Tolerable Movement Criteria for Highway Bridges Vol. 1

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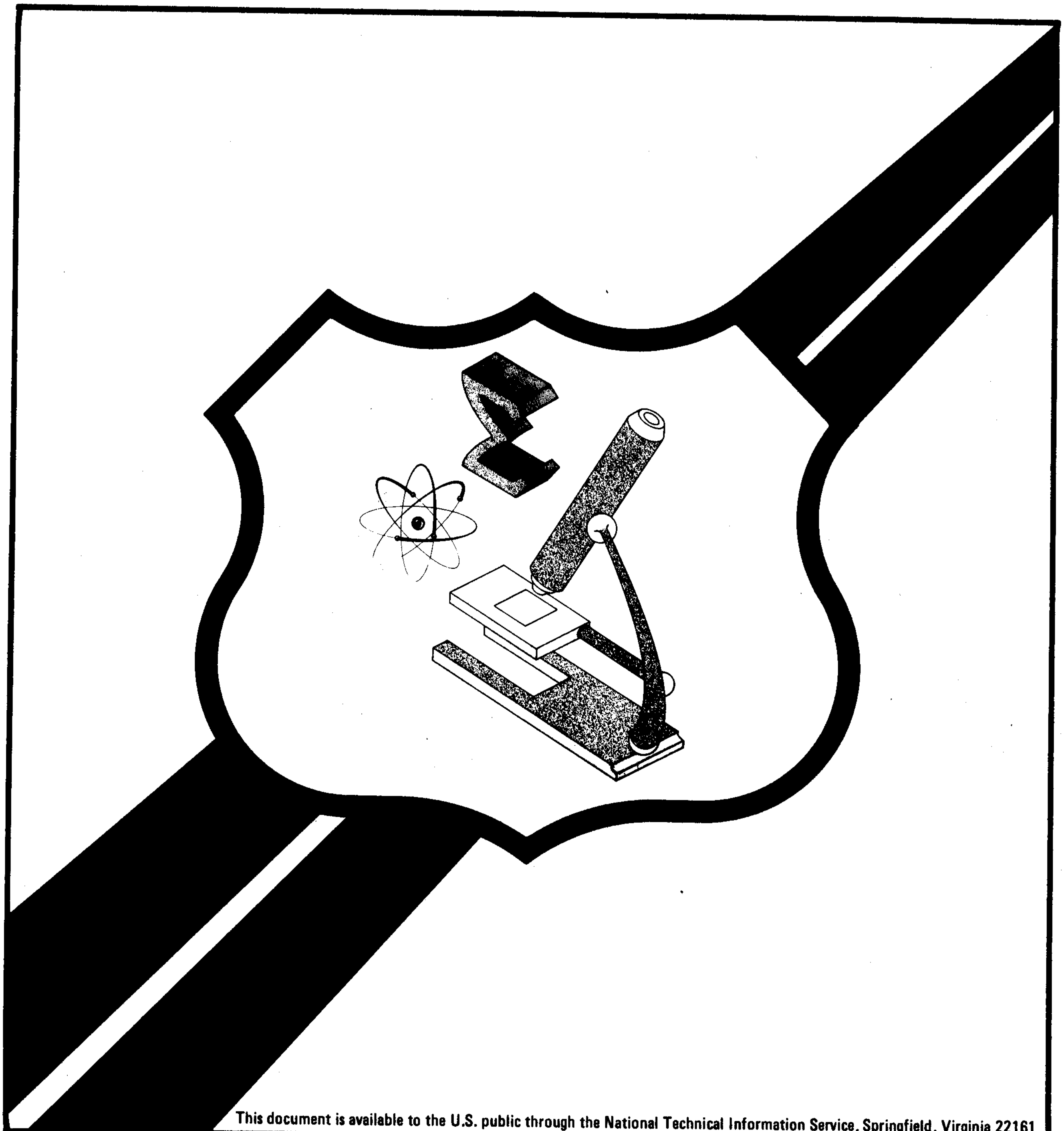

FOREWORD

This report describes a new method for the design of steel bridges that uses a rational set of tolerable movement criteria which are based on strength and serviceability. The supporting data from analytical and field performance studies are also described for the steel bridges, plus the results of a preliminary analysis of concrete bridges. This report will be of interest to bridge engineers and geotechnical specialists concerned with allowable foundation movements for highway bridges.

This report presents the results of West Virginia University Research Project, "Tolerable Movement Criteria for Highway Bridges." The program was conducted for the Federal Highway Administration, Office of Research, Washington, D.C., under contract DOT-FH-11-9440. This interim report covers the period of research and development from June 28, 1978 to December 31, 1981.

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NOTICE

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16. Abstract

This investigation included (a) the collection of field data on bridge movements, structural damage, and tolerance to movements for a large number of bridges in the United States and Canada; (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 bridge structures of different span lengths and stiffnesses; and (c) the development of a methodology for the design of bridges that embodies a rational set of criteria for tolerable movement of steel bridges based on strength and serviceability.

This volume is an interim report on the entire study.

This volume is the first in a series. The others in the series are:

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II	81/163	Appendix A - Bridge Data Storage and Retrieval System
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1. INTRODUCTION

In current practice, the design of highway bridges commonly begins with the selection of a structure type, based on geometric, functional, architectural, engineering and economic considerations. A preliminary design is prepared and used as the basis for initiating a detailed geotechnical investigation. A program of subsurface explorations, sampling and testing is then undertaken, and, based on the results of these studies, appropriate geotechnical analyses are conducted. These usually include an evaluation of bearing capacity and estimates of immediate and long term total and differential movements. The resulting estimates are used as a basis for deciding how the structure should be founded in order to provide the best combination of safety and economy. Often, one of the major considerations involved in making this decision is whether or not the proposed structure can tolerate the estimated total and differential movements.

If it should be determined that the bridge structure, as originally designed, is unable to tolerate the anticipated foundation movements, then a variety of design alternatives could be considered. These include the use of piles or other deep foundations, the use of precompression or other soil improvement techniques to minimize or eliminate post construction movements, modification of the structure to a design capable of withstanding the estimated movements, or some combination of these alternatives. Ideally, a cooperative evaluation of the various design alternatives by bridge designer and geotechnical engineer should lead to an 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 investigation described herein was initiated as part of a broad research effort designed to establish design methods and criteria that will permit this systems approach to the design of bridges and their foundations to be utilized routinely. It is concerned with the development of rational criteria for determining whether a proposed bridge structure can tolerate the estimated total and differential movements to which it may be subjected.

A great deal of data has been collected and used as the basis for establishing criteria for tolerable movements of buildings and some industrial structures. Among the most significant published accounts of this work are papers by Skempton and MacDonald (67), Polshin and Tokar (63), Feld (24), Grant, Christian and Vanmarcke (33) and Burland and Wroth (15). Unfortunately, the criteria presented in these papers are not applicable to highway bridges. Because of the lack of well founded criteria for tolerable movements of bridges, the designer is 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. Another less restrictive set of guidelines has been suggested by Thornley (72), who recommended that differential and total settlements under working loads be restricted to 1/4 inch (6.4 mm) and 3/4 inch (19.1 mm) respectively, and that total settlement under 200 percent of the working load be restricted to less than 1 1/2 inches (38.1 mm). The current AASHTO "Standard Specifications for Highway Bridges" (5) states "In general, piling shall be considered when footings cannot, at a reasonable expense be founded on rock or other solid material". Regardless of the intent of these quidelines, their employment in practice has often led to the decision to use piling or other costly deep foundations, without detailed consideration of other design alternatives, such as those mentioned above, that might have resulted in satisfactory performance at a lower overall cost.

It was recognition of the need for the development of more rational criteria for the tolerable movements of bridges that led the Federal Highway Administration to award Contract No. DOT-FH-11-9440 to West Virginia University to conduct the research described in this report. Although this 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 the collection and analysis of field data on movements, structural damage and the tolerance to movements for a large number of bridges in the United States and Canada; (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. This volume contains a summary of these studies and their results to date. Additional details are presented in Volumes II and III.

2. LITERATURE REVIEW

The initial approach to the literature review was to utilize published indices and abstracts to identify appropriate references relating to bridge movements and their effects. These included the Highway Research Abstracts, the Road Research Laboratory Abstracts, the British Technology Index, the Applied Science and Technology Index, the Engineering Index, the Geodex Structural and Geotechnical Information Service and the Highway Research Information Service (HRIS). Each of the pertinent references, identified in this manner, was obtained, reproduced, and placed in notebooks for future reference. The reference lists contained in each of these publications were reviewed and any pertinent references not previously identified were secured, reproduced, and placed in the literature review notebooks. This process was continued until no additional pertinent references or cross-references could be identified.

As an outgrowth of this rather comprehensive process, a substantial number of references were collected dealing with the investigation of approach embankments (9,13,21,34,37-39,51,52,55,60) and bridge foundation movements (7,9,11,13,28,29,32,43,45,50,53,54,61,64-66,69,74,77). These references are discussed in some detail in Volume III of this report. However, it was found that until recently there was virtually nothing of a specific nature in the literature with respect to the tolerable movement of bridges.

In an effort to gain some insight into the ability of highway bridge structures to withstand foundation movements, Committee SGF-B3 of the Transportation Research Board conducted a survey of bridge movements in 1967, and later Committee A2K03 (Foundations of Bridges and Other Structures) conducted a more comprehensive study, which began in 1975. The results of the 1975 survey were presented in 1978 in papers by Grover (34), Keene (42), Walkinshaw (75), and Bozozuk (12).

Grover (34) reported on the 1961 study of 68 bridges conducted by the Ohio Department of Transportation and the 1975 follow-up study of 79 bridges. The 1961 study showed that 90 percent of the bridges had abutment settlements of four inches (101.6 mm) or less and only 20 percent had settlements of one inch (25.4 mm) or less. On the basis of this study, revisions to the bridge design policy and construction specifications were adopted. The design revision dictated the use of piles at all abutments located in embankment fills, the addition of more positive drainage behind abutments and the increased use of instrumentation to monitor the performance of approach embankments. The specification revision provided for increased compaction of bridge approach embankments. The follow-up study in 1975, which was conducted on 79 bridges that had been designed and constructed using the revised policies, showed that only 20 percent of the abutments had measurable movements. Based on the experience gained during these studies, Grover concluded that settlements of one inch (25.4 mm) or less can be classified as tolerable and will not adversely affect the riding quality of bridges or cause structural damage. Further, Grover concluded that settlements of two inches (50.8 mm) to three inches (76.2 mm) would be noticeable in terms of reduced riding quality, but that only minor structural damage, if any, would occur. However, settlements of four inches (101.6 mm) or more were judged to be intolerable, both in terms of riding quality and the potential for structural damage.

Keene (42) described movements of seven of the most significant of the bridges reported on by Connecticut. He did not suggest any specific numerical limits for tolerable movements but used these case histories to illustrate his contention that the definition of tolerable movements is variable and must be defined in terms of (a) amount of movement, (b) type of structure, (c) effect on each part of the structure, (d) cost of alternative choices, (e) effects on the traveling public, (f) subjective reasons, and (g) apprehension during design.

Walkinshaw (75) reported on the results of the 1975 survey for a total of 35 bridges from 10 western states. Based on these results, he concluded that horizontal movements were usually the most critical, and that structural distress usually accompanied horizontal movements of two inches (50.8 mm) or more. Although some very large vertical movements were reported as being tolerable from a structural standpoint, poor riding quality was usually reported once the settlement exceeded 2.5 inches (63.5 mm). Detrimental movements of smaller magnitudes were reported when both horizontal and vertical movements occurred simultaneously. Walkinshaw concluded that these movements, i.e. two inches (50.8 mm) horizontally, and 2.5 inches (63.5 mm) vertically, appear to be reasonable limits for tolerable movements for many structures.

Bozozuk (12) attempted to summarize all of the data produced by the 1975 survey and the reported field ratings on the tolerance of bridges to movement to suggest critieria for tolerable movements. He concluded that vertical movements less than 50 millimeters (2 inches) and horizontal movements less than 25 millimeters (1 inch) would be tolerable. Vertical movements of from 50 to 100 millimeters (2 to 4 inches) and horizontal movements of from 25 to 50 millimeters (1 to 2 inches) were judged to be harmful but tolerable. Vertical movements in excess of 100 millimeters (4 inches) and horizontal movements in excess of 50 millimeters (2 inches) were classified as intolerable. Bozozuk's suggested criteria were seriously questioned by Stermac (68), in his discussion of Bozozuk's paper, on the grounds that they were developed without consideration of the bridge type, width, span length and type of movement (total or differential).

Thus, in spite of the pioneering efforts of Transportation Research Board Committee A2K03 and the large amount of data that it collected on the influence of movements on the performance of bridge structures, no well defined set of criteria for tolerable bridge movements were generally agreed upon.

3. FIELD STUDIES

3.1 Data Collection and Analysis

3.1.1 Sources of Data

The process of collecting field data on bridge movements and their effects began with the acquisition of the survey data in the files of Transportation Research Board Committee A2K03. As noted earlier, a great deal of information was obtained from the 1975 survey conducted by Committee A2K03 and from the previous survey conducted in 1967 by Committee SGF-B3. Both surveys consisted of sending questionnaires to highway agencies throughout the United States and Canada. In addition to identification information, the questionnaires requested information on the year of completion, the type and number of spans, the type of abutment, soil and foundation conditions, estimated and observed movements, and their effects on the structure. The 1975 questionnaire (see Table 1) addressed the question of tolerance to movement, while the 1967 questionnaire did not. In addition to the information requested by the surveys, some of the highway agencies supplied information such as soil reports and design drawings. Overall, information was supplied by 34 states, the District of Columbia and 4 Canadian provinces.

In an effort to supplement the data in the files of Committee A2K03, various highway agencies were asked to supply additional information, including boring logs, settlement data, as-built plans, and tolerance ratings for those bridges that had been included in the 1967 and 1975 surveys. Information was also requested on any bridges that had experienced movement that were not included in the 1967 and 1975 survey responses. Although, in general, the number of responses from the various highway agencies was not as high as had been hoped, supplementary data, including as-built plans, were supplied by 15 states, including Connecticut, Idaho, Indiana, Iowa, Minnesota, Missouri, New Jersey, New York, North Dakota, Oregon, Virginia, Washington, Wisconsin, West Virginia, and Wyoming. In addition, data were obtained on 28 bridges in the state of Washington that were included in a Federal Highway Administration staff study reported by Seguirant (64). Overall, data were available on 204 bridges that had experienced some type of movement. As-built plans have been obtained for 98 of these structures.

During the data collection process, it was found that substantially more data were available on bridge movements and their effects from the States of Connecticut and Washington than from any of the other states that supplied data. Consequently, field trips were made to these states, bridge foundation design and performance were discussed with cognizant state officials, and selected bridges within each state were visited and photographed.

3.1.2 Limitations of the Data, Assumptions and Definitions

The data that were available for analysis have certain limitations that must be recognized. Since some of the data was obtained by questionnaires, the quality of the data was dependent on the information requested in the questionnaire and the completeness and accuracy of the

Table 1.--Questionnaire Used by TRB Committee A2KO3 for its 1975 Survey

TRANSPORTATION RESEARCH BOARD

COMMITTEE A2KO3, FOUNDATIONS OF BRIDGES AND OTHER STRUCTURES

SUBCOMMITTEE ON TOLERABLE MOVEMENTS OF STRUCTURES

MOVEMENT OF STRUCTURE DUE TO SOILS AND/OR FOUNDATIONS

STATE ________ COUNTY OR TOWN ___________ ROUTE NO. ________ CROSSING OVER UNDER _____

YEAR BUILT ______ NO. OF SPANS _______ TYPE OF SPANS ______ HT.APPROX.FILL ______

TYPE OF ABUT. ______________________ TYPE OF FOUNDATIONS ________________________

GENERALIZED SOIL STRATA*

CONSTRUCTION SEQUENCE: Substructure _____ Superstructure _____ Embankment _____

VERTICAL MOVEMENTS:	Abutments	Piers	HORIZONTAL MOVEMENTS:	Abutments	Piers
Estim.			Estim.		
Observed			Observed		

EFFECTS ON STRUCTURE (cracks, opened joints, jammed beams, etc.): Tolerable or** Not Tolerable:

*NOTE FOR SOIL STRATA: Blow counts given are per foot, using the Standard Penetration Test on samples. WC is natural water content. PCL is preconsolidation load (when applicable). OBL is overburden load (when applicable).
**NOTE: Movement is Not Tolerable if damage requires costly maintenance and/or repairs and a more expensive construction to avoid this would have been preferable.

SUBMITTED BY ______________________ TITLE __________________

PLEASE RETURN TO:

information supplied by the respondent. This was also true with respect to the supplementary data supplied by the various highway agencies. In some instances, the data were incomplete or unclear, and there was a general lack of common terminology. Consequently, a number of definitions and simplifying assumptions were adopted in order to generalize the data for classification and analysis (44). For the sake of brevity, a complete description of all of these definitions and simplifying assumptions has been omitted from this volume. However, many of these are self explanatory or will be obvious from the manner in which the data are organized and presented below. Therefore, this volume includes only those definitions and simplifying assumptions that are necessary for an understanding of the various analyses that were performed and their results. The remaining definitions and simplifying assumptions are presented in detail in Volumes II and III of this report.

Most of the data included in the questionnaires and the supplemental data supplied by the various highway agencies were quite specific with regard to foundation movements. However, in a relatively small number of cases there was some question as to the magnitude of movements or to which unit of the substructure the reported movements applied. In those cases, the following assumptions were made relative to foundation movement: (a) if only one magnitude was given for a vertical or horizontal movement of abutments or piers, it was assumed that both abutments or all piers moved that amount, unless otherwise specified; (b) if differential movement or a range of movements were reported for a single abutment or pier, the average movement was used in the analysis; (c) if a range of movements was reported for the abutments, it is assumed that one abutment moved the amount of the lower limit and the other abutment moved the amount of the upper limit, unless otherwise specified; (d) only "observed movements" reported in the questionnaires were considered, unless they were omitted or appeared to be in error, in which case "estimated movements" were considered if listed; and (e) all movements were rounded to the nearest 0.1 inch for this study.

A wide variety of subsurface conditions were reported for the bridges included in the field studies. In order to put these data into manageable form, the reported soil profiles were simplified into eight general categories as follows: (a) "fine-grained soils" - soil profiles of predominately silts and/or clays; (b) "fine-grained soils overlying granular soils" - soil profiles with at least 5 feet of silt and/or clay overlying sand and/or gravel; (c) "granular soils overlying fine grained soils" - soil profiles with at least 5 feet of sand and/or gravel overlying silt and/or clay; (d) "granular soils" - soil profiles of predominantly sands and/or gravels; (e) "interlayered or intermixed soils" - soil profiles with at least two nonadjacent silt and/or clay layers separated by one or more sand and/or gravel layers or soil profiles with a mixture of fine-grained and coarse-grained soils; (f) "miscellaneous soils" - soil profiles with substantial amounts of unspecified fill; (g) "bed-rock" - soil profiles with less than 3 feet of soil over bedrock; and (h) "permafrost soils" - soil profiles of permanently frozen soil.

The various kinds of structural damage reported were broken down into ten primary categories and descriptive terms were assigned to each category. These terms and the descriptions of the specific structural effects included in each category are as follows: (a) "damage to abutments"

- includes cracking and spalling of abutments, abutment footings, abutment pile caps, or abutment slope protection; also included in this category are the opening, closing or damage to abutment joints, the separation of the wingwall from the abutment, and the rupturing or exposure of abutment piles; (b) "damage to piers" - includes cracking and spalling of piers, pier footings, pier pile caps, or struts of diaphragms between pier columns; (c) "vertical displacement" - includes the raising or lowering of the superstructure above or below planned grade or a sag or heave in the deck; structures requiring shimming or jacking as well as truss structures with increased camber are also included; (d) "horizontal displacement" - includes structures with a misalignment of bearings and superstructure or beams jammed against the abutments; also included in this category of damage are bridges where the superstructure extended beyond the abutment, where beams required cutting, or where there was horizontal movement of the floor system; (e) "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 shearing of anchor bolts, the opening, closing or damage of deck joints and cases where the cutting of relief joints were required; (f) "damage to railings, curbs, sidewalks, or parapets" - includes the cracking, deformation, or misalignment of railing, curb, sidewalks, or parapets; jammed curbs and crushed concrete and open, closed or damaged portions of those bridge elements also fall into this category; (g) "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 under this category are deformed neoprene bearing pads, sheared anchor bolts in the bearing shoes, damage to expansion dams or devices, and the cracking of concrete at the bearings; (h) "poor riding quality" - refers to conditions where noticeable driver discomfort was reported; (i) "not given/corrected during construction" - describes those cases where any mention of structural effects was omitted or where foundation movement was corrected prior to construction of the superstructure; and (j) "none" - is applied to structures where no noticeable effects due to movements were reported.

