches to establishment of tolerable movement criteria based upon strength, discussed above, the possible application of a design procedure based upon the load factor concept was studied in some detail. Such procedures have become widely accepted and are recognized as being more realistic than working stress design. The research efforts that were undertaken in this connection concentrated on the development of a load factor for settlement stresses. However, it was recognized that the establishment of a load factor for settlement stresses on a strictly theoretical basis required a knowledge of the statistical reliability of the settlement prediction. Although, as noted earlier in this report, reasonably reliable settlement predictions can be made as long as good subsurface information and laboratory test results are available, after some study it was concluded that there were insufficient field data available upon which to determine the statistical reliability of settlement predictions for bridge foundations. Consequently, it was not possible to develop a load factor for settlement stresses on a rational basis. However, it does appear that in the light of the load factors and coefficients used in the existing AASHTO Specifications, it may be possible to establish a reasonable empirical load factor for settlement stresses. This is discussed in greater detail in the Final Report.

Serviceability Criteria

Serviceability criteria deal with the maintenance of rider comfort and the control of functional distress. The types of movements that were identified as being sufficiently important for consideration with respect to serviceability are: (a) vertical displacements, including total settlement, differential settlements, longitudinal angular distortion, and transverse angular distortion; (b) horizontal displacements, including translation, differential translation, and tilting; and (c) dynamic displacements.

The establishment of realistic limits on these movements can only be accomplished if sufficient and relevant field data are available. Based upon the data accumulated during this study, limits could only be established on some of these movements. The establishment and implementation of criteria for limiting the remaining types of movements will have to await the accumulation of additional relevant field data on these movements and their effects. For example, based on the existing field data presented above, it is clear that horizontal movements of abutments and piers, either by translation or tilting, must be very carefully controlled in order to avoid structural damage. Although setting tolerable limits on these horizontal movements has not been difficult, at present we do not have well established procedures for predicting these horizontal movements with reasonable reliability.

On the basis of the data that were assembled during the course of this research, tolerable limits were established on (a) differential settlements, both in terms of angular distortion and deck cracking; (b) horizontal movement of abutments; and (c) bridge vibrations.

Differential Settlements. The field data assembled during the course of this project indicated that structural damage requiring costly maintenance tended to occur more frequently as the longitudinal angular distortion (differential settlement/span length) increased. In order to evaluate this phenomenon, the frequency of occurrence of the various ranges of tolerable and intolerable angular distortions was studied for both simply supported and continuous steel bridges. The results of this study, presented earlier in this report, showed that, for continuous steel bridges, 93.7 percent of the angular distortions less than 0.004 were considered to be tolerable. In contrast, for simply supported steel bridges, 97.2 percent of the angular distortions less than 0.005 were reported as being tolerable. Similar results were reported for the concrete bridges. It was found that the tolerance of both types of bridges to angular distortions dropped very rapidly for values greater than these. A statistical analysis of these field data showed that there was a very high probability that angular distortions less than 0.004 and 0.005 would be tolerable for continuous and simply supported bridges, respectively, of both steel and concrete. Tolerable limits on angular distortion were thus established at these values.

The potential for deck cracking as a result of differential settlement is normally restricted to continuous bridges. This is a function of the tensile stress developed over the supports (i.e., in the negative moment region), the allowable tensile stress in the deck concrete, and the spacing and size of negative reinforcement. The maximum negative stress (tension at the top of the bridge deck) due to anticipated vertical differential settlement of abutments or piers can be determined analytically, or by the use of appropriate design aids, such as figure 27. The total maximum negative stress is then obtained by adding this value to the negative stress produced at the same point by the design live and dead loads. This total maximum negative stress is limited to the allowable value given by Equation 6-30 in Section 1.5.39 of the AASHTO Specifications. In essence, this comparison, between the total maximum negative stress and the limiting stress provided for in the AASHTO Specifications, constitutes a check on the tolerance of the bridge to the anticipated differential settlements in terms of deck cracking. If it is found that the computed total maximum negative stress exceeds the AASHTO requirement, then some adjustment may be required in the size and/or spacing of the deck reinforcement.