The information regarding structural damage, as supplied by the various highway agencies, was assumed to be complete and accurate for the purposes of analysis. Only those effects which were specifically reported were included in the analysis. No additional effects were inferred.

The subjectivity of the term "tolerable" may be one reason for the lack of generally accepted tolerable movement criteria. Movements that are considered to be tolerable by one engineer may be considered to be intolerable by another. In an attempt to eliminate some of this subjectivity, Transportation Research Board Committee A2K03 defined intolerable movement as follows (44,75): "Movement is <u>not</u> tolerable if damage requires costly maintenance and/or repairs <u>and</u> a more expensive construction to avoid this would have been preferable." For the sake of consistency, this definition was also adopted for the study reported herein.

As noted earlier, the 1967 questionnaire did not address the question of tolerance to movements. Therefore, this information was absent from the

1967 data as well as from some of the 1975 data, where it was omitted by the respondent. Although the supplementary information supplied by the various highway agencies did provide some of the missing data, there were a significant number of bridges for which specific data on tolerance to movements were missing. Therefore, in order to better delineate the general trends with regard to tolerance, the sample size was increased by designating the movements as tolerable or intolerable based upon the description of the structural damage, the maintenance required and other comments provided by the respondents. These designations were made only if the descriptions of the damage and the resulting maintenance were such that the movements were obviously tolerable or intolerable in accordance with the above definition. Overall, there were 171 of the 204 structures available for analysis where data on tolerance to foundation movements were available or could reasonably be assumed.

It should be recognized that the data on foundation movements presented herein are biased in the sense that they represent the observed behavior of only those bridge foundations that have experienced some type of movement. To date, no effort has been made in this study to compile data that would permit the comparison of the relative performance of different foundation systems (i.e. piles vs. spread footings). Consequently, no inferences of this type should be drawn from the data presented without proper recognition of their limitations. Furthermore, it should be recognized that, although the total number of bridges that reportedly experienced foundation movements is substantial, only a relatively small number of bridges were reported to have moved in each of the States that contributed data. Thus, the results of this limited study of bridge movements and their effects should not be construed as implying that the occurrence of bridge foundation movements is widespread and that it constitutes a major problem.

3.1.3 Methods of Data Analysis

The objective of the analysis of the collected field data was to delineate general trends with regard to the nature of bridge foundation movements, their effects, and the ability of the bridge to tolerate these movements. In effect, three separate analyses were conducted, each with a somewhat different methodology.

The first analysis involved the investigation of the influence of substructure variables on bridge abutment and pier movements. For the abutments, the variables considered were (a) general soil conditions, (b) type of abutment (full height, perched or spill-through), (c) type of foundation (spread footings or piles), and (d) height of approach embankment. A general summary of the substructure data that were incorporated into this analysis is presented in Table 2. In addition to considering the effect of each of these variables on abutment movements, various combinations of variables were considered in an effort to determine combinations that may or may not result in foundation movement. Additional variables considered for the piers were (a) the span type (simply supported or continuous) and (b) the abutment-embankment-pier geometry. A general summary of the superstructure data, including type of span, that have been incorporated into this and other analyses, is presented in Table 3. Again,

Table 2.--General Summary of Substructure Data

Substructure Variables (1)	Number of Bridges (2)
General Soil Conditions	
Fine Grained Soil	77
Granular Soils	45
Fine Grained Soils Over Granular Soils	12
Granular Soils Over Fine Grained Soils	24
Interlayered/Intermixed Soils	24
Bedrock	12
Permafrost Soils	3
Soil Conditions not given	7
Foundation Type	
Spread Footings	73
Piles	66
Abutments on Spread Footings/Piers on Piles	17
Abutments on Piles/Piers on Spread Footings	22
Abutments and Piers on Both Spread Footings and Piles	15
Miscellaneous Combinations of Spread Footings, Caissons, etc.	3
Foundation type not given	8
Abutment Type	
Full Height	33
Perched	131
Spill-through	14
Full Height and Perched	2
Perched and Spill-through	3
Abutment type not given or unknown	21
Height of Approach Embankments	
Cut	3
0 feet to 9 feet	8
10 feet to 19 feet	38
20 feet to 29 feet	77
30 feet to 39 feet	41
40 feet to 49 feet	9
50 feet to over 100 feet	15
Approach height not given	13

Note: 1 foot = 0.3048 meters

Table 3.--Summary of the Superstructure Data

Superstructure Variables (1)	Number of Bridges (2)
Type of Span	
Simple	84
Continuous	75
Simple and Continuous	9
Rigid Frame	7
Cantilever	7
Miscellaneous or not given	22
Type of Structural Material	
Steel	101
Concrete	65
Steel and Concrete	4
Material type not given	34
Number of Spans	
One	23
Two	21
Three	73
Four	33
Five	14
More than five	37
Number of spans not given	3

Table 4.--General Summary of Data on Structural Damage

Type of Structural Damage (1)	Number of Bridges (2)
Damage to Abutments	56
Damage to Piers	17
Vertical Displacement	13
Horizontal Displacement	44
Distress in the Superstructure	98
Damage to Rails, Curbs, Sidewalks, Parapets	30
Damage to Bearings	33
Poor Riding Quality	12
Not Given/Corrected During Construction	10
None	25

the influence of each of the selected variables was considered separately and in selected combinations. A valuable by-product of this analysis was the identification of the most common causes of foundation movements for the bridges studied. In addition, it was possible to explore, in a limited way, the influence of construction sequence and precompression (1,41) 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) the span type, (d) the type of structural material (steel or concrete), (e) the number of spans, and (f) abutment type. 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 4. It should be noted that many of these structures experienced multiple damaging effects. The implications of this fact will be brought out later in this report.

The third analysis involved the investigation of the tolerance of the various bridge structures to movements. The variables considered in this analysis were (a) type of structural damage, (b) type of movement, (c) magnitude of movements (maximum differential vertical movement between successive units of the substructure, maximum longitudinal angular distortion, and maximum horizontal movement), (d) the span type, (e) the type of structural material, (f) the number of spans, and (g) type of abutment.

Initially, the three analyses described above were conducted in great detail, using a manual data reduction and processing system (44). However, these preliminary analyses were begun before the data collection process was entirely complete, and therefore considered data from only 180 bridges. The final analyses employed a computerized data storage and retrieval system (20) and used data from all 204 bridges. These analyses resulted in the generation of a very large amount of information on the influence of substructure variables on bridge foundation movements, the influence of these movements on bridge structures, and the tolerance of bridges to these movements. For the sake of brevity, only a limited portion of the results can be presented here. The details of the data storage and retrieval system and the preliminary analyses are presented in Volume II and Volume III of this report, respectively.

3.2 Influence of Substructure Variables on Foundation Movement

3.2.1 Abutment Movements

There were a total of 362 abutments which had sufficient data to be included in the analysis. Over three-quarters of these experienced some type of movement. A general summary of the movement data for these 273 abutments is presented in Table 5. These data show that a great majority of

Table 5.--General Summary of Abutment Movements

Movement Type (1)	Frequency: Number of Abutments (2)	Frequency: Percent Moved (3)	Range in Inches (4)	Average in Inches (5)	Standard Deviation in Inches (6)
All Types	273	100.0			
Vertical	221[a]	81.0	0.1-50.4	4.4	6.4
Horizontal	114	41.8	0.1-14.4	2.7	2.3
Vertical &	61	22.3	0.1-50.4	7.7	10.2
Horizontal			0.1-14.4	2.7	2.6

[a]Two abutments, which raised vertically, are not included in total, range, average or standard deviation. Note: 1 inch = 25.4 mm.

Table 6.--Ranges of Magnitudes of Abutment Movements in General

Movement Interval in Inches (1)	Type of Movement: Vertical: Number of Abutments (2)	Vertical: Percent of Total (3)	Horizontal: Number of Abutments (4)	Horizontal: Percent of Total (5)
0 - 1.9	100	45.3	45	39.4
2.0- 3.9	48	21.7	47	41.2
4.0- 5.9	24	10.9	13	11.4
6.0- 7.9	14	6.3	2	1.8
8.0- 9.9	8	3.6	6	5.3
10.0-14.9	11	5.0	1	0.9
15.0-19.9	9	4.1	0	0.0
20.0-60.0	7	3.1	0	0.0
Total	221[a]	100.0	114	100.0

[a]Two abutments, which raised vertically, are not included.
Note: 1 inch = 25.4 mm.

the abutments that moved experienced vertical movement, less than half of them moved horizontally, and a substantial number moved both vertically and horizontally. This is further illustrated in Table 6, which shows the frequency of occurrence of the various ranges of vertical and horizontal movements. **The magnitudes of the vertical movements tended to be substantially greater than the horizontal movements.** This can be explained, in part, by the fact that in many instances the abutments moved inward until they became jammed against the beams or girders, which acted as struts, thus preventing further horizontal movements. In fact, all but 5 of the 114 abutments that experienced horizontal movements moved inward. Those 5 abutments that moved outward were perched abutments founded on piles driven through approach fills placed over compressible foundation soils. This type of movement has been described by Stermac (69). Table 5 also shows that **abutment movements tended to be larger and more variable for those abutments that experienced both vertical and horizontal movements.**

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. The summary of abutment movements in terms of abutment type, given in Tables 7 and 8, shows that **perched and spill-through abutments tended to undergo larger and a wider range of movements than did the full height abutments.** This was especially true with respect to vertical movements. This would suggest that **in the future greater attention needs to be directed to the design and construction of the foundation systems for perched and spill-through abutments.**

In this connection, it was also found that the construction sequence and/or the use of precompression (1,41) exerted a significant influence on the movements of perched abutments founded on spread footings on fill. This is illustrated in Table 9, which shows that **the frequency, range and average magnitude of abutment movements were substantially lower, when a preload and/or waiting period was employed prior to construction of the abutments, than when the abutments were constructed immediately following completion of the embankments.** For the 37 perched abutments where a preload and/or waiting was used, the abutment construction was delayed for one month to six months following completion of the approach embankments. Usually these delays permitted most of the embankment and foundation movement to take place before the beginning of abutment construction.

In terms of foundation type, **abutments founded on spread footings had a higher incidence of movement than abutments founded on piles,** with 83.0 percent of 188 abutments on spread footings moving as compared to 66.9 percent of 172 abutments founded on piles. However, the summary of abutment movements in terms of foundation types, presented in Table 10, shows that **abutments founded on piles actually experienced a larger range and average vertical movement than did those founded on spread footings.** This situation also existed with respect to horizontal movements. These same general trends were observed in most cases when the data were further broken down in terms of abutment type, as shown in Table 11. 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 acceptable limits, particularly for the case of perched abutments on

Table 7.--Summary of Movements in Terms of Abutment Types

Abutment Type (1)	Movement Type (2)	Frequency: Number of Abutments (3)	Frequency: Percent Moved (4)	Range in Inches (5)	Average in Inches (6)	Standard Deviation in Inches (7)
Full Height	All Types	60	100.0			
	Vertical[a]	52	86.7	0.3-17.0	3.3	3.2
	Horizontal	28	46.7	0.1-8.0	2.2	1.9
	Vertical &	20	33.3	0.3-17.0	3.6	4.3
	Horizontal			0.1-8.0	2.2	2.0
Perched	All Types	195	100.0			
	Vertical	153	78.5	0.1-50.4	4.5	7.1
	Horizontal	73	37.4	0.3-14.4	3.1	2.6
	Vertical &	31	15.9	0.1-50.4	10.3	12.4
	Horizontal			0.3-14.4	3.4	3.0
Spill-Through	All Types	21	100.0			
	Vertical	16	76.2	1.2-24.0	8.2	8.1
	Horizontal	13	61.9	0.5-8.8	2.4	2.2
	Vertical &	8	38.1	1.2-24.0	7.8	10.0
	Horizontal			0.5-3.0	1.4	1.0

[a]Two full height abutments, which raised 3 inches, are not included.
Note: 1 inch = 25.4 mm.

Table 8.--Ranges of Magnitudes of Abutment Movements in Terms of Abutment Types

Movement Interval in Inches (1)	Type of Abutment With Given Types of Movement											
	Full Height				Perched				Spill-Through			
	Vertical		Horizontal		Vertical		Horizontal		Vertical		Horizontal	
	Number of Abutments (2)	Percent of Total (3)	Number of Abutments (4)	Percent of Total (5)	Number of Abutments (6)	Percent of Total (7)	Number of Abutments (8)	Percent of Total (9)	Number of Abutments (10)	Percent of Total (11)	Number of Abutments (12)	Percent of Total (13)
0 - 1.9	19	36.5	15	53.5	80	52.3	24	32.9	1	6.2	6	46.2
2.0- 3.9	18	34.6	10	35.7	23	15.0	31	42.4	7	43.8	6	46.2
4.0- 5.9	7	13.5	1	3.6	17	11.1	12	16.4	0	0.0	0	0.0
6.0- 7.9	4	7.7	1	3.6	8	5.2	1	1.4	2	12.5	0	0.0
8.0- 9.9	1	1.9	1	3.6	5	3.3	4	5.5	2	12.5	1	7.6
10.0-14.9	2	3.9	0	0.0	9	5.9	1	1.4	0	0.0	0	0.0
15.0-19.9	1	1.9	0	0.0	6	3.9	0	0.0	2	12.5	0	0.0
20.0-60.0	0	0.0	0	0.0	5	3.3	0	0.0	2	12.5	0	0.0
Total	52[a]	100.0	28	100.0	153	100.0	73	100.0	16	100.0	13	100.0

[a]Two full height abutments, which raised vertically, are not included.
Note: 1 inch = 25.4 mm

Table 9.--Summary of Movements of Perched Abutments on Spread Footings on Fill in Terms of Construction Sequence

Construction Sequence (1)	Movement Type (2)	Frequency		Range in Inches (5)	Average in Inches (6)	Standard Deviation in Inches (7)
		Number of Abutments (3)	Percent Moved (4)			
Preload and/or Waiting Period	All Types	37	100.0			
	Vertical	37	100.0	0.2- 5.0	1.7	1.5
	Horizontal	2	5.4	0.3- 0.3	0.3	0.0
	Vertical &	2	5.4	4.0- 5.0	4.5	0.7
	Horizontal			0.3- 0.3	0.3	0.0
No Preload or Waiting Period	All Types	61	100.0			
	Vertical	58	95.1	0.1-35.0	6.8	7.9
	Horizontal	13	21.3	0.3- 5.0	3.5	1.2
	Vertical &	10	16.4	0.1-35.0	18.2	10.4
	horizontal			0.3- 5.0	3.7	1.3

Note: 1 inch = 25.4 mm

Table 10.--Summary of Abutment Movements in Terms of Foundation Type

Foundation Type (1)	Movement Type (2)	Frequency: Number of Abutments (3)	Frequency: Percent Moved (4)	Range in Inches (5)	Average in Inches (6)	Standard Deviation in Inches (7)
Spread Footings	All Types	162	100.0			
	Vertical	150	92.6	0.1-35.0	4.2	5.6
	Horizontal	38	23.5	0.1-8.8	2.5	2.1
	Vertical &	26	16.1	0.1-35.0	9.3	9.8
	Horizontal			0.1-8.0	2.4	2.0
Piles	All Types	114	100.0			
	Vertical	71	62.3	0.1-50.4	5.2	8.1
	Horizontal	76	66.7	0.5-14.4	2.9	2.4
	Vertical &	33	29.0	0.3-50.4	6.4	10.7
	Horizontal			0.5-14.4	3.1	2.9

Note: 1 inch = 25.4 mm.

Table 11.--Summary of Abutment Movements in Terms of Foundation Type and Abutment Type

Abutment Type (1)	Movement Type (2)	Spread Footing Foundations: Frequency: Number of Abutments (3)	Percent Moved (4)	Range in Inches (5)	Average in Inches (6)	Standard Deviation in Inches (7)	Pile Foundations: Frequency: Number of Abutments (8)	Percent Moved (9)	Range in Inches (10)	Average in Inches (11)	Standard Deviation in Inches (12)
Full Height	All Types	45	100.0				15	100.0			
	Vertical	39	86.7	0.4 - 11.4	3.4	2.6	13	86.7	0.3 - 17.0	3.0	4.6
	Horizontal	18	40.0	0.1 - 8.0	1.8	2.1	10	66.7	1.1 - 5.5	2.8	1.3
	Vertical &	12	26.7	0.5 - 11.4	3.8	3.2	8	53.3	0.3 - 17.0	3.3	5.8
	Horizontal			0.1 - 8.0	1.6	2.1			1.1 - 5.5	3.1	1.2
Perched	All Types	111	100.0				84	100.0			
	Vertical	107	96.4	0.1 - 35.0	4.4	6.5	46	54.8	0.1 - 50.4	4.9	8.4
	Horizontal	18	16.2	0.3 - 5.0	2.7	1.7	55	65.5	0.5 - 14.4	3.1	2.7
	Vertical &	14	12.6	0.1 - 35.0	14.0	11.1	17	20.2	0.8 - 50.4	7.2	12.9
	Horizontal			0.3 - 5.0	2.8	1.8			0.9 - 14.4	3.9	3.7
Spill-Through	All Types	6	100.0				15	100.0			
	Vertical	4	66.7	3.6 - 8.0	6.4	2.1	12	80.0	1.2 - 24.0	8.8	9.3
	Horizontal	2	33.3	3.0 - 8.8	5.9	4.1	11	73.3	0.5 - 3.0	1.7	1.1
	Vertical &	0	0.0				8	53.3	1.2 - 24.0	7.8	10.0
	Horizontal								0.5 - 3.0	1.4	1.0

Note: 1 inch = 25.4 mm.

fills. In fact, there is an existing body of evidence that, under some circumstances, bridges founded on piles or other deep foundations can move, sometimes substantially (7,11,29,43,50,61,69). In the light of this information, it is suggested that **in the future the design and construction of pile supported abutments should be pursued with great care and attention to detail, in order to assure that the performance of these substructure units meets expectations.**

With respect to foundation soil type, there was a high incidence of vertical movement for abutments founded on spread footings on soil profiles with substantial quantities of fine grained soils. With reference to major soil profiles, horizontal movements occurred most often for pile foundations in fine grained soils overlying granular soils. The largest vertical movements tended to occur for abutments on spread footings in fine grained soils and on pile foundations in granular soils overlying fine grained soils. The largest horizontal movements occurred for pile foundations and spread footings in fine grained soil.

Although some general trends were evident, approach embankment heights did not correlate particularly well with the frequency and magnitude of abutment movements. This tends to agree with the findings reported by Grover (34) for Ohio bridges. As might be expected, there was a general trend toward increasing frequency and magnitude of vertical movements with increase in height of approach embankments, as shown in Table 12. However, additional analyses with regard to embankment height, in terms of abutment type, foundation type and soil conditions, did not show a great deal of evidence of meaningful trends.

3.2.2 Pier Movements

The results of the analysis of pier movements showed that, in general, **piers moved less often than abutments.** Only 28.8 percent of the 706 piers considered in the analysis showed any movement. The general summaries of pier movements given in Table 13 and 14 shows that **vertical movements tended to be substantially less than for abutments.** Unlike the abutment movements, average **horizontal pier movements tended to be larger than the vertical movements.**

Although many more piers were founded on piles (456) than on spread footings (242), over half of the piers that moved were founded on spread footings. Movements were reported for 104, or 43.0 percent, of the piers founded on spread footings, and 91, or only 20.0 percent, of those founded on piles. When compared with corresponding data for abutments, these data suggest that the rate of success in founding piers on piles is substantially greater than that of founding abutments on piles, particularly for perched and spill-through abutments. Tables 15 and 16 which summarize the pier movements in terms of foundation type, show that **the average magnitude of vertical movement was greater for pile foundations than for spreading footings.** However, **the vertical movements for the piers on spread footings had a wider range than for those founded on piles.**

Very few trends were evident 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

Table 12.--Summary of Abutment Movements in Terms of Height of Approach Embankment

Embankment Height Interval in Feet (1)	Movement Type (2)	Frequency Number of Abutments (3)	Frequency Percent Moved (4)	Range in Inches (5)	Average in Inches (6)	Standard Deviation in Inches (7)
0 - 9.9	All Types	8	100.0			
	Vertical	7	87.5	0.3 - 8.8	2.5	2.8
	Horizontal	2	25.0	1.8 - 3.6	2.7	1.3
	Vertical &	1	12.5	2.4	2.4	0.0
	Horizontal			1.8	1.8	0.0
10 - 19.9	All Types	57	100.0			
	Vertical	45	79.0	0.3 - 18.0	4.4	4.7
	Horizontal	27	47.4	0.5 - 9.1	3.5	2.7
	Vertical &	15	26.3	0.3 - 18.0	5.6	7.0
	Horizontal			1.0 - 9.0	3.4	2.5
20 - 29.9	All Types	125	100.0			
	Vertical	99	79.2	0.1 - 50.4	4.3	7.0
	Horizontal	44	35.2	0.1 - 14.4	2.7	2.9
	Vertical &	18	14.4	0.5 - 50.4	8.5	12.9
	Horizontal			0.1 - 14.4	2.9	4.3
30 - 39.9	All Types	50	100.0			
	Vertical	40	80.0	0.1 - 24.0	4.3	6.2
	Horizontal	22	44.0	0.3 - 4.0	1.6	1.1
	Vertical &	12	24.0	0.1 - 24.0	7.7	9.9
	Horizontal			0.3 - 4.0	1.2	1.0
40 - 49.9	All Types	14	100.0			
	Vertical	11	78.6	1.0 - 18.0	6.7	6.3
	Horizontal	9	64.3	0.3 - 8.8	3.3	3.0
	Vertical &	6	42.9	1.0 - 18.0	8.3	7.6
	Horizontal			0.3 - 5.5	2.3	2.2
50 - 100+	All Types	19	100.0			
	Vertical	19	100.0	0.3 - 35.0	10.0	9.7
	Horizontal	8	42.1	3.5 - 5.0	4.1	0.6
	Vertical &	8	42.1	2.1 - 35.0	14.7	11.1
	Horizontal			3.5 - 5.0	4.1	0.6

Note: 1 foot = 304.8 mm, and 1 inch = 25.4 mm.

Table 13.--General Summary of Pier Movements

Movement Type (1)	Frequency: Number of Piers (2)	Frequency: Percent Moved (3)	Range in Inches (4)	Average in Inches (5)	Standard Deviation in Inches (6)
All Types	203	100.0			
Vertical	171[a]	84.2	0.1-42.0	2.8	4.2
Horizontal	47	23.2	0.1-20.0	3.3	4.3
Vertical &	15	7.4	0.3-13.7	3.4	3.7
Horizontal			0.6-20.0	3.1	5.2

[a]The number of piers with movement includes 7 piers which raised vertically. These piers are not included in the total with vertical movement. Note: 1 inch = 25.4 mm.

Table 14.--Ranges of Magnitudes of Pier Movements in General

Movement Interval in Inches (1)	Type of Movement: Vertical: Number of Abutments (2)	Vertical: Percent of Total (3)	Horizontal: Number of Abutments (4)	Horizontal: Percent of Total (5)
0.0 - 1.9	99	60.7	31	67.4
2.0 - 3.9	22	13.5	3	6.5
4.0 - 5.9	14	8.6	4	8.7
6.0 - 7.9	22	13.5	2	4.3
8.0 - 9.9	1	0.6	2	4.3
10.0 - 14.9	3	1.8	2	4.3
15.0 - 19.9	1	0.6	1	2.2
20.0 - 60.0	1	0.6	1	2.2
Total	163[a]	100.0	46	100.0

[a]Seven piers, which raised vertically, are not included.
Note: 1 inch = 25.4 mm.

Table 15.--Summary of Pier Movements in Terms of Foundation Type

Foundation Type (1)	Movement Type (2)	Frequency: Number of Piers (3)	Frequency: Percent Moved (4)	Range in Inches (5)	Average in Inches (6)	Standard Deviation in Inches (7)
Spread Footings	All Types	104	100.0			
	Vertical	94	90.4	0.1-42.0	2.1	4.9
	Horizontal	17	16.4	0.5-20.0	3.3	5.0
	Vertical &	7	6.7	0.8-9.0	3.8	2.6
	Horizontal			0.6-20.0	4.9	7.3
Piles	All Types	90	100.0			
	Vertical[a]	69	76.7	0.1-14.0	3.8	3.0
	Horizontal	29	32.2	0.1-16.0	3.1	3.9
	Vertical &	8	8.9	0.3-13.7	3.0	4.6
	Horizontal			0.6-4.0	1.6	1.2

[a]Number of piers on piles with movement includes 7 piers which raised vertically. These are not included for vertical movements. Note: 1 inch = 25.4 mm.

Table 16.—Ranges of Magnitudes of Pier Movements in Terms of Foundation Type

Movement Interval in Inches (1)	Spread Footings, Vertical: Number of Piers (2)	Spread Footings, Vertical: Percent of Total (3)	Spread Footings, Horizontal: Number of Piers (4)	Spread Footings, Horizontal: Percent of Total (5)	Piles, Vertical: Number of Piers (6)	Piles, Vertical: Percent of Total (7)	Piles, Horizontal: Number of Piers (8)	Piles, Horizontal: Percent of Total (9)
0.0 - 1.9	72	76.6	12	70.6	27	39.1	19	65.5
2.0 - 3.9	12	12.8	0	0.0	10	14.5	3	10.3
4.0 - 5.9	5	5.3	2	11.8	9	13.0	2	6.9
6.0 - 7.9	1	1.1	1	5.9	21	30.4	1	3.5
8.0 - 9.9	1	1.1	1	5.9	0	0.0	1	3.5
10.0 - 14.9	1	1.1	0	0.0	2	2.9	2	6.9
15.0 - 19.9	1	1.1	0	0.0	0	0.0	1	3.5
20.0 - 60.0	1	1.1	1	5.9	0	0.0	0	0.0
Total	94	100.0	17	100.0	69[a]	100.0	29	100.0

Type of Foundation With Given Types of Movement

[a]Seven piers, which raised vertically, are not included.
Note: 1 foot = 304.8 mm, and 1 inch = 25.4 mm

Table 17.—Summary of Pier Movements in Terms of Pier Location

Pier Location (1)	Movement Type (2)	Frequency: Number of Piers (3)	Frequency: Percent Moved (4)	Range in Inches (5)	Average in Inches (6)	Standard Deviation in Inches (7)
In or Near Embankment	All Types	140	100.0			
	Vertical	117	83.6	0.1 - 42.0	2.2	4.7
	Horizontal	40	28.6	0.1 - 20.0	3.2	4.0
	Vertical &	17	12.1	0.3 - 13.7	3.1	3.5
	Horizontal			0.4 - 20.0	2.9	4.9
Away From Embankment	All Types	57	100.0			
	Vertical	51	89.5	0.1 - 6.8	4.0	2.5
	Horizontal	6	10.5	0.1 - 4.0	1.1	1.4
	Vertical & Horizontal	0	0.0	0.0	0.0	0.0

Note: 1 inch = 25.4 mm

with fine grained 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, as shown in Table 17. These data show that, contrary to what might be expected, the magnitudes of vertical movements tended to be slightly larger for piers located away from the embankments, with an average movement of 4.0 inches (101.6 mm), as compared to 2.2 inches (55.9 mm) for piers located in or near the embankment. **The magnitudes of horizontal movements, however, were significantly larger for piers located in or near the embankment** with an average of 3.2 inches (81.3 mm) as compared to only 1.1 inches (27.9 mm) for the piers located away from the embankment. **This would suggest that, in designing bridge piers in or near the toe of embankments, more consideration needs to be given to the increased level of horizontal stresses that exist in these areas.**

3.2.3 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 for the majority of the bridges studied. **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 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. In fact, a substantial number of these types of abutments that moved along with their underlying embankments were founded on piling, as shown in Table 11.

Although, as noted earlier, a substantial number of pile supported foundations were reported to have experienced movements, thus suggesting unsatisfactory performance of the piles in resisting applied loads, in many instances it was difficult to pinpoint the reasons for this poor performance. This is because many of the case histories studied lacked sufficient detail with respect to the design and construction of the pile foundations to permit a reliable evaluation to be made. An effort is being made to obtain additional information for these bridges in order to determine what factors might have contributed to the inability of the pile foundations to resist the applied loads without movements. Of course, in those cases where pile supported perched or spill-through abutments moved as a result of embankment sliding, it is obvious that the pile foundations were not designed to resist the loads imposed by the embankment movements.

In fact, it would be unreasonable to expect a pile foundation to resist the loads imposed by an unstable embankment unless it was specifically designed to do so.

In those instances of horizontal abutment movement, either by sliding or rotation or both, where slope stability was not a factor, it was apparent that the abutment foundation could not adequately resist the applied lateral earth pressures. However, in most of these cases it was not readily apparent whether the lateral earth pressures had been underestimated or the foundation design did not provide adequate resistance against sliding and overturning. Further study will be devoted to these case histories in an effort to resolve this matter.

3.3 Influence of Foundation Movements on Bridges

As indicated in Table 4, the most frequently occurring types of structural damage were distress in the superstructure, damage to abutments, "horizontal displacement", and damage to bearings. Those structures with only abutment movements had a high frequency of distress in the superstructure and a somewhat lower incidence of "horizontal displacement" and abutment damage. Distress in the superstructure also occurred very frequently for bridges with only pier movements and for bridges with both abutment and pier movements. Table 18, which relates structural damage to type of foundation movement, shows that **the most types of structural damage appear to occur for those bridges with both vertical and horizontal movements occurring simultaneously.** "Horizontal displacement", abutment damage and distress in the superstructure occurred relatively frequently for bridges with both vertical and horizontal movements. In contrast, structures for which only vertical movement was reported had the lowest frequency of damaging structural effects, with 23 structures having no damage at all.

This same general trend was evident in terms of magnitudes of movements, in that even moderate differential vertical movements tended to produce a relatively low incidence of structural damage. Of the 69 bridges with maximum differential vertical settlements of less than 4 inches (101.5 mm), 23 experienced no damage whatsoever. The majority of the remaining 46 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. However, the abutment damage was strangely absent from bridges with larger differential vertical movements. For differential vertical movements in excess of 4 inches (101.5 mm), distress in the superstructure tended to be the predominate structural effect. "Vertical displacement" and poor riding quality were fairly common for differential vertical movements of 8 inches (203.2 mm) and greater. However, it should be pointed out that there were only 12 bridges, out of the 204 considered, for which poor riding quality was reported. This matter will be given further consideration later 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, even for relatively small horizontal movements, suggesting that **horizontal movements are much more**

Table 18.—Types of Structural Damage Associated With Types of Movements

Structural Damage (1)	Type of Movement: Vertical — Number of Bridges (2)	Vertical — Percent of Category[a] (3)	Horizontal — Number of Bridges (4)	Horizontal — Percent of Category (5)	Vertical and Horizontal — Number of Bridges (6)	Vertical and Horizontal — Percent of Category (7)
Damage to Abutments	23	23.5	7	18.0	22	39.3
Damage to Piers	3	3.1	5	12.8	8	14.3
Vertical Displacement	5	5.1	0	0.0	6	10.7
Horizontal Displacement	2	2.0	16	41.0	23	41.1
Distress in Superstructure	43	43.9	24	61.5	28	50.0
Damage to Rails, Curbs, Sidewalks, Parapets	16	16.3	3	7.7	10	17.9
Damage to Bearings	1	1.0	18	46.2	12	21.4
Poor Riding Quality	8	8.2	0	0.0	4	7.1
Not Given or Corrected During Construction	6	6.1	1	2.6	1	1.8
None	23	23.5	0	0.0	2	3.6
Total Bridges in Category	98		39		56	

[a]Percent of bridges in this category with indicated structural damage.

critical than vertical movements in causing structural damage. For those structures with horizontal movements alone, movements of from 1.0 to 2.0 inches (25.4 to 50.8 mm) caused distress in the superstructure very commonly, occurring in more than two-thirds of the cases. The bearings were also affected in more than a third of these structures. Abutment damage and "horizontal displacement" appeared to begin occurring with greater frequency for horizontal movements of 4 inches (101.6 mm) and greater.

It was more difficult to correlate structural damage with magnitudes of substructure movements for those cases where vertical and horizontal movements occurred simultaneously, because of the possible interaction of the two types of movements. However, some observations can be made. Abutment damage was more frequent at movements less than 4.0 inches (101.6 mm) for bridges with both vertical and horizontal movements than for the structures with movement in only one direction. About one third of the 35 bridges having movement in both directions, with the magnitude of the vertical component less than 4.0 inches (101.6 mm), had abutment damage. Similarly, more than one third of 46 bridges, with the magnitude of the horizontal component of movement less than 4.0 inches (101.6 mm), had abutment damage reported. In addition, "horizontal displacement" and damage to bearings were most frequent for bridges having both vertical and horizontal movements. However, 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. Thus, as suggested earlier, horizontal movements appear to be much more critical than differential vertical settlement in causing most types of structural distress. This tends to confirm the findings of Walkinshaw (75) and Bozozuk (12).

In terms of span type (simply supported or continuous), the data presented in Table 19 show that distress in the superstructure was the most common structural effect reported for both continuous and simply supported bridges. However, this type of distress was reported more frequently for the 72 continuous structures, occurring in 56.9 percent of the cases, than for the 83 simply supported bridges, where it occurred in just 38.6 percent of the cases. Table 19 also shows that abutment damage was the second most frequently reported effect for the simply supported structures, occurring in 33.7 percent of those bridges, while this type of damage was reported in only 16.7 percent of the continuous structures. The analysis of structural distress, in terms of the foundation elements undergoing movement, indicated some definite differences between the two types of structures. For the 42 simply supported structures with abutment movement only, abutment damage and distress in the superstructure, occurring in 31.0 percent, were the most common types of effects. For the 33 continuous structures with abutment movement only, distress in the superstructure was much more frequent, occurring in 69.7 percent of the bridges, while "horizontal displacement" occurred in 45.5 percent. Damage to bearings was also reported quite frequently, occurring in 36.4 percent of the cases. However, for structures with both abutment and pier movement, those 30 with simple spans had a fairly high occurrence of horizontal displacement and distress in the superstructure with each occurring in about 40 percent of the bridges. For the 31 continuous structures having both abutment and pier movement, the most prominent structural effect was distress in the

Table 19.--Types of Structural Damage Associated With Span Type

Structural Damage (1)	Type of Span: Simple — Number of Bridges (2)	Simple — Percent of Category[a] (3)	Continuous — Number of Bridges (4)	Continuous — Percent of Category (5)
Damage to Abutments	28	33.7	12	16.7
Damage to Piers	6	7.2	9	12.5
Vertical Displacement	4	4.8	4	5.6
Horizontal Displacement	18	21.7	16	22.2
Distress in Superstructure	32	38.6	41	56.9
Damage to Rails, Curbs, Sidewalks, Parapets	8	9.6	16	22.2
Damage to Bearings	9	10.8	16	22.2
Poor Riding Quality	5	6.0	5	6.9
Not Given or Corrected During Construction	2	2.4	5	6.9
None	14	16.9	10	13.9
Total Bridges in Category	83		72	

[a]Percent of bridges in this category with indicated structural damage.

Table 20.--Types of Structural Damage Associated With Material Type

Structural Damage (1)	Type of Material: Steel — Number of Bridges (2)	Steel — Percent of Category[a] (3)	Concrete — Number of Bridges (4)	Concrete — Percent of Category (5)
Damage to Abutments	39	39.8	11	16.9
Damage to Piers	7	7.1	6	9.2
Vertical Displacement	5	5.1	4	6.2
Horizontal Displacement	25	25.5	7	10.8
Distress in Superstructure	42	42.9	39	60.0
Damage to Rails, Curbs, Sidewalks, Parapets	10	10.2	17	26.1
Damage to Bearings	21	21.4	5	7.7
Poor Riding Quality	4	4.1	4	6.2
Not Given or Corrected During Construction	5	5.1	3	4.6
None	13	13.3	7	10.8
Total Bridges in Category	98		65	

[a]Percent of bridges in this category with indicated structural damage.

superstructure, reported in 54.8 percent of the bridges. Sample groups for structures with only pier movements were too small to make valid comparisons. The frequency of bridges with no structural damage was greater for those cases where both abutments and piers moved, regardless of span type, presumably because of the lower level of differential movement in those cases. For both types of spans, the most frequent and most serious types of structural distress appeared to be related to horizontal movements.

The data on the frequency of occurrence of the various types of bridge damage in terms of structural material, presented in Table 20, show that distress in the superstructure was reported much more frequently for concrete structures than for steel structures. However, the steel structures had a higher frequency of abutment damage, "horizontal displacement" and damage to bearings. In terms of vertical and horizontal movements, Table 21 shows that the steel bridges, with differential vertical movement alone, had a lower incidence and severity of structural damage than did the concrete bridges. Of the 38 steel bridges which experienced only vertical movements, only 26.3 percent experienced distress in the superstructure, while this type of damage was reported in 60.9 percent of the 46 concrete bridges with the same type of movement. This situation was reversed for those bridges which experienced horizontal movements only and vertical and horizontal movements occuring simultaneously. Over half of the steel bridges, with horizontal movement only, experienced distress in the superstructure and damage to bearings. 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 much 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, because of sample sizes. However, the data do tend to indicate that multispan structures had a higher frequency of more severe structural effects than did single span bridges.

The data on the frequency of occurrence of each of the various types of structural distress in terms of abutment type, presented in Table 22, show that structures on full height abutments tended to have the highest occurrence of abutment damage, but a relatively low occurrence of distress in the superstructure, damage to bearings and "vertical and horizontal displacement". Although those bridges on 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. This is somewhat of a paradox, since, as reported earlier, perched abutments tended to undergo a larger and a wider range of movements than did the full height abutments. However, a detailed examination of the data revealed that it was primarily differential vertical abutment movements of less than 4 inches (101.6 mm) that caused no damage to these bridges with perched abutments. The most damaging effects were produced primarily by horizontal movements between one inch (25.4 mm) and 4 inches (101.6 mm) in magnitude, and these effects were particularly serious when these horizontal movements were accompanied by larger differential vertical movements, i.e. differential settlements in excess of 4 inches (101.6 mm).

Table 21.--Types of Structural Damage Associated With Types of Movements for Different Types of Construction Materials.

Construction Material (1)	Structural Damage (2)	Type of Movement: Vertical — Number of Bridges (3)	Vertical — Percent of Category[a] (4)	Horizontal — Number of Bridges (5)	Horizontal — Percent of Category (6)	Vertical and Horizontal — Number of Bridges (7)	Vertical and Horizontal — Percent of Category (8)
Steel	Damage to Abutments	14	36.8	6	33.3	17	47.2
	Damage to Piers	0	0.0	4	22.2	4	11.1
	Vertical Displacement	2	5.3	0	0.0	2	5.6
	Horizontal Displacement	2	5.3	8	44.4	15	41.7
	Distress in Superstructure	10	26.3	13	72.2	19	52.8
	Damage to Rails, Curbs, Sidewalks, Parapets	5	13.2	0	0.0	5	13.9
	Damage to Bearings	1	2.6	11	61.1	10	27.8
	Poor Riding Quality	2	5.3	0	0.0	2	5.6
	Not Given or Corrected During Construction	2	5.3	0	0.0	1	2.8
	None	11	29.0	0	0.0	2	5.6
	Total Bridges in Category	38		18		36	
Concrete	Damage to Abutments	7	15.2	1	9.1	3	50.0
	Damage to Piers	3	6.5	1	9.1	2	33.3
	Vertical Displacement	3	6.5	0	0.0	1	16.7
	Horizontal Displacement	0	0.0	5	45.5	2	33.3
	Distress in Superstructure	28	60.9	7	63.6	3	50.0
	Damage to Rails, Curbs, Sidewalks, Parapets	10	21.7	3	27.3	4	66.7
	Damage to Bearings	0	0.0	5	45.5	0	0.0
	Poor Riding Quality	4	8.7	0	0.0	0	0.0
	Not Given or Corrected During Construction	2	4.4	1	9.1	0	0.0
	None	7	15.2	0	0.0	0	0.0
	Total Bridges in Category	46		11		6	

[a]Percent of bridges in this category with indicated structural damage.

Table 22.--Types of Structural Damage Associated With Types of Abutments

Structural Damage (1)	Type of Abutment: Full Height — Number of Bridges (2)	Full Height — Percent of Category[a] (3)	Perched — Number of Bridges (4)	Perched — Percent of Category (5)	Spill-Through — Number of Bridges (6)	Spill-Through — Percent of Category (7)
Damage to Abutments	23	67.7	25	19.5	2	15.4
Damage to Piers	3	8.8	9	7.0	1	7.7
Vertical Displacement	1	2.9	8	6.3	1	7.7
Horizontal Displacement	5	14.7	26	20.3	3	23.1
Distress in Superstructure	5	14.7	71	55.5	10	76.9
Damage to Rails, Curbs, Sidewalks, Parapets	1	2.9	22	17.2	1	7.7
Damage to Bearings	4	11.8	22	17.2	1	7.7
Poor Riding Quality	1	2.9	9	7.0	1	7.7
Not Given or Corrected During Construction	1	2.9	6	4.7	0	0.0
None	2	5.9	20	15.6	0	0.0
Total Bridges in Category	34		128		13	

[a]Percent of bridges in this category with indicated structural damage.