Horizontal Movements of Abutments. As noted earlier in this report, bridges that experienced either horizontal movement alone or horizontal movement in conjunction with differential vertical movement, had a high frequency of damaging structural effects, suggesting that horizontal movements are much more critical than vertical movements in causing structural damage. In terms of horizontal movements alone, movements less than 2.0 inches (50.8 mm) were considered to be tolerable in 88.8 percent of the cases. When accompanied by vertical movements, horizontal movements less than 2.0 inches (50.8 mm) were considered to be tolerable in only 60.0 percent of the cases. However, horizontal movements of 1.0 inch (25.4 mm) and less were almost always reported as being tolerable. On the basis of these data, it appeared that a logical tolerable limit on horizontal movements could be established at a value somewhere between 1.0 and 2.0 inches (25.4 and 50.8 mm). Consequently, it is suggested that horizontal movements of abutments be limited to 1.5 inches. However, it is evident that more consideration needs to be directed to the possibility of horizontal movements and their potential effects during the design stage. A study of the factors contributing to horizontal movements of abutments and methods for limiting these movements would also be desirable.

Bridge Vibrations. As noted earlier in this report, it was found that a substantial increase in dynamic deflections leading to uncomfortable levels of human response were likely to occur if the "reasonance factor", i.e. the ratio of the forced (ω_f) to natural (ω_n) frequencies, approached one. This relationship can be used to determine if a proposed bridge has sufficient mass and stiffness to prevent excessive dynamic deflections. The details of this procedure are presented in the Interim Report.

SUMMARY

Field Studies

The data resulting from the field studies showed that a rather wide range of both vertical and horizontal movements of substructure elements has been experienced by a substantial number of highway bridges throughout the United States and Canada. Generally, abutment movements occurred much more frequently than pier movements. Although both the frequency and magnitude of vertical movements were often substantially greater than horizontal movements, the horizontal movements generally tended to be more damaging to bridge superstructures. The data suggest that more consideration needs to be directed to the potential effects of horizontal movements during the design stage, particularly for perched and spill-through abutments on fills and piers located near the toe of approach embankments. Furthermore, care should be exercised in the design and construction of approach embankments in order to eliminate this important potential source of damaging post-construction movements. The data show that precompression and/or the use of a waiting period, following embankment construction and prior to abutment construction, can be helpful in this regard.

The field studies also showed that, for both abutments and piers that experienced foundation movements, substantially more were founded on spread footings than on piles. However, the average magnitude of the movements of pile foundations were slightly longer than those of the spread footing foundations. Since the data included in these field studies represent the observed behavior of only those bridge foundations that experienced foundation movements, no inferences can be drawn with respect to the relative performance of the different foundation systems (i.e. piles vs. spread footings). However, these findings do suggest the need for a more detailed examination of those cases of pile foundation movement, in order to determine the reasons for the failure of the pile foundations to serve their intended function of eliminating or minimizing substructure movements.

The results of this study have shown that, depending on type of spans, length and stiffness of spans, and the type of construction material, many highway bridges can tolerate significant magnitudes of total and differential vertical settlement without becoming seriously overstressed, sustaining serious structural damage, or suffering impaired riding quality. In particular, it was found that a longitudinal angular distortion (differential settlement/span length) of 0.004 would most likely be tolerable for continuous bridges of both steel and concrete, while a value of angular distortion of 0.005 would be a more suitable limit for simply supported bridges.

It was found that the settlement data for bridges founded on sands was insufficient to permit a valid assessment of the reliability of settlement prediction techniques for sands. However, it was shown that reasonably reliable predictions of the settlement of bridges founded on clays could be obtained as long as adequate subsurface information and laboratory test data were available.

Analytical Studies

The data resulting from the analytical evaluation of the effects of support settlements and dynamic vibrations on continuous steel bridges show that the tolerance of any given bridge to movements of these types is dependent upon a number of structural and geometric parameters of the system, such as flexural rigidity (EI), stiffness (I/ℓ), magnitude of differential settlement, number of spans, span length, vehicle velocity, axle spacing and structural mass.

For continuous two- and four-span steel bridges, it was found that differential settlements of one inch (25.4 mm) or more would be intolerable for span lengths up to 50 feet (18.3 meters) because of the rather significant increase in stresses caused by these settlements (see table 7). However, for span lengths between 100 and 200 feet (30.5 and 61.0 meters), the stress increases caused by differential settlements up to 3 inches (76.2 mm) were quite modest, and for span lengths in excess of 200 feet (61.0 meters), the stress increases caused by 3 inch (76.2 mm) differential settlements were negligible. For span lengths ranging from 50 feet (18.3 meters) to 200 feet (61.0 meters), a 3 inch (76.2 mm) differential settlement would most likely be tolerable if the stiffness (I/ℓ) were

20 in^3 (327,742 mm^3) or less. However, care should be exercised in implementing these findings, since the stress increases in continuous steel bridges caused by differential settlement are very sensitive to the stiffness (I/ℓ , and it is not uncommon for a design to result in a stiffness that is in excess of 20 in^3 (327,742 mm^3).