Table 23.--Tolerance of Bridges to Structural Damage

Structural Damage (1)	Movement Category: Tolerable — Number of Bridges (2)	Tolerable — Percent of Category[a] (3)	Tolerable — Multiple Damage[b] (4)	Intolerable — Number of Bridges (5)	Intolerable — Percent of Category (6)	Intolerable — Multiple Damage (7)
Damage to Abutments	31	28.7	13	17	27.0	16
Damage to Piers	7	6.5	6	8	12.7	8
Vertical Displacement	3	2.8	2	10	15.9	8
Horizontal Displacement	9	8.3	5	26	41.3	22
Distress in Superstructure	40	37.0	20	36	57.1	29
Damage to Rails, Curbs, Sidewalks, Parapets	17	15.7	16	8	12.7	8
Damage to Bearings	8	7.4	6	16	25.4	16
Poor Riding Quality	1	0.9	1	11	17.5	4
Not Given or Corrected During Construction	6	5.6	0	2	3.2	0
None	25	23.2	0	0	0.0	0
Total Bridges in Category	108			63		

[a]Percent of bridges in this category with indicated structural damage.
[b]Multiple damage refers to the number of bridges in this category that had structural damage in addition to the indicated effects.

The relatively high vertical movements experienced by the spill-through abutments (Table 7) were found to be largely responsible for the high incidence of superstructure distress reported for bridges with this type of abutment.

3.4 Tolerance of Bridges to Foundation Movements

Overall, of the 171 structures where data on tolerance to foundation movements were available or could reasonably be assumed, the movements were considered tolerable for 108 bridges and intolerable for 63. The data in Table 23 show that, of all the structural effects associated with foundation movements that were considered tolerable, damage to abutments and distress in the superstructure appear most frequently. In most instances, the reported damage involved relatively minor cracking and/or the opening or closing of construction joints in the abutments and cracking and spalling of concrete decks. Of course, as would be expected, the foundation movements associated with all of the 25 bridges which experienced no structural damage were considered as being tolerable.

For those 63 bridges with intolerable movements, Table 23 shows that more than half were reported to have distress in the superstructure. Horizontal displacement and damage to bearings were also reported quite frequently. In addition, more than one quarter of those bridges with intolerable movements had abutment damage. As might have been expected, a larger number of bridges having intolerable movements exhibited multiple damaging effects than did the bridges having tolerable movements. **The most frequently occurring combinations of intolerable structural effects were distress in the superstructure, "horizontal displacement", damage to abutments, and damage to bearings.** A detailed study of the 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 horizontal movement of abutments, jamming the beams or girders against the back wall of the abutments, closing the expansion joints in the deck and causing serious damage to the bearings.**

Because of the rather common problem of poor riding quality associated with the approaches to bridges (34,37-39), riding quality was initially identified as one of the major areas of emphasis with respect to the evaluation of tolerable bridge movements. However, as shown in Table 23, with respect to the bridge structure itself, **poor riding quality was only reported for 12 bridges, and it was reported as being intolerable in 11 of these.** However, for these 11 structures, the maximum differential vertical settlement ranged from 2.4 inches (61.0 mm) to 35 inches (889 mm), with an average of 14.0 inches (355.6 mm). More important, however, is the fact that the maximum longitudinal angular distortion (differential vertical settlement divided by the span length) ranged from 0.0077 to 0.063, with an average of 0.021. As illustrated by data presented below, even the smallest of these values is larger than what might reasonably be expected to be tolerable either from a stress or serviceability standpoint. In other words, **the data appear to indicate that the foundation movements would become intolerable for some other reason before reaching a magnitude that would create intolerable rider discomfort.** Consequently, it appears that, in terms of static displacement, riding quality will probably not have to be given serious consideration in the establishment of tolerable movement criteria for highway bridges.

The results of the analysis of tolerance to bridge foundation movements in terms of type and magnitude of movement are presented in Tables 24 and 25. Table 24 gives a summary of movement characteristics, including type of movement, range of movements and average movements, while Table 25 gives the frequency of occurrence of the various ranges of magnitudes of both tolerable and intolerable movements. With regard to movements in general, it is evident from Table 24, as might have been expected, that the intolerable movements generally tended to be substantially larger than the tolerable movements. Table 25 shows that **moderate magnitudes of differential vertical movements occurring by themselves were most often considered tolerable, while horizonal movements were most commonly considered to be intolerable.** All 51 of the differential vertical settlements less than 2.0 inches (50.8 mm) and 95.3 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 differential vertical movements decreased significantly for values over 4.0 inches (101.6 mm). Only 57.1 percent of the differential vertical settlements between 4.0 inches (101.6 mm) and 8 inches (203.2 mm) and 30.0 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 inches (50.8 mm), the movements were considered tolerable in 83.3 percent of the cases. However, a large majority (78.9 percent) of the maximum horizontal movements of 2 inches (50.8 mm) and greater were found to be intolerable. Furthermore, Table 25 shows that even horizontal movements less than 2 inches (50.8 mm) were only reported as being tolerable in 68.2 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 cases, **when their magnitudes approached one inch (25.4 mm) and less.**

Although the sample sizes were smaller, the same general trends with respect to the magnitudes of tolerable and intolerable foundation movements, shown in Table 25 and described above, were observed to hold, regardless of span type (simply supported or continuous) and structural materials (steel or concrete). This is illustrated in Tables 26 and 27. However, the apparent lack of tolerance to horizontal movements tended to be slightly more pronounced for all continuous structures and for concrete bridges. Although these same general trends were also found to hold regardless of number of spans, when the tolerance data were broken down in terms of number of spans, the sample sizes were frequently too small to be statistically reliable.

When the data shown in Table 25 were broken down in terms of abutment type, some differences in tolerance to foundation movements became evident.

Table 24.--Summary of Tolerance to Movements in General

Tolerance to Movements (1)	Movement Type (2)	Frequency: Number of Bridges (3)	Frequency: Percent Moved (4)	Range in Inches (5)	Average in Inches (6)	Standard Deviation in Inches (7)
Tolerable	All Types	103	100.0			
	Vertical	71	68.9	0.1-24.2	2.0	3.2
	Horizontal	9	8.7	0.1-7.0	1.7	1.9
	Vertical &	23	22.3	0.1-11.4	2.4	2.4
	Horizontal			0.1-20.0	1.7	3.3
Intolerable	All Types	54	100.0			
	Vertical	13	24.1	1.0-19.5	7.3	4.9
	Horizontal	16	29.6	0.5-12.0	3.9	3.1
	Vertical &	25	46.3	0.6-50.4	10.2	11.5
	Horizontal			1.0-14.4	4.0	3.3

Note: 1 inch = 25.4 mm.

Table 25.--Range of Movement Magnitudes Considered Tolerable or Intolerable

Interval[a] in Inches (1)	Number of Bridges With the Given Type of Movement							
	Vertical Only		Horizontal Only		Vertical and Horizontal			
					Vertical Component		Horizontal Component	
	Tolerable (2)	Intolerable (3)	Tolerable (4)	Intolerable (5)	Tolerable (6)	Intolerable (7)	Tolerable (8)	Intolerable (9)
0.0 - 0.9	30	0	2	0	7	0	7	0
1.0 - 1.9	21	0	5	1	6	3	6	7
2.0 - 3.9	10	3	1	7	6	4	5	8
4.0 - 5.9	1	1	2	0	2	4	0	7
6.0 - 7.9	3	2	1	3	0	2	0	1
8.0 - 9.9	0	4	0	3	0	1	0	2
10.0 - 14.9	2	1	0	2	0	4	0	2
15.0 - 19.9	1	2	0	0	0	2	0	1
20.0 - 60.0	0	0	0	0	0	4	1	0
Total	68	13	11	16	21	24	19	28

[a]For vertical movements, 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.

Table 26.--Range of Movement Magnitudes Considered Tolerable or Intolerable in Terms of Span Type

Type of Span (1)	Interval[a] in Inches (2)	Number of Bridges With the Given Type of Movement							
		Vertical Only		Horizontal Only		Vertical and Horizontal			
						Vertical Component		Horizontal Component	
		Tolerable (3)	Intolerable (4)	Tolerable (5)	Intolerable (6)	Tolerable (7)	Intolerable (8)	Tolerable (9)	Intolerable (10)
Simply Supported	0.0 - 0.9	16	0	2	0	3	0	2	0
	1.0 - 1.9	7	0	2	0	4	1	5	5
	2.0 - 3.9	2	0	1	5	3	1	4	2
	4.0 - 5.9	0	0	0	0	1	4	0	2
	6.0 - 7.9	1	0	0	0	0	0	0	1
	8.0 - 9.9	0	2	0	0	0	1	0	1
	10.0 - 14.9	2	1	0	0	0	0	0	0
	15.0 - 19.9	0	2	0	0	0	1	0	0
	20.0 - 60.0	0	0	0	0	0	1	0	0
	Total	28	5	5	5	11	9	11	11
Continuous	0.0 - 0.9	16	0	0	0	2	0	2	1
	1.0 - 1.9	9	0	2	1	0	1	0	0
	2.0 - 3.9	4	3	0	4	0	3	0	3
	4.0 - 5.9	0	1	2	0	1	0	0	4
	6.0 - 7.9	2	0	1	1	0	0	0	0
	8.0 - 9.9	0	1	0	2	0	0	0	1
	10.0 - 14.9	0	0	0	1	0	2	0	1
	15.0 - 19.9	0	0	0	0	0	1	0	0
	20.0 - 60.0	0	0	0	0	0	3	1	0
	Total	31	5	5	9	3	10	3	10

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

Table 27.--Range of Movement Magnitudes Considered Tolerable or Intolerable in Terms of Construction Material

Construction Material (1)	Interval[a] in Inches (2)	Number of Bridges With the Given Types of Movement							
		Vertical Only		Horizontal Only		Vertical and Horizontal			
						Vertical Component		Horizontal Component	
		Tolerable (3)	Intolerable (4)	Tolerable (5)	Intolerable (6)	Tolerable (7)	Intolerable (8)	Tolerable (9)	Intolerable (10)
Steel	0.0 - 0.9	14	0	1	0	5	0	4	0
	1.0 - 1.9	7	0	2	1	5	2	5	2
	2.0 - 3.9	3	1	0	5	3	2	5	3
	4.0 - 5.9	0	0	2	0	1	2	0	7
	6.0 - 7.9	1	1	0	1	0	2	0	0
	8.0 - 9.9	0	1	0	2	0	0	0	2
	10.0 - 14.9	2	0	0	1	0	3	0	0
	15.0 - 19.9	1	2	0	0	0	1	0	1
	20.0 - 60.0	0	0	0	0	0	3	0	0
	Total	28	5	5	10	14	15	14	15
Concrete	0.0 - 0.9	14	0	0	0	1	0	1	0
	1.0 - 1.9	11	0	0	0	0	1	2	0
	2.0 - 3.9	6	0	0	2	2	0	1	1
	4.0 - 5.9	1	1	0	0	1	0	0	0
	6.0 - 7.9	2	1	1	1	0	0	0	0
	8.0 - 9.9	0	2	0	1	0	0	0	0
	10.0 - 14.9	0	1	0	2	0	0	0	0
	15.0 - 19.9	0	0	0	0	0	0	0	0
	20.0 - 60.0	0	0	0	0	0	0	1	0
	Total	34	5	1	4	4	1	5	1

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

Again, the sample sizes were smaller than shown in Table 25, but the data definitely showed that bridges with full height abutments were more tolerant to both differential vertical and horizontal movements than bridges with either perched or spill-through abutments. This is shown in Table 28, where a comparison between the tolerance to movements of bridges with full height and perched abutments is presented. For full height abutments, all of the differential vertical movements less than 4.0 inches (101.6 mm) were reported as being tolerable. For bridges with perched abutments, although 95.7 percent of the differential vertical movements less than 4.0 inches (101.6 mm) were reported as being tolerable, only 30.8 percent of those greater than 4.0 inches (101.6 mm) were considered tolerable. Similar trends were observed with respect to horizontal movements. These findings seem to reflect the nature and seriousness of the structural damage induced by the foundation movements. Although it would be expected that this difference in tolerance to foundation movements might be explained in terms of the design and construction parameters that would commonly be associated with the selection of a particular type of abutment, no meaningful correlations between these parameters and tolerance to movements have been found.

The influence of span length on the tolerance of bridges to foundation movements were studied in terms of maximum longitudinal angular distortion (differential vertical settlement divided by span length). There were 104 of the 171 bridges with tolerance data, where the data were sufficiently complete to permit this type of analysis. Of these 104 bridges, the movements were reported to be tolerable for 76 and intolerable for 28. Table 29 presents a summary of the frequency of occurrence of the various ranges of magnitudes of angular distortion considered tolerable and intolerable for all types of bridges included in this portion of the study and for a subdivision by span type. When all of the bridges in the analysis are considered, Table 29 shows that all 30 of the angular distortions less than 0.001 and 95.6 percent of the 68 angular distortions less than 0.004 were considered to be tolerable. However, only 43.8 percent of the values of angular distortion between 0.004 and 0.01, and 20.0 percent of those over 0.01, were considered to be tolerable. **This would suggest that, on the basis of all the available field data, an upper limit on angular distortion of 0.004 would be reasonable.** However, when the data are subdivided by span type, Table 29 shows that the simply supported bridges tended to be 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 that shown in Table 29. **For the continuous bridges, 96.0 percent of the 25 angular distortions less than 0.004 were considered to be tolerable,** while only 23.1 percent of those over 0.004 were considered to be tolerable. In contrast, **for the simply supported bridges, 97.1 percent of the 34 angular distortions less than 0.005 were reported as being tolerable.** Translated in terms of differential settlement, these data suggest that, for simply supported bridges, differential settlements of 3.0 inches (76.2 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.

Table 28.--Range of Movement Magnitudes Considered Tolerable or Intolerable in Terms of Abutment Type

Type of Abutment (1)	Interval[a] in Inches (2)	Number of Bridges With the Given Type of Movement							
		Vertical Only		Horizontal Only		Vertical and Horizontal			
						Vertical Component		Horizontal Component	
		Tolerable (3)	Intolerable (4)	Tolerable (5)	Intolerable (6)	Tolerable (7)	Intolerable (8)	Tolerable (9)	Intolerable (10)
Full Height	0.0 - 0.9	6	0	0	0	2	0	2	0
	1.0 - 1.9	6	0	2	0	1	0	1	1
	2.0 - 3.9	2	0	0	3	1	0	2	0
	4.0 - 5.9	0	0	0	0	1	1	0	0
	6.0 - 7.9	0	0	0	0	0	0	0	0
	8.0 - 9.9	0	0	0	0	0	0	0	0
	10.0 - 14.9	0	0	0	0	0	0	0	0
	15.0 - 19.9	0	0	0	0	0	0	0	0
	20.0 - 60.0	0	0	0	0	0	0	0	0
	Total	14	0	2	3	5	1	5	1
Perched	0.0 - 0.9	23	0	2	0	3	0	3	0
	1.0 - 1.9	14	2	3	1	4	3	4	2
	2.0 - 3.9	6	0	1	6	2	2	2	5
	4.0 - 5.9	1	3	2	0	1	1	0	6
	6.0 - 7.9	2	3	1	2	0	1	0	0
	8.0 - 9.9	0	2	1	3	0	0	0	2
	10.0 - 14.9	1	2	0	1	0	3	0	1
	15.0 - 19.9	0	0	0	0	0	2	0	0
	20.0 - 60.0	0	0	0	0	0	4	1	0
	Total	47	12	10	13	10	16	10	16

[a]For vertical movements, 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.

Table 29.--Ranges of Magnitudes of Longitudinal Angular Distortion Considered Tolerable or Intolerable

Angular Distortion Interval (x 10^{-3}) (1)	Number of Bridges of the Given Type and Tolerance					
	All Bridges		Span Type			
			Simple		Continuous	
	Tolerable (2)	Intolerable (3)	Tolerable (4)	Intolerable (5)	Tolerable (6)	Intolerable (7)
0 - 0.99	30	1	15	1	12	0
1.0 - 1.99	18	2	7	0	9	1
2.0 - 2.99	10	0	4	0	2	0
3.0 - 3.99	7	0	5	0	1	0
4.0 - 4.99	4	2	2	0	0	2
5.0 - 5.99	0	2	0	1	0	1
6.0 - 7.99	2	4	1	2	1	2
8.0 - 9.99	1	1	0	1	1	0
10.0 - 19.99	3	9	2	3	1	3
20.0 - 39.9	1	5	1	4	0	1
40.0 - 59.9	0	0	0	0	0	0
60.0 - 79.9	0	2	0	1	0	1
Total	76	28	37	13	27	11

Table 30.--Ranges of Magnitudes of Longitudinal Angular Distortion Considered Tolerable or Intolerable in Terms of Construction Material

Angular Distortion Interval (X10^{-3}) (1)	Number of Bridges of Given Material and Tolerance			
	Concrete		Steel	
	Tolerable (2)	Intolerable (3)	Tolerable (4)	Intolerable (5)
0.0 - 0.99	13	0	16	1
1.0 - 1.99	10	0	6	2
2.0 - 2.99	3	0	7	0
3.0 - 3.99	2	0	5	0
4.0 - 4.99	2	1	1	1
5.0 - 5.99	0	0	0	2
6.0 - 7.99	1	0	1	1
8.0 - 9.99	1	0	0	1
10.0 - 19.99	1	2	2	4
20.0 - 39.9	0	2	1	3
40.0 - 59.9	0	0	0	0
60.0 - 79.9	0	0	0	1
Total	33	5	39	16

When the data in Table 29 were broken down in terms of material type, as shown in Table 30, they suggested that the **concrete bridges might be slightly more tolerant to angular distortion than the steel bridges.** For the concrete bridges, 96.7 percent of the 31 angular distortions less than 0.005 were considered to be tolerable, while for the steel bridges, only 89.7 percent of the 39 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 terms of the tolerance data. This implies that the frequently reported distress in the superstructure of concrete bridges was quite often considered to be tolerable. A detailed breakdown of the data in Table 23, in terms of material type, as shown in Table 31, provided verification for this observation.

Table 31.--Tolerance of Bridges to Structural Damage in Terms of Construction Material

Construction Material (1)	Structural Damage (2)	Movement Category: Tolerable — Number of Bridges (3)	Tolerable — Percent of Category[a] (4)	Tolerable — Multiple Damage[b] (5)	Intolerable — Number of Bridges (6)	Intolerable — Percent of Category (7)	Intolerable — Multiple Damage (8)
Steel	Damage to Abutments	23	45.1	6	12	37.5	11
	Damage to Piers	3	5.9	3	4	12.5	4
	Vertical Displacement	1	2.0	1	4	12.5	4
	Horizontal Displacement	4	7.8	2	16	50.0	15
	Distress in Superstructure	9	17.7	6	23	71.9	20
	Damage to Rails, Curbs, Sidewalks, Parapets	6	11.7	5	3	9.4	3
	Damage to Bearings	7	13.7	5	12	37.5	12
	Poor Riding Quality	0	0.0	0	4	12.5	3
	Not Given or Corrected During Construction	13	25.5	0	0	0.0	0
	None	2	3.9	0	1	3.1	0
	Total Bridges in Category	51			32		
Concrete	Damage to Abutments	8	19.1	7	2	18.2	2
	Damage to Piers	4	9.5	3	1	9.1	1
	Vertical Displacement	2	4.8	1	3	27.3	2
	Horizontal Displacement	2	4.8	2	3	27.3	2
	Distress in Superstructure	26	61.9	14	5	45.5	3
	Damage to Rails, Curbs, Sidewalks, Parapets	11	26.2	11	2	18.2	2
	Damage to Bearings	0	0.0	0	2	18.2	2
	Poor Riding Quality	1	2.4	1	3	27.3	1
	Not Given or Corrected During Construction	7	16.7	0	0	0.0	0
	None	2	4.8	0	0	0.0	0
	Total Bridges in Category	42			11		

[a]Percent of bridges in this category with indicated structural damage.
[b]Multiple damage refers to the number of bridges in this category that had structural damage in addition to the indicated effects.

4. ANALYTICAL STUDIES OF STEEL BRIDGES

The primary objective of the analytical studies reported herein was to evaluate the effects of differential vertical movements of various magnitudes on continuous two-span and four-span steel bridges for a wide variety of span lengths. In general, the tolerance of superstructure systems to support settlements was investigated as a function of span length, stiffness and other problem parameters. Both static and dynamic loading conditions were studied. The results are presented in a series of graphs 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. For the sake of brevity, only a limited discussion of these analyses, their results, and observations are presented here, and the reader is referred to the works of Haslebacher (36) and GangaRao and Halvorsen (25) for additional information on the methods of analysis and detailed results.

4.1 Methods of Analysis

4.1.1 Static Loading

The analysis of the effect of support settlement for static loading was accomplished with the aid of ICES-STRUDL-II (49) computer package. The bridge superstructures were designed according to the "Standard Specifications for Highway Bridges" (5) of the American Association of State Highway and Transportation Officials (AASHTO). The assumed loading conditions for the bridges included both live and dead loads. The live loading consisted of the AASHTO HS-20-44 wheel loading or its equivalent lane loading (5), 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 movement.

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. In addition, 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 meters), were investigated. More specifically, the short span bridges used W30, W33 and W36 stringers, with 6 and 8 feet (1.83 and 2.44 meters) stringer spacing (see Figures 1 and 2), and spans of 30, 40, 50 and 60 feet (8.1, 12.2, 15.2 and 18.3 meters). The intermediate span bridges used W36 stringers with cover plates, an 8 foot (2.44 meter) stringer spacing, and spans of 100, 125 and 150 feet (30.5, 38.1 and 45.7 meters). The plate girder bridges utilized an 8 foot (2.44 meter) stringer spacing, and spans of 150, 200 and 250 feet (45.7, 61.0 and 76.2 meters). 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.

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.

For the two-span and four-span slab/stringer systems, the computer-aided analysis 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 3 and 4. Additional moment diagrams for bridges with a range of span lengths up to 250 feet (76.2 meters) are given in the Appendix. From the moment diagrams, the effect of settlement on member stresses was determined.

4.1.2 Dynamic Loading

The vibrations induced by traffic are mainly 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 bridge structures were modeled using a specially modified form of the governing equation developed by GangaRao and Wilhelm (27). The dynamic truck wheel loading was modeled using the forcing function suggested by Linger and Hulsbos (48). The analysis of each structure considered the effect of the weight of the load, the stiffness of the structures, the velocity of the moving

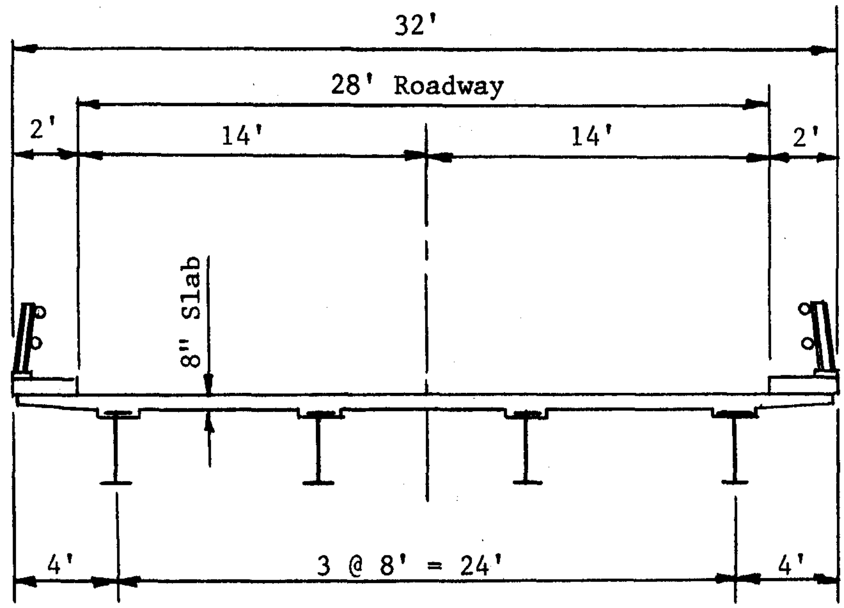

Figure 1 - Cross-Section of Typical Slab/Stringer Bridge with 8 Foot Stringer Spacing

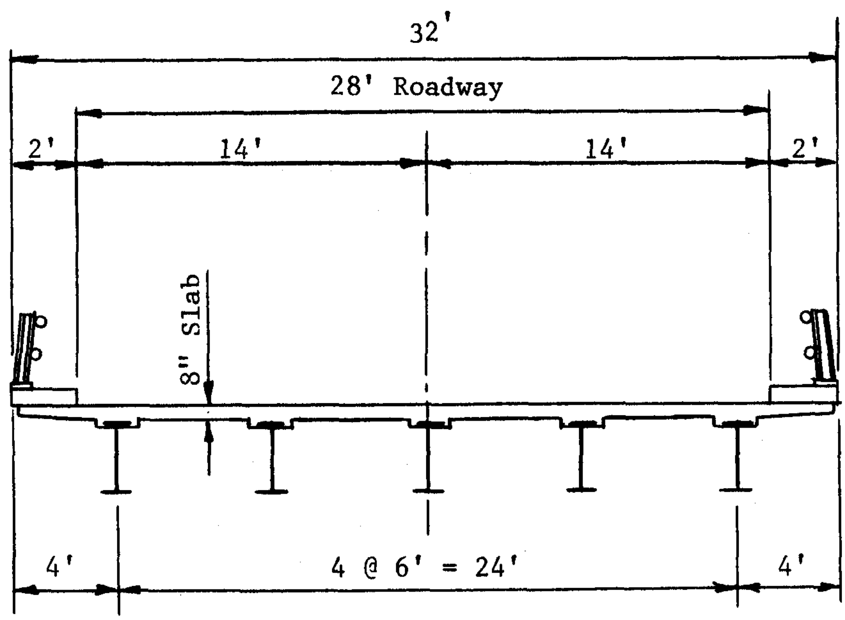

Figure 2 - Cross-Section of Typical Slab/Stringer Bridge with 6 Foot Stringer Spacing

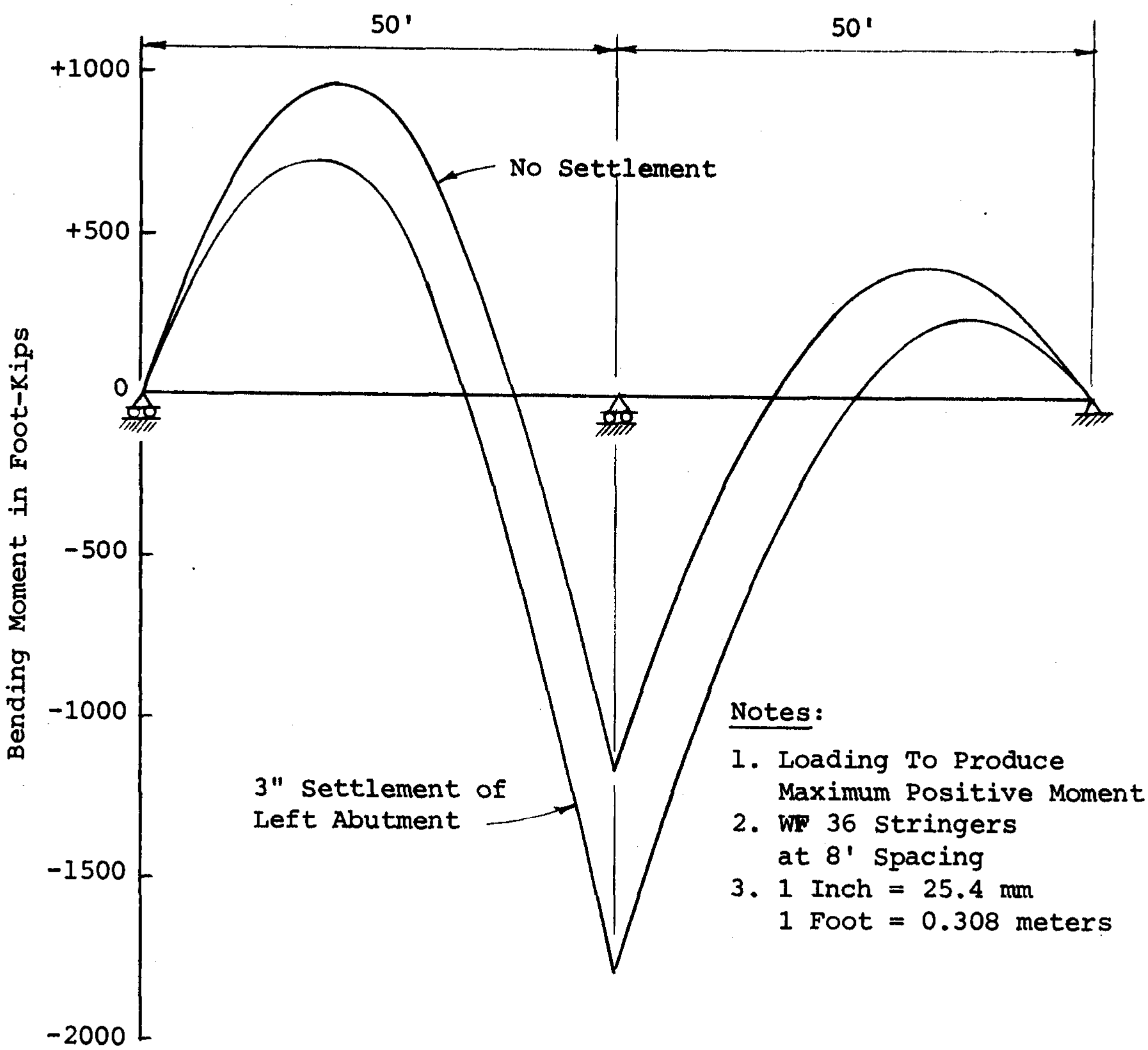

Figure 3 - Typical Moment Diagram for Two-Span Continuous Bridge Loaded with Dead Load, Live Load and Settlement of Left Abutment

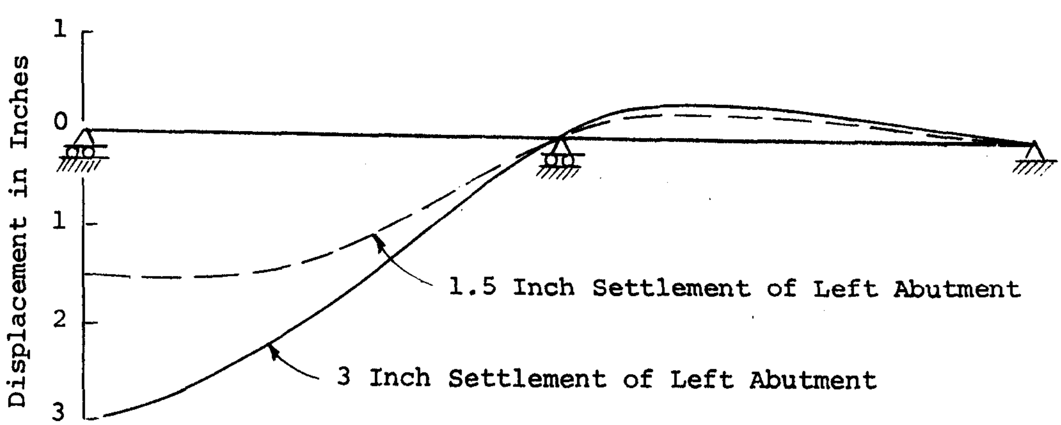

Figure 4 - Typical Displacement Diagrams for Two-Span Continuous Bridge Loaded with Dead Load, Live Load and Settlement of Left Abutment

load, and the truck axle spacing. Computer methods were utilized to perform these analyses. The details of the analytical procedures that were used have been presented by Haslebacher (36).

4.2 Results of Analysis of Slab/Stringer Systems

4.2.1 Static Loading

The results of the analysis of the slab/stringer systems showed that two settlement conditions were critical. For the two-span bridges, the maximum negative stress occurred at the center support, with settlement at the exterior suppport, 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 center support, under conditions of loading to produce maximum positive moment in that span. These results are predictable in terms of generalized continuous beam behavior (36). It should be recognized, however, that the combinations of loading and support settlement used in this study were limited to some extent and there may be other combinations of loading and multiple support settlements that could produce different results.

A synthesis of the data for 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 both of 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 5 and 6. These figures show the effects of changing span length on the percentage increase in stresses in two-span continuous bridges caused by differential settlements of one, two and three inches (25.4, 50.8, and 76.2 mm) 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 5 and 6) 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. It is clear from Figures 5 and 6 that even a one inch (25.4 mm) differential settlement of abutment or pier would cause an intolerable increase in stress (about 150 percent) for a two-span bridge with 30 foot (9.1 meter) spans. This effect could be expected to be even greater for the four-span bridges. However, as the span lengths

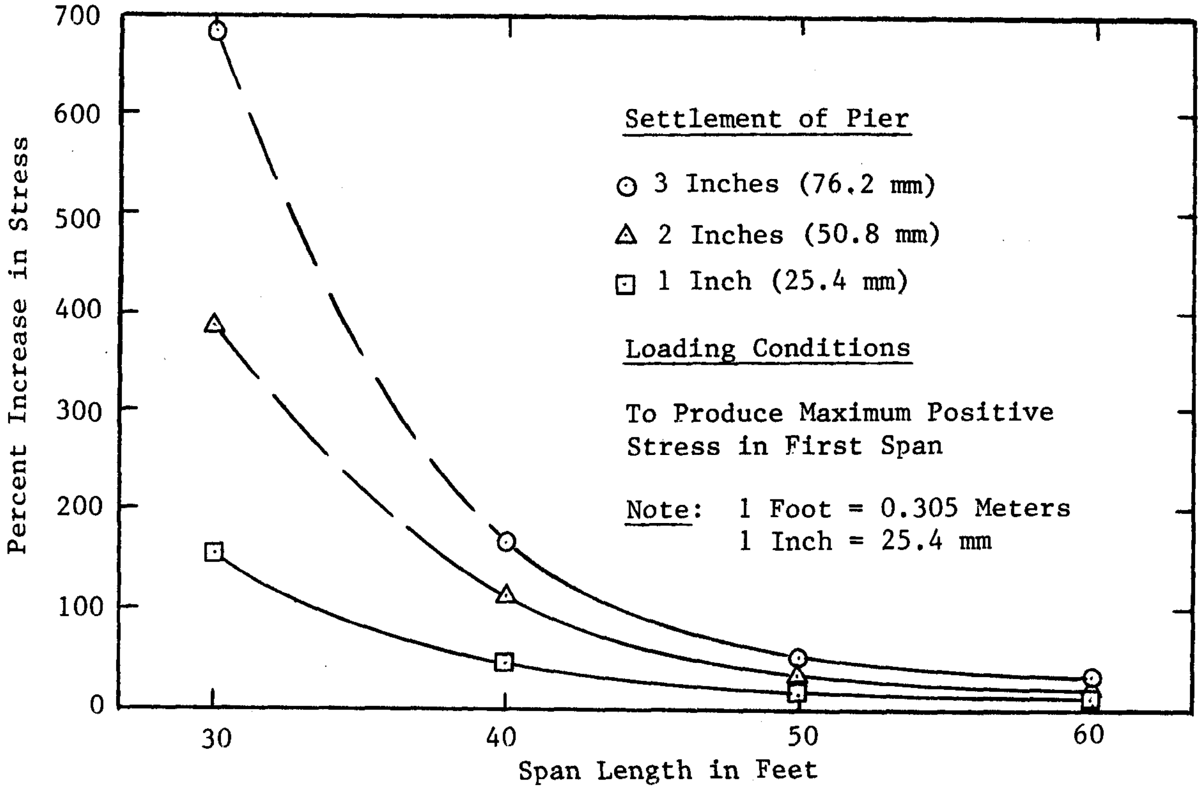

Figure 5 - Theoretical Percent Increase in Positive Stress at Mid-Point of First Span vs. Span Length for Two-Span Continuous Bridge (W36 Composite)

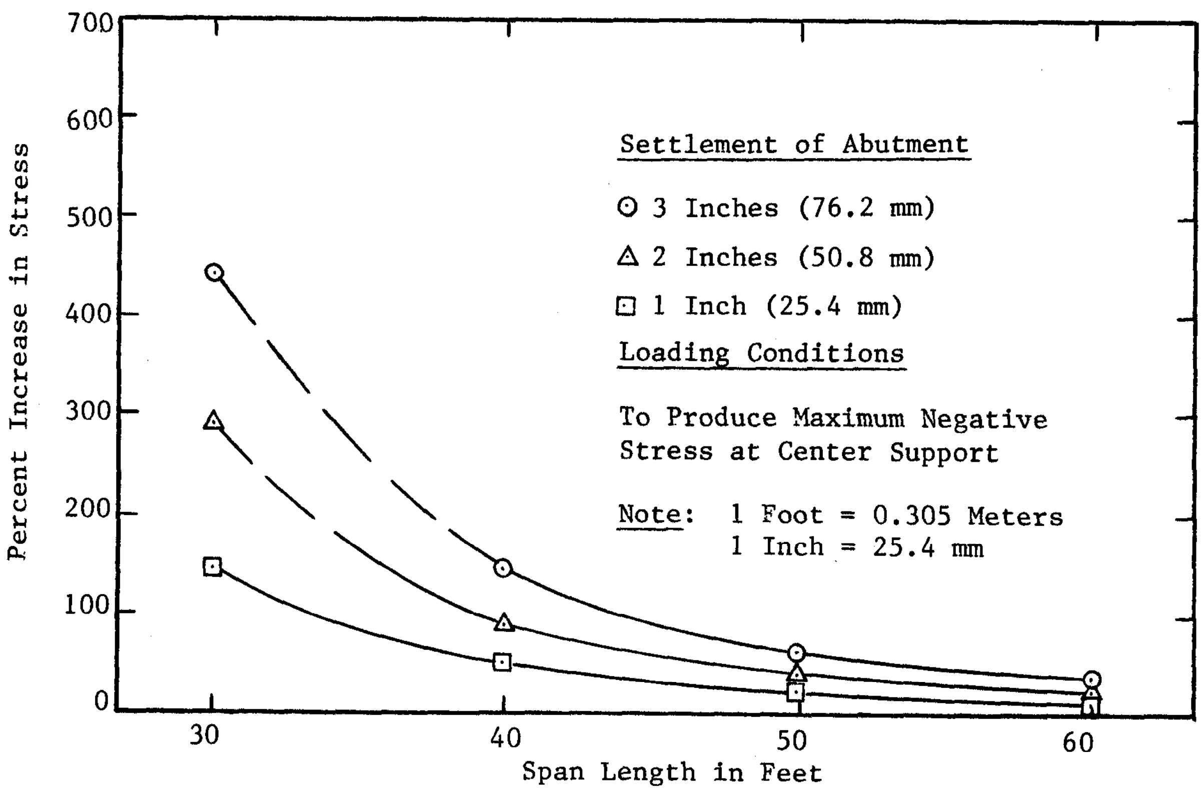

Figure 6 - Theoretical Percent Increase in Negative Stress at Center Support vs. Span Length for Two-Span Continuous Bridge (W36 Composite)

increase, the stresses caused by differential settlements decrease substantially, as illustrated in Figures 5 and 6 and by a comparison of the typical moment diagrams given in Figure 3 and Figures 42-44. This is further illustrated by the typical results of the analyses given in Table 32, 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 and one inch (25.4 mm) settlement cases for the shorter spans are, in part, the result of the overdesign produced by using W36 stringers for these short spans. The data in Table 32 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.**

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 5 and 6 for the W36 - composite design were compared with similar data developed for designs using W33 and W30 stringers, as illustrated in Figures 7 and 8 for the case of a 3 inch differential settlement of pier and abutment, respectively. These figures show 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 9 and 10 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. (327,741 mm) or less for both two-span and four-span bridges.**

4.2.2 Dynamic Loading

The results of the analysis of the slab/stringer systems under dynamic loading, in terms of computed dynamic deflections, are presented in Figure 11. These data were compared with similar data presented in the 1979 Supplements to the Ontario Highway Bridge Design Code (56), and good agreement was achieved. Both sets of data indicated the likelihood that excessive dynamic deflection and frequency increases might occur as the "resonance factor", i.e., the ratio of the forced (ω_f) to the natural (ω_n) frequencies, approaches one. For the purposes of this study, the "resonance factor" has been defined as $(2v/\pi sn^2)\ \sqrt{mL^4/EI}$, where v is the velocity of the moving load, L is the total length of the bridge, s is the truck axle spacing, n is the number of spans, m is the mass per unit length of bridge section, and EI is the flexural rigidity of the composite bridge section. It was found that, in order to limit the dynamic deflections to 1.2 times the static deflections, the value of the "resonance factor" should be less than 0.5 or greater than 1.5. This relationship can be expressed as:

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Table 32.--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] (1)	Settlement in Inches (2)	Maximum Calculated Stresses (ksi)	
		Two-Span Bridges With Settlement of Exterior Support (3)	Four-Span Bridges With Settlement of First Interior Support (4)
30	0	14.6	11.0
	1	18.8	21.0
	2	28.2	36.5
	3	38.4	50.5
40	0	--	14.0
	1	--	21.0
	2	--	28.0
	3	--	37.0
50	0	18.0	17.0
	1	22.5	23.2
	2	26.5	29.0
	3	30.0	35.0
60	0	19.0	18.5
	1	21.0	21.3
	2	23.2	24.5
	3	26.0	28.5
100	0	18.8	18.4
	3	21.2	23.0
120	0	18.0	--
	3	20.4	--
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 to 60 foot spans were designed with W36 stringers, the 100 and 120 foot spans were 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.