The stress increases produced in the two-span continuous parallel and non-parallel chord steel trusses by differential support settlements up to 3 inches (76.2 mm) in magnitude were less than 10 percent and, in most instances, were negligible.

A limited analytical study of the effects of instantaneous and time-dependent support settlements on continuous concrete bridges was performed considering the influence of dead loads, live loads, prestressing loads and the effects of shrinkage and creep. It was found that consideration of time-dependent material properties is absolutely necessary to accurately assess the effects of support settlements on concrete bridge superstructures.

"Real world" settlements are most likely to be gradual in nature. However, sudden settlements are much easier to analyze, and the stresses calculated on the basis of assumed sudden settlement do provide a guide to the overall significance of settlement effects on concrete bridges. Creep may reduce the effect of settlement to about one-third of its initial value, if the settlement occurs early in the life of the structure. Settlements occurring after a few months cannot be reduced as significantly.

The analyses reported herein tend to confirm intuitive estimates of the effects of support settlements on continuous concrete bridges. For example, as expected, it was found that settlement effects increase with overall stiffness of the structure. Thus, a two-span continuous structure has settlement stresses about 43 percent less than a four-span structure with the same cross section. In terms of structural configuration, settlement-induced stresses increase approximately as the ratio of d/ℓ^2, where d is the overall depth of the cross section and ℓ is the span length. However, the ratio of settlement stresses to dead load stresses increases as the ratio I/ℓ^4, where I is the moment of inertia for the cross section. Overall, the span length was found to be the most significant term governing settlement stresses.

Continuous concrete bridges with span lengths less than 100 feet (30.5 meters) are very sensitive to differential foundation movements, while those with span lengths of 200 feet (61.0 meters) or more can tolerate differential settlements as large as three inches (76.2 mm) with only a relatively small change in total stresses.

Design Methodology

A basic design procedure has been suggested which will permit a systems approach to be used for the design of highway bridges. In this procedure, an initial design is prepared on the assumption that no foundation movement will take place. The potential foundation movements are then estimated and the tolerance of the structure to these movements is evaluated using tolerable movement criteria based upon both strength and serviceability. If the original design will not tolerate the estimated movements, then a variety of design alternatives can be considered in order to reduce the potential movements or increase the tolerance of the structure to these movements. It is anticipated that this procedure will result in the optimization of the design of the superstructure and its supporting substructure as a single integrated system offering the best combination of long-term performance and economy.

The results of both field and analytical studies were utilized in an investigation aimed at developing tolerable movement criteria based upon both strength and serviceability. Because of the complexities associated with the time dependent behavior of concrete bridges, this investigation concentrated on steel bridges, and only limited consideration was given to tolerable movement criteria for concrete bridges. It was found that a basis does exist for the establishment of strength criteria for steel bridges based on defining limits of "overstress", caused by differential foundation movements, that would be acceptable for various bridge systems without risking serious damage. An alternate, more conservative, procedure that was investigated involves the design of bridges under the assumption that no settlement will take place, using the AASHTO working stress design procedure, with the allowable stress being reduced to compensate for anticipated settlements. The resulting design is then checked for compliance with serviceability criteria based on limiting longitudinal angular distortion, horizontal movement of abutments, deck cracking and bridge vibrations. Convenient equations and graphical design aids were developed to facilitate these operations. This procedure may lead to the modification of the original design in order to satisfy minimum strength and serviceability criteria. Another approach that was studied was the use of load factor design, which has been increasing in popularity in recent years. Although it was found that there was insufficient data presently available on the statistical reliability of settlement predictions to permit the development of a load factor for settlement stresses on a strictly theoretical basis, it was concluded that the selection of a reasonable empirical load factor for settlement stresses may be possible. Serviceability criteria were developed based on limiting longitudinal angular distortion, horizontal movement of abutments, deck cracking and bridge vibrations.

☆U.S. Government Printing Office : 1986 - 620-477/40652