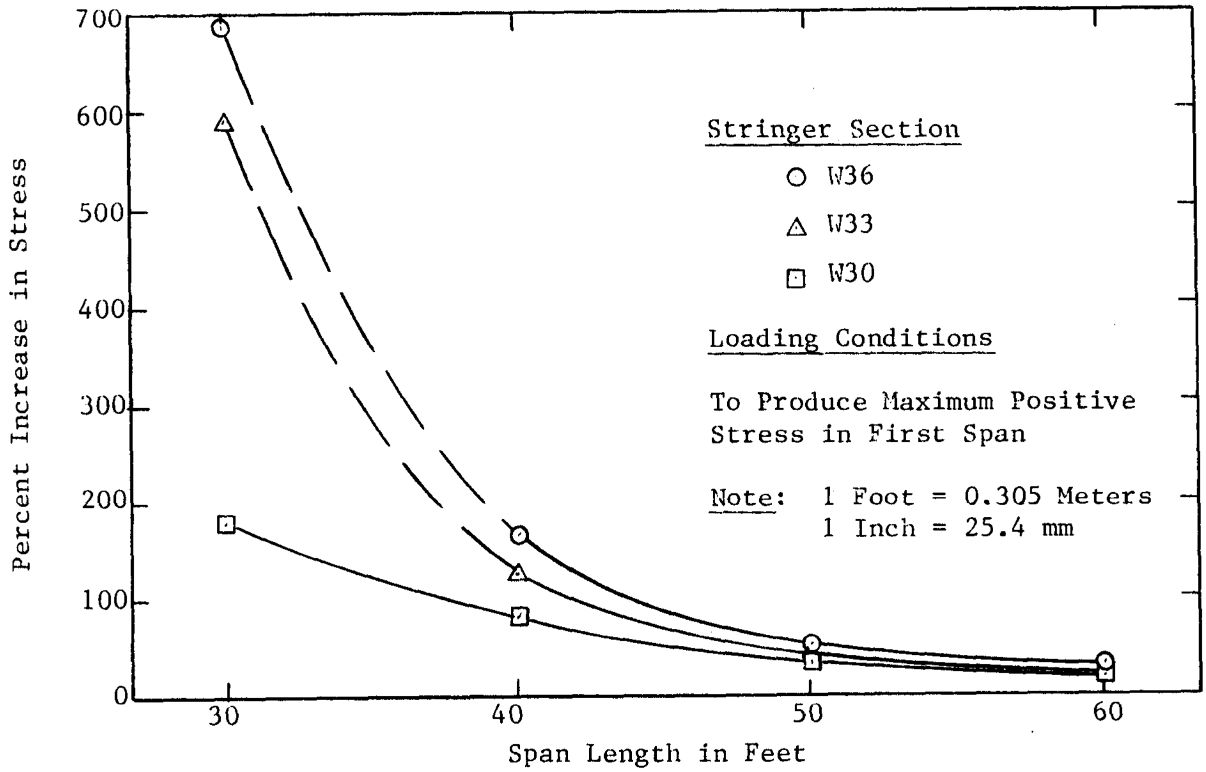


Figure 7 - Theoretical Percent Increase in Positive Stress in First Span for 3 Inch Pier Settlement for Two-Span Continuous Bridges with W30, W33 and W36 Stringers

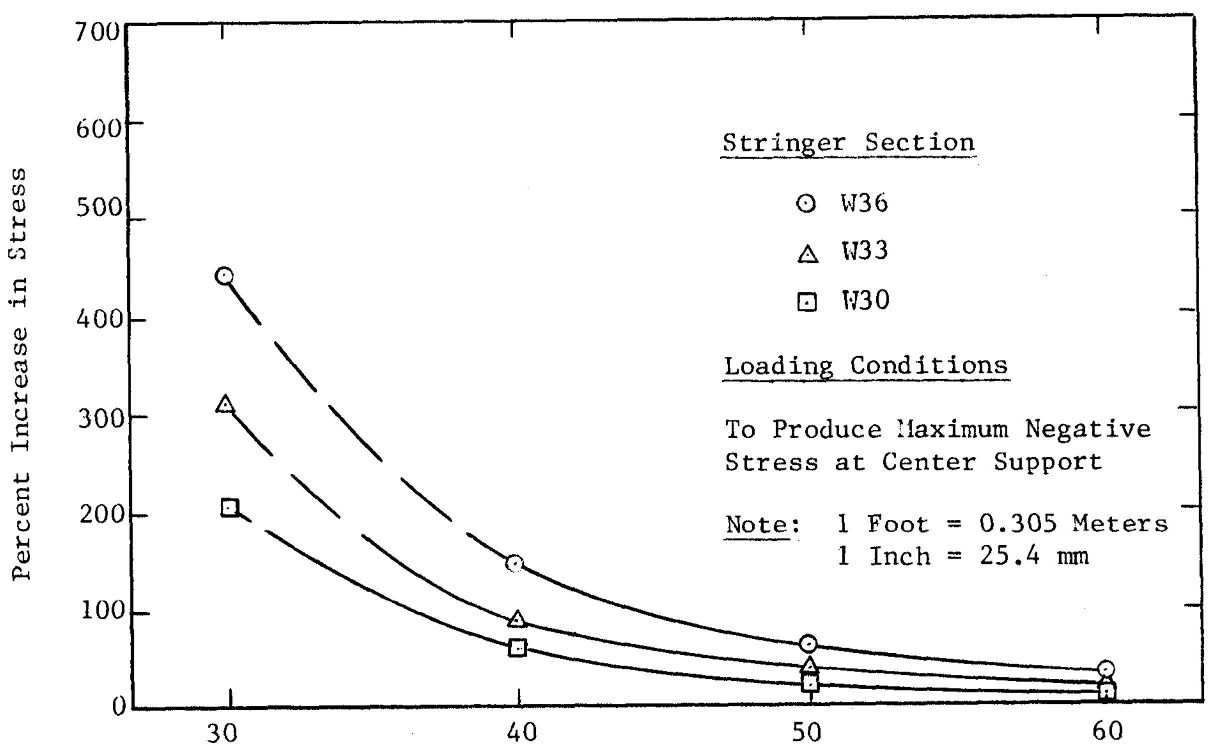


Figure 8 - Theoretical Percent Increase in Negative Stress at Center Support for 3 Inch Abutment Settlement for Two-Span Continuous Bridges with W30, W33 and W36 Stringers

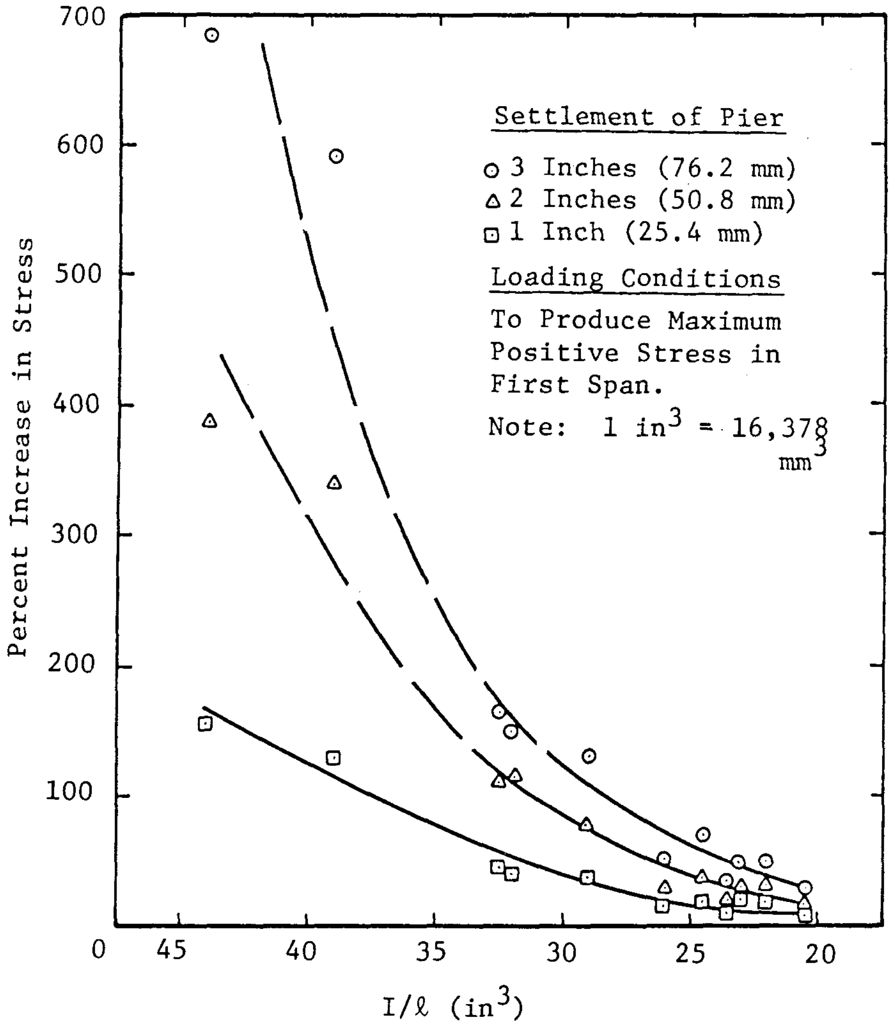


Figure 9 - Theoretical Percent Increase in Positive Stress at Mid-point of First Span vs. I/ℓ for 2-Span Continuous Bridges

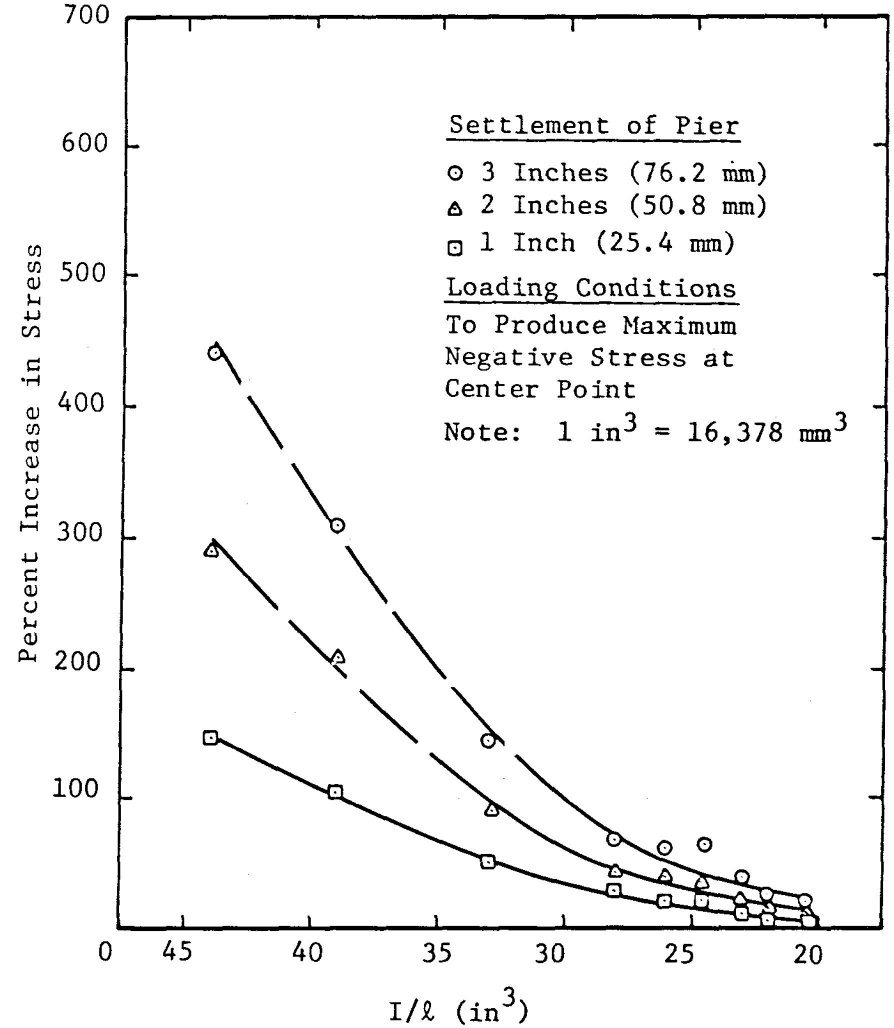


Figure 10- Theoretical Percent Increase in Negative Stress at Center Support vs. I/ℓ for 2-Span Continuous Bridges

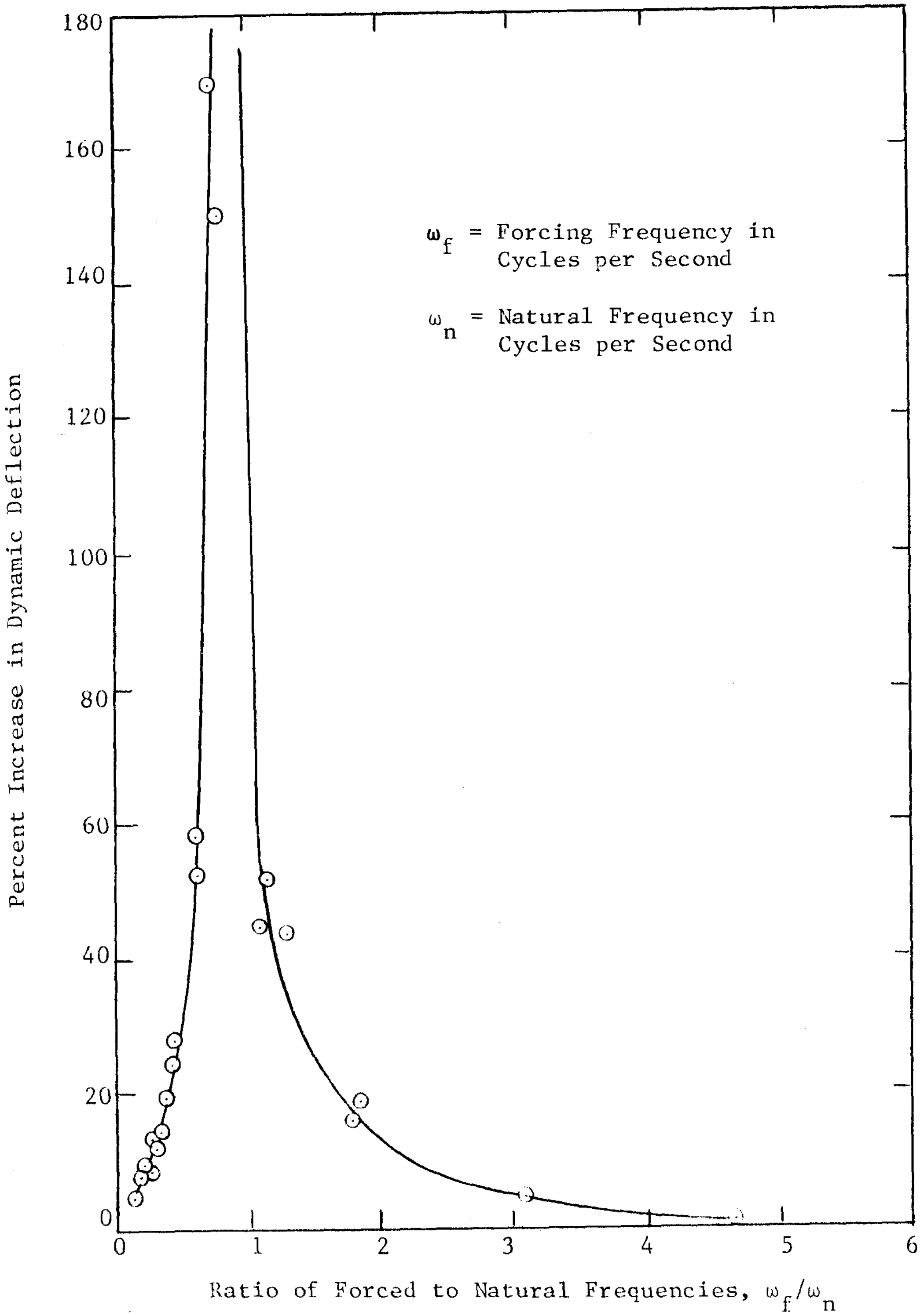


Figure 11 - Theoretical Percent Increase in Dynamic Deflection for Slab/Stringer Systems as a Function of the Resonance Factor, ω_f/ω_n

$$\frac{\omega_f}{\omega_n} = \frac{2v}{\pi s n^2} \sqrt{\frac{mL^4}{EI}} \begin{matrix} > 1.5 \\ < 0.5 \end{matrix} \qquad (1)$$

By using this inequality, the designer can determine if a proposed structure has sufficient mass and stiffness to prevent excessive dynamic deflections. The comparison of calculated vibrations with the human response data presented by Wright and Green (78) suggested that, if the dynamic deflections are within tolerable limits, as defined by Equation 1, the dynamic vibrations will be tolerable from a human response viewpoint.

It was found that the criterion embodied in Equation 1 can be applied both for the normal bridge deck and for the "ramp" effect produced by differential settlement of abutment or piers. However, in the latter case, a study of data produced by vehicular traffic on bridges and roads (46, 47, 76) has indicated a maximum of 20 percent increase in forcing frequency from the normal road surface to the "ramp" condition. Hence, in considering the "ramp" effect produced by differential settlement, the forcing frequency for ramp effects, ω_{fr}, should be taken as 1.2 ω_f in applying Equation 1. A comparison of the results of the use of Equation 1 with limited field data showed good agreement.

4.3 Results of Analysis of Truss Systems

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), as shown in Figure 12. It can be seen that 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, as shown in Figure 13.

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, as illustrated in Figure 14. 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, as shown in Figure 15.

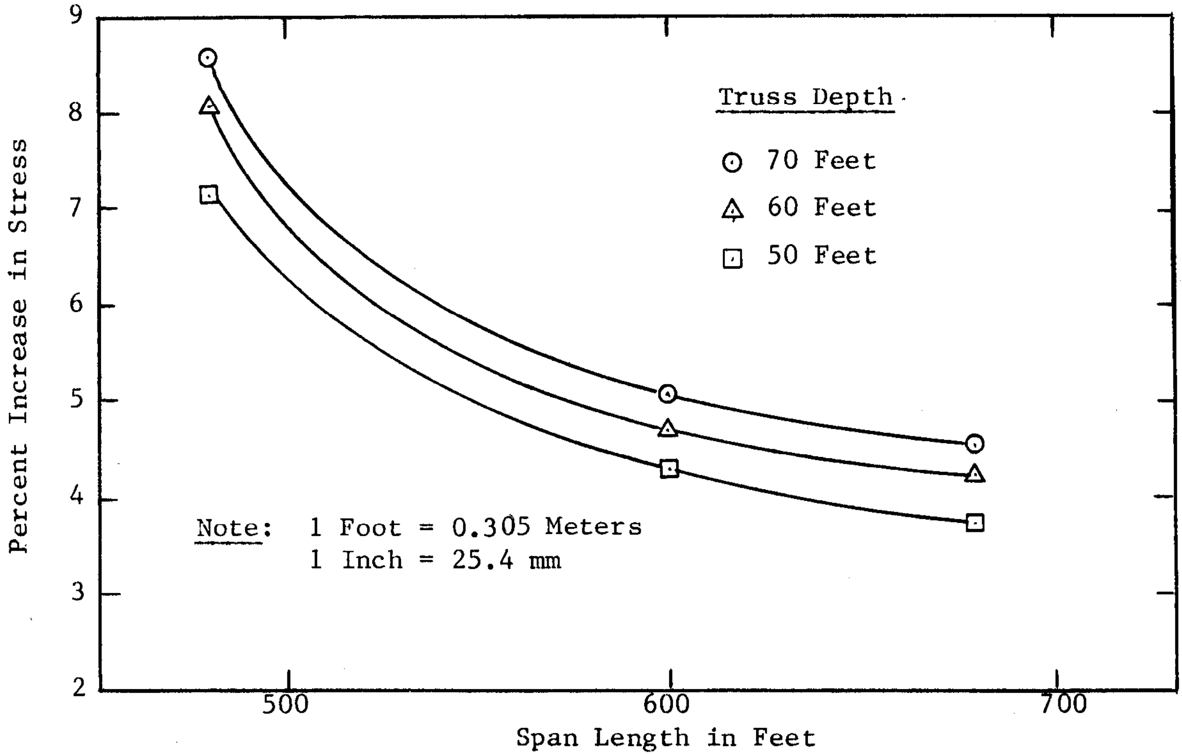


Figure 12 - Theoretical Percent Increase in Stress at Quarter Point of Two-Span Continuous Parallel Chord Trusses Caused by 3 Inch Pier Settlement

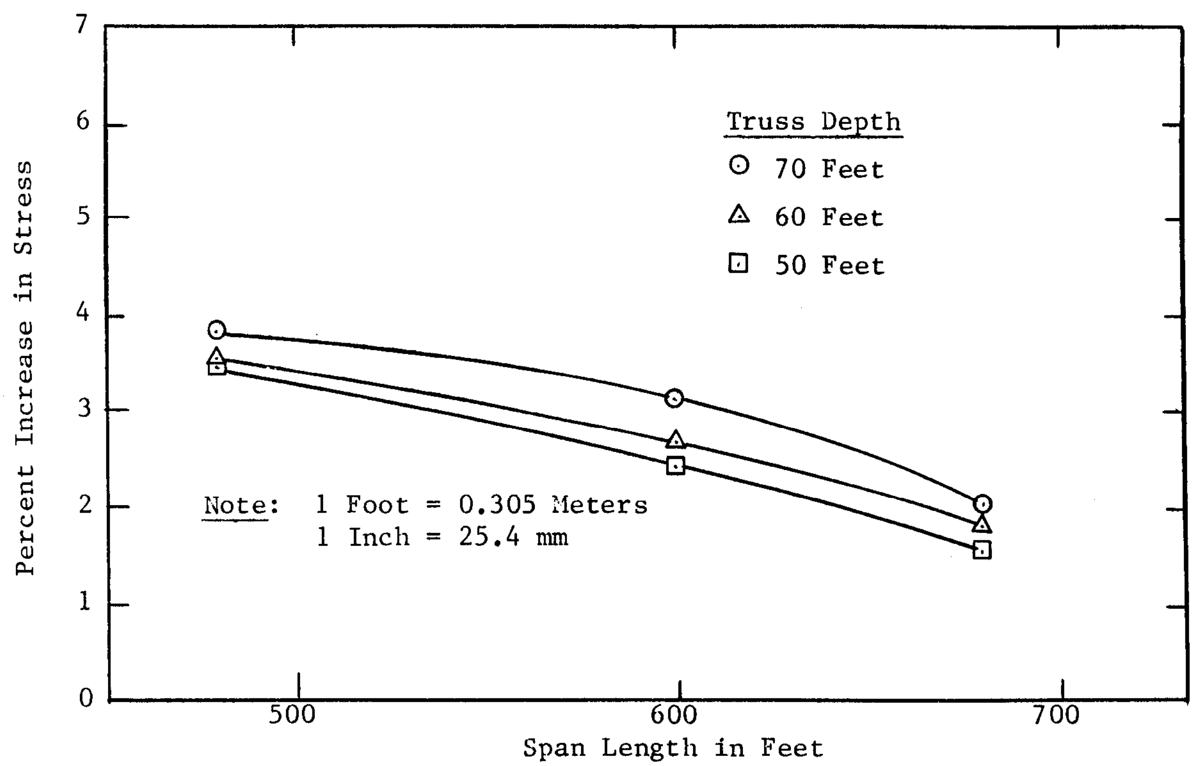


Figure 13 - Theoretical Percent Increase in Stress at Center Support of Two-Span Continuous Parallel Chord Trusses Caused by 3 Inch Abutment Settlement

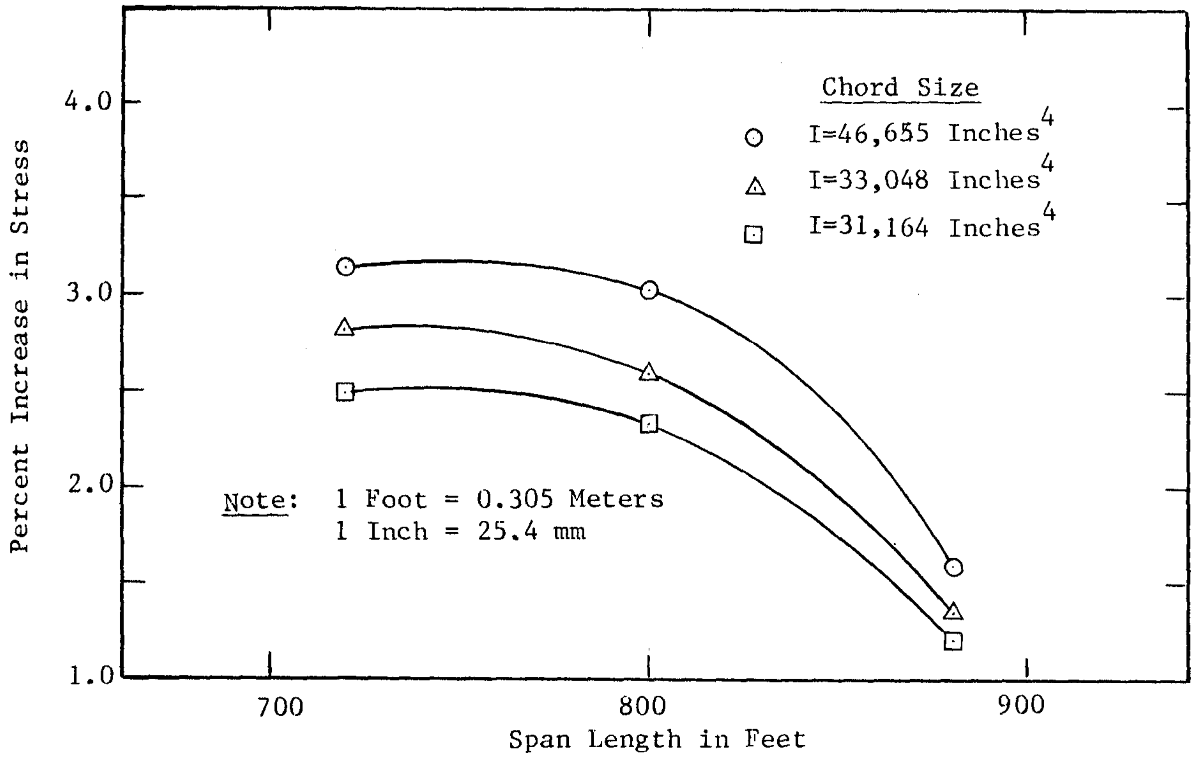


Figure 14 - Theoretical Percent Increase in Stress at Center Support of Two-Span Continuous Non-Parallel Chord Trusses Caused by 3 Inch Abutment Settlement

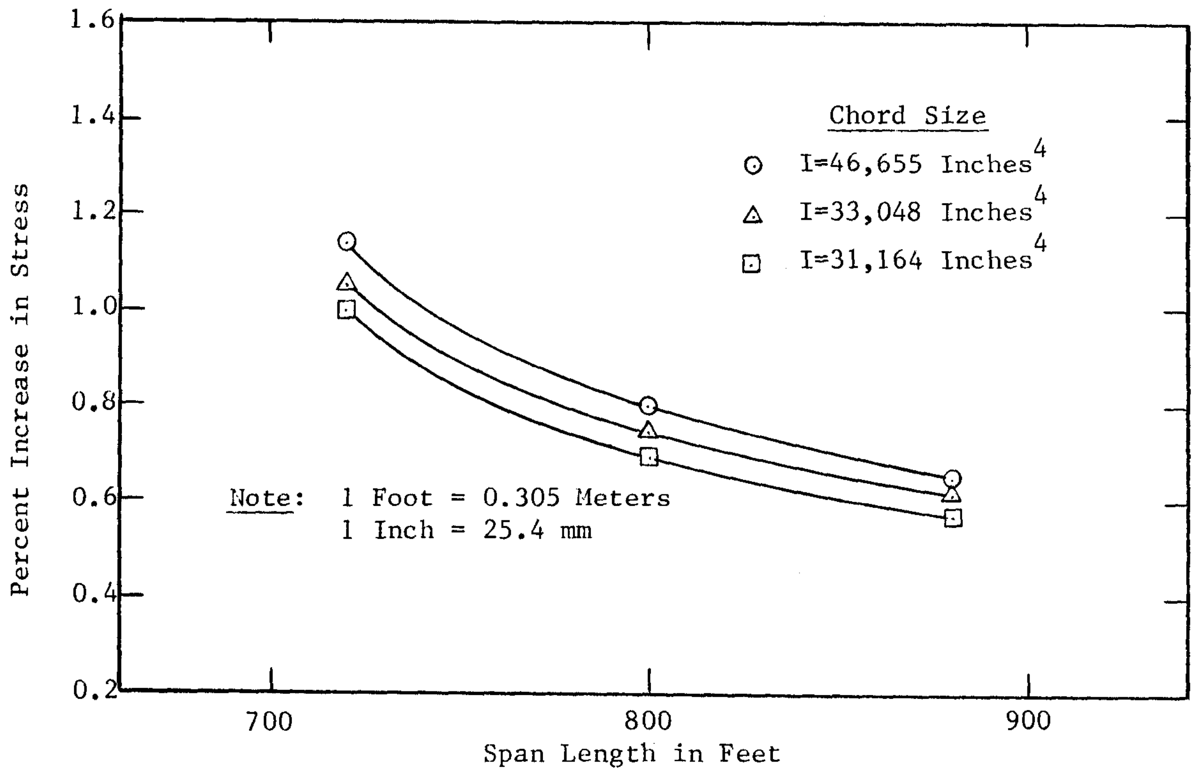


Figure 15 - Theoretical Percent Increase in Stress at Quarter Point of Two-Span Continuous Non-Parallel Chord Trusses Caused by 3 Inch Pier Settlement

4.4 Mathematical Model for the Behavior of Continuous Slab/Stringer Systems

Although the results produced by the analysis of the various steel bridge systems, as illustrated in Figures 5 through 10, 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 developed by Dean and GangaRao (22). 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 for the maximum stress increase, f_o, produced by the settlement, Δ_o, of an exterior support (abutment) and the maximum stress increase, f_α, produced by interior support (pier) settlement, Δ_α, are given as:

$$f_o(+) = \frac{2.5Ecn^3\Delta_o}{L^2} \sum_{i=n-1}^{n+1} \frac{\sin\frac{i\pi}{n} \cdot \sin\frac{2i\pi}{n}}{i^2\left(3\cot^2\frac{i\pi}{2n} + 1\right)} \tag{2}$$

$$f_o(-) = \frac{2.5E\bar{c}n^3\Delta_o}{L^2} \sum_{i=n-1}^{n+1} \frac{\sin^2\frac{i\pi}{n}}{i^2\left(3\cot^2\frac{i\pi}{2n} + 1\right)} \tag{3}$$

$$f_\alpha(+) = \frac{10Ecn^3\Delta_\alpha}{L^2} \sum_{i=n-1}^{n+1} \frac{\sin^2\frac{i\pi}{2n} \cdot \sin\frac{i\pi\alpha}{n} \cdot \sin\frac{i\pi(\alpha - 0.5)}{n}}{i^2\left(3\cot^2\frac{i\pi}{2n} + 1\right)} \tag{4}$$

$$f_\alpha(-) = \frac{10E\bar{c}n^3\Delta_\alpha}{L^2} \sum_{i=n-1}^{n+1} \frac{\sin^2\frac{i\pi}{2n} \cdot \sin\frac{i\pi\alpha}{n} \cdot \sin\frac{i\pi(\alpha + 1)}{n}}{i^2\left(3\cot^2\frac{i\pi}{2n} + 1\right)} \tag{5}$$

Where, Δ_o = differential settlement of an abutment with respect to the adjacent pier;

Δ_α = differential settlement of a pier with respect to the adjacent pier or abutment;

$f_o(+)$ = maximum increase in tension in the bottom fiber caused by Δ_o;

$f_o(-)$ = maximum increase in tension in the top fiber caused by Δ_o;

$f_\alpha(+)$ = maximum increase in tension in the bottom fiber caused by Δ_o;

$f_\alpha(-)$ = maximum increase in tension in the top fiber caused by Δ_o;

E = Young's modulus;

n = number of spans;

$L = n\ell$ = total length of bridge;

ℓ = length of spans;

c, $\bar{c}$ = distance from the neutral axis to the extreme bottom and top fibers, respectively; and

α = number of the pier (interior support) with settlement, counted in ascending order from left to right.

Equations 4 and 5 are valid for values of α corresponding to pier locations at or outside the point of symmetry of the bridge. For example, for a four-span continuous bridge, equations 4 and 5 would be valid for α = 1 and α = 2, that is, for the first interior support and the center support. Values for settlement of the third interior support would, by symmetry, be the same as those for the first interior support. However, such symmetry is not readily apparent from these equations.

Equations 2 through 5 are approximations of Fourier series solutions, and they contain small empirical correction factors to account for the neglect of additional terms. In addition, the location of the maximum positive or negative stress that is incorporated in equations 2 through 5 has been approximated from the deflected shape of the bridge superstructure.

An apparent limitation of equations 2 through 5 is that they are only valid for those continuous bridge systems that have equal span lengths and constant moments of inertia. However, this limitation usually does not lead to serious error as long as the individual span lengths of the continuous system are within 20 percent of each other (3). Furthermore, the proposed equations lead to an upper bound (conservative) solution, i.e. maximum settlement stresses, when the smallest span length of a continuous system is considered.

Typical results produced by the use of Equations 2 through 5 are presented in Figures 16 and 17. Figure 16 shows the maximum stress increase caused by settlement of the abutment of two-span continuous bridges and Figure 17 shows the maximum stress increase caused by settlement of the first interior support of four-span continuous bridges. A comparison of these results with those produced by the use of the ICES-STRUDL-II computer package (49) for the corresponding bridges showed very good agreement. Figures 16 and 17 tend to substantiate the observations made during the field studies, described above, and show that, for long span bridges, even differential settlements of 6 inches (152.4 mm) do not produce particularly large stress increases.

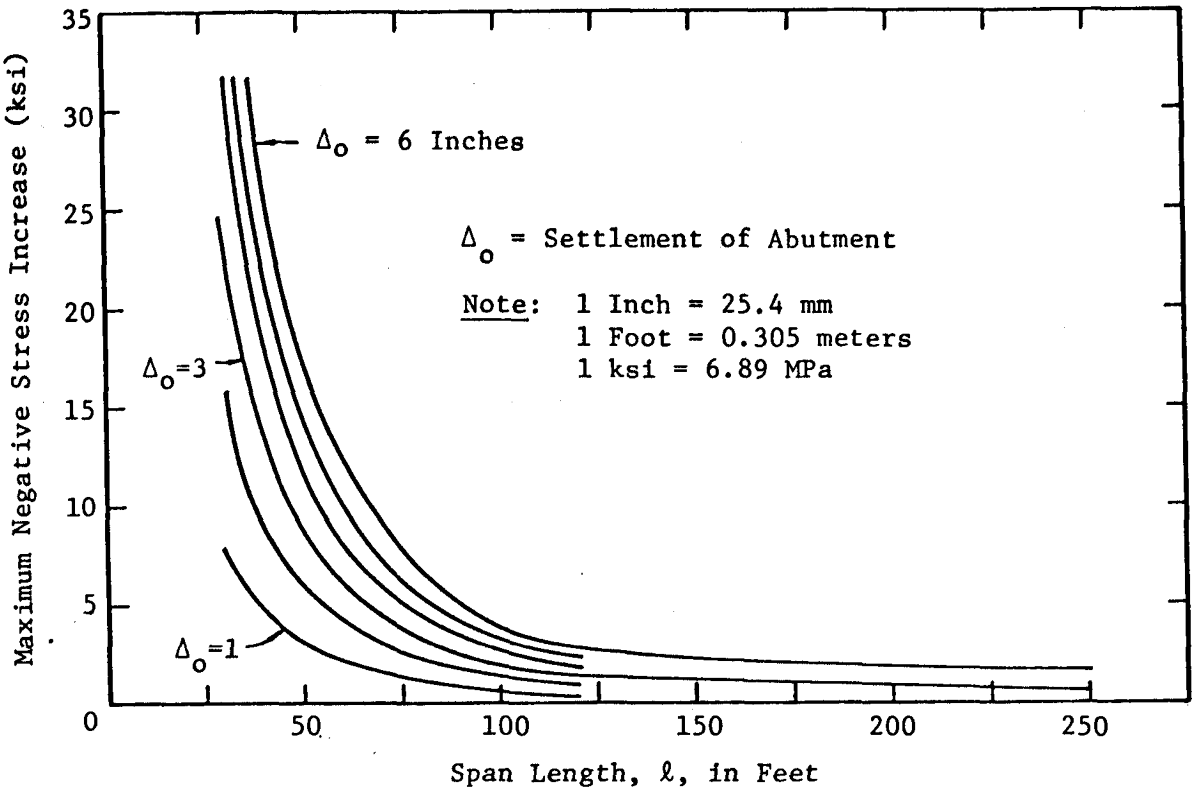


Figure 16 - Maximum Calculated Increase in Stress Caused by Settlement of Abutment of 2-Span Continuous Bridges

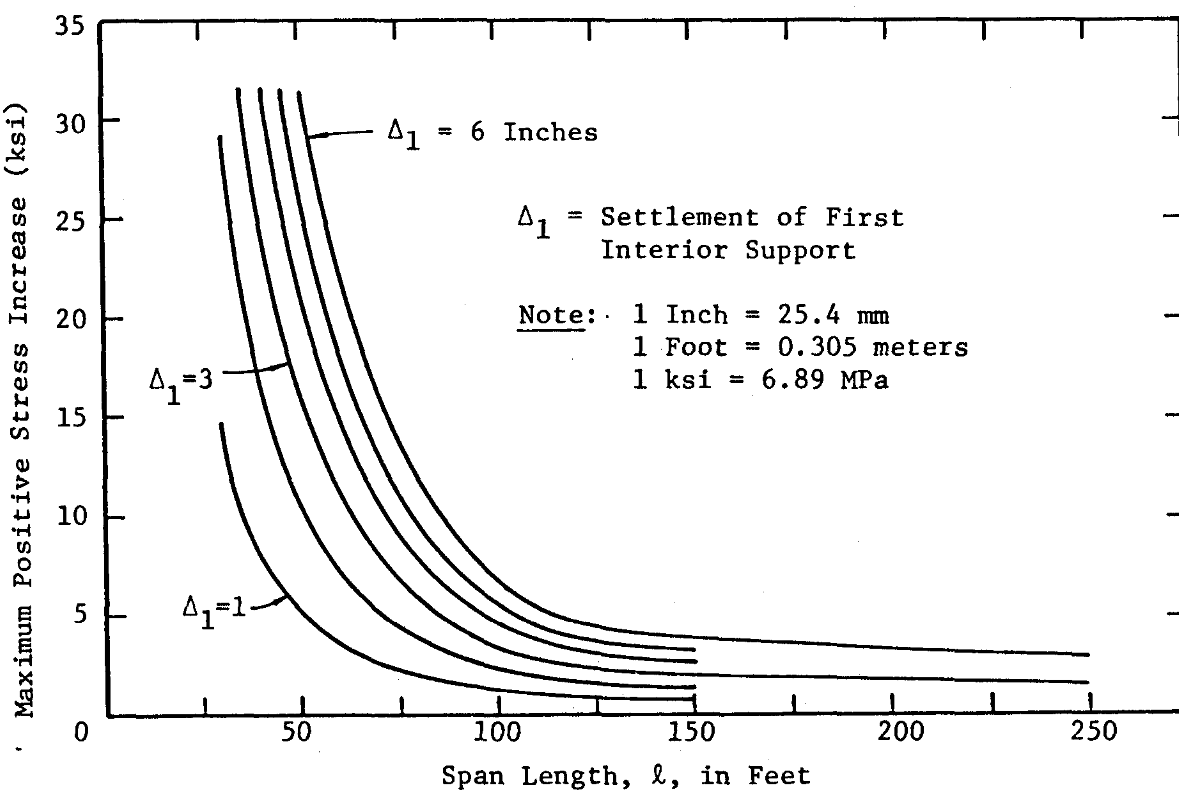


Figure 17 - Maximum Calculated Increase in Stress Caused by Settlement of the First Interior Support of 4-Span Continuous Bridges

In order to facilitate the estimation of the effect of differential settlement on continuous steel bridges, a series of six design aids were developed with the use of Equations 2 through 5, corresponding to maximum positive and negative stresses caused by differential settlement of abutments or piers. These design aids are presented in Figures 18 through 23 and provide solutions for continuous steel bridges with up to five spans and with span lengths up to 250 feet (76.2 meters).

In practice, the designer would use the appropriate design aids to pick off values of $\Delta_o c/f_o(+)$ and $\Delta_o \bar{c}/f_o(-)$, for the case of abutment settlement, or values of $\Delta_\alpha c/f_\alpha(+)$ and $\Delta_\alpha \bar{c}/f_\alpha(-)$, for the case of pier settlement. Thus, the anticipated settlement and estimated values of c and $\bar{c}$ could be used to solve for the corresponding maximum positive and negative settlement stresses.

For example, consider a two span continuous bridge with 70 foot (21.3 meter) spans, a seven inch (177.8 mm) deck slab, assuming composite action for both positive and negative moments, and a 2 inch (50.8 mm) differential settlement of one abutment. In the positive movement region, where it is assumed that the live load moment is resisted by the composite action of steel and concrete with a modular ratio of 8, a W36 x 160 beam with a 10 inch x 1 inch (254 mm x 25.4 mm) bottom cover plate was chosen to resist the positive moment. In this region, the effect of the differential settlement of the abutment is a net reduction (decrease) in the positive bending moment and, thus, in the maximum positive stress. However, in the negative moment region, where the design resulted in the use of 10 inch x 1 inch (254 mm x 25.4 mm) cover plates both top and bottom, the differential settlement of the abutment would produce an increase in the maximum negative stress. This can be evaluated by entering Figure 19 with $\ell = 70$ and $n = 2$, giving $\Delta_o \bar{c}/f_o(-) = 17.0$. Thus, for an abutment settlement of 2 inches (50.8 mm) and a value of $\bar{c} = 17.55$ inches (445.8 mm), it is found that the maximum negative settlement stress is $f_o(-) = 2(17.55)/17.0 = 2.06$ ksi (14.19 MPa).

It is proposed that settlement stresses computed in this way would be used to establish the tolerance of a bridge structure to foundation movements as described in Section 6.2.

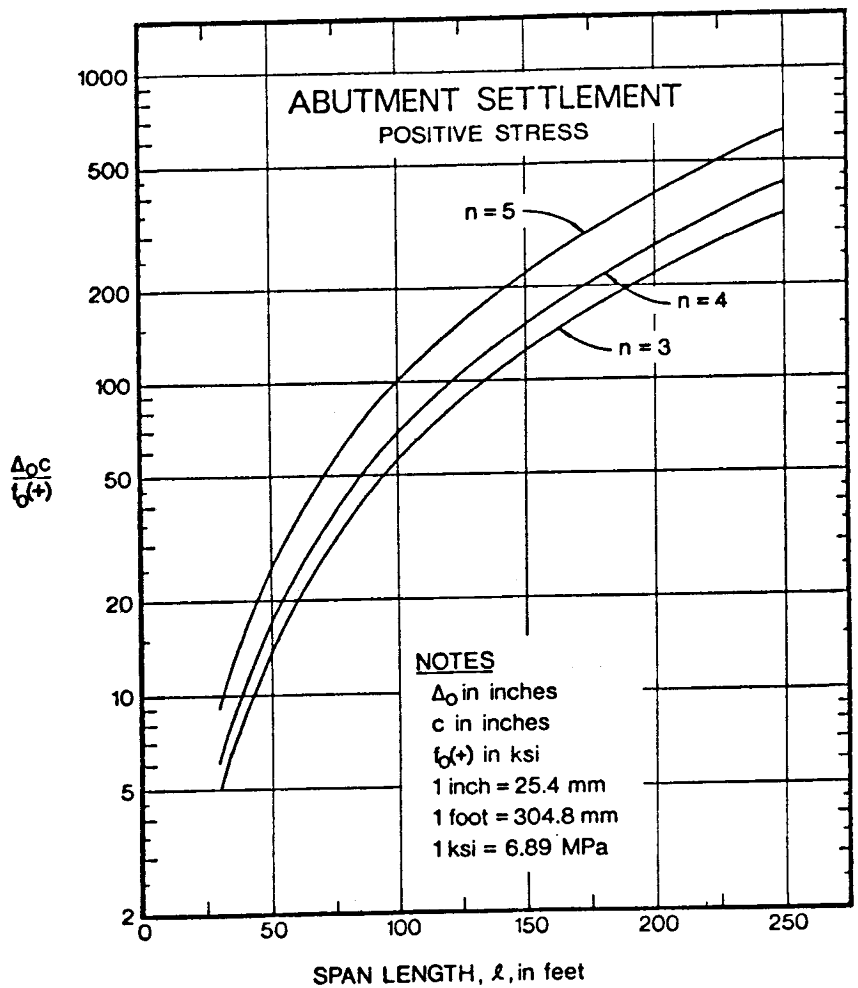


Figure 18 - Design Aid for Determining the Maximum Positive Stress Increase Caused by Differential Settlement of Abutment.

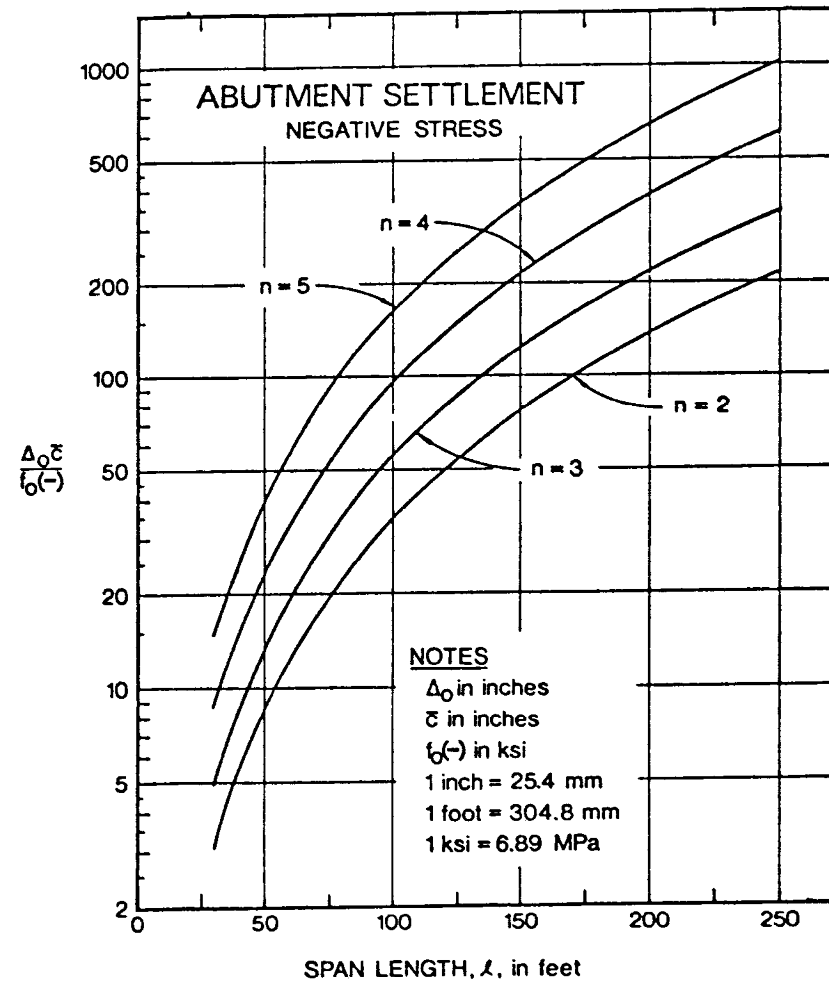


Figure 19 - Design Aid for Determining the Maximum Negative Stress Increase Caused by Differential Settlement of Abutment.

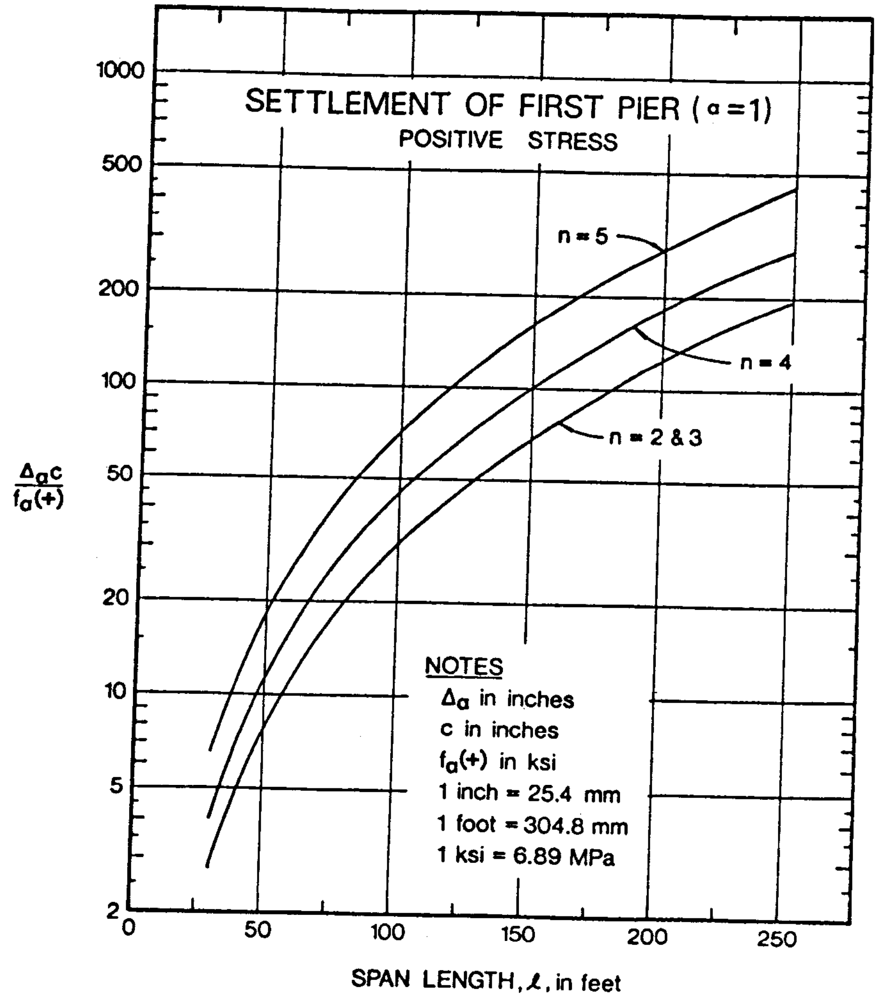


Figure 20 - Design Aid for Determining the Maximum Positive Stress Increase Caused by Differential Settlement of the First Interior Support.

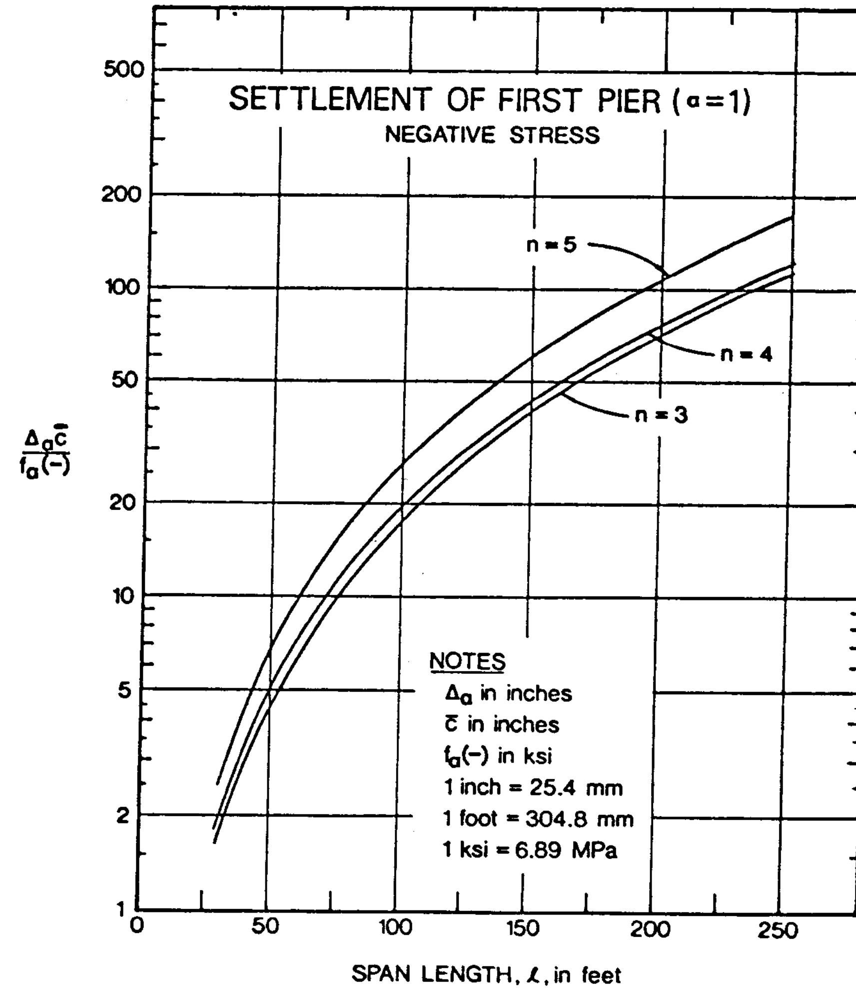


Figure 21 - Design Aid for Determining the Maximum Negative Stress Increase Caused by Differential Settlement of the First Interior Support.

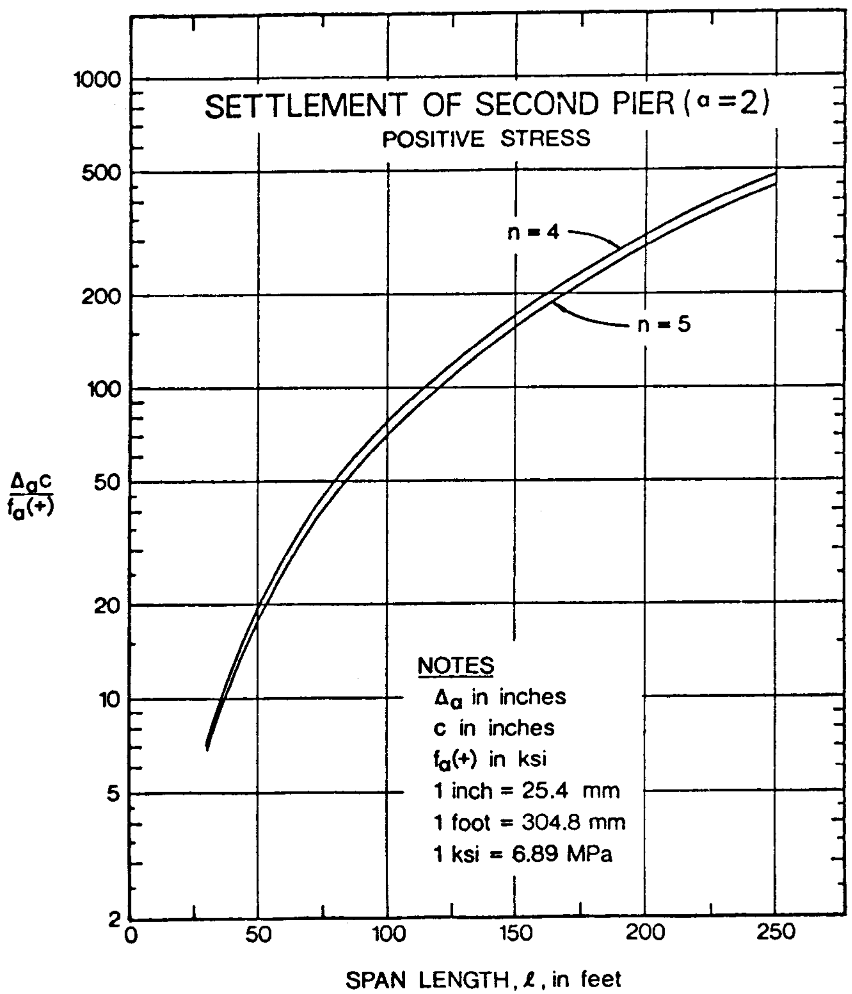


Figure 22 - Design Aid for Determining the Maximum Positive Stress Increase Caused by Differential Settlement of the Second Interior Support.

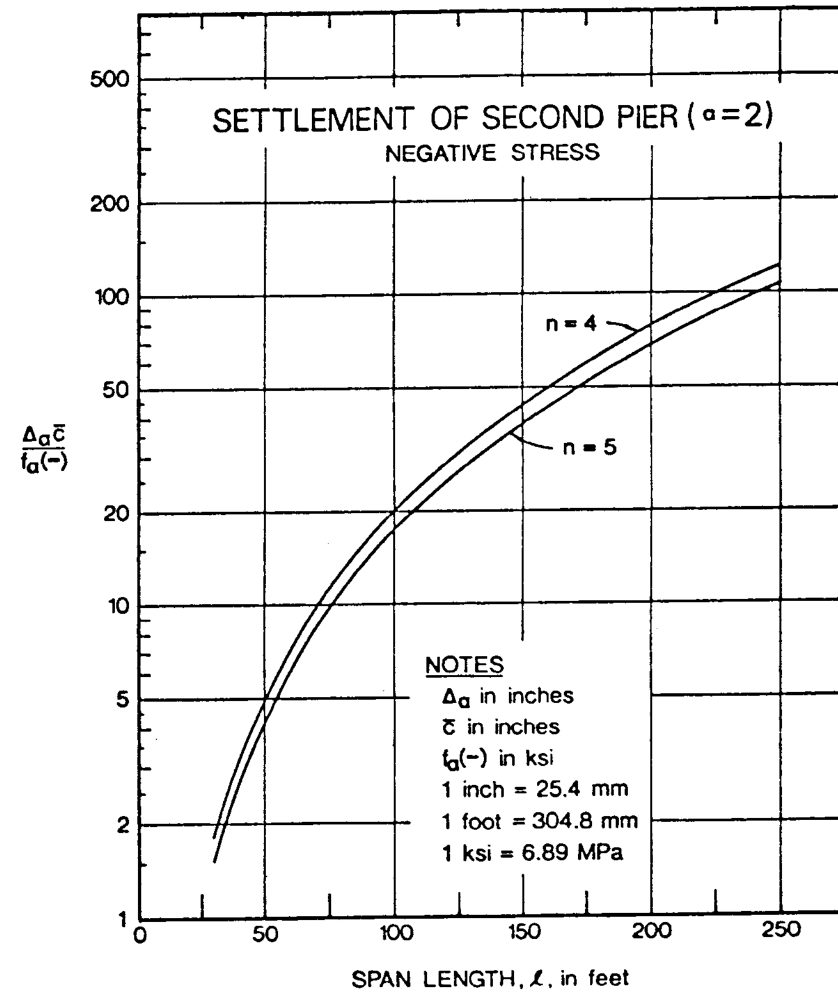


Figure 23 - Design Aid for Determining the Maximum Negative Stress Increase Caused by Differential Settlement of the Second Interior Support.

5. ANALYTICAL STUDIES OF 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. It is now 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, structural configuration, sequences 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 of the curing environment. Recommendations by an American Concrete Institute (ACI) committee and the Comite European du Beton (CEB) (2,18,19) provide convenient methods to account for these factors in prescribing a creep vs. time relationship for a particular situation.

Considerations of structural configurations 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 just 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, i.e. 63 inches (1.6 meters) as compared to 54 inches (1.37 meters). 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.

Sequences of construction are 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 construction sequences: (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 possibilities of creep relief of settlement-induced stress.

Each of these aspects of creep material properties, structural configuration, and construction sequencing could be accounted for by using a sophisticated time-incremental solution, such as used by Tadros (70) and Nikjeh (62). This procedure is very expensive to implement, because of the computer time required to analyze any particular case. It rapidly becomes infeasible when the number of cases for a meaningful parameter study is large. To account for the complexities just described, a full parametric study would require many tens of thousands, if not hundreds of thousands, of individual analyses. For any smaller class of problems, suitable solutions, although more approximate in nature, can be found and implemented by hand or on the computer.

The research reported in this section deals with the analysis of the effect of differential support settlements on composite and non-composite two-span continuous AASHTO-PCI standard I-girder bridges and two and four-span continous box girder bridges. As mentioned earlier in this section, the perception of this research changed significantly during the course of the investigation. Consequently, some elements of the proposed investigation were modified significantly. The details of these modifications are reported with the particular research.

5.1 Methods of Analysis

As mentioned earlier, the analysis of reinforced concrete and prestressed concrete structures to account for effects of support settlement can be a complex process. The basic reason for this complexity is the time-dependent variation of concrete material properties due to continued curing, shrinkage and creep. These material properties are related to the properties of the concrete mix constituents (such as the cement and aggregates), the concrete mix proportions (reflected by cement content, water-cement ratio, aggregate content and gradation) and to the environmental and loading history of the structure (curing history, relative humidity, age at first loading, and nature of the sustained loading on the structure). There will also be interactions between the concrete materials and the steel reinforcement. For example, mild steel restrains shrinkage and inhibits creep, while the presence of prestressed reinforcement may increase creep and is itself subject to additional time-dependent losses. Clearly, analytical procedures must account for material properties, which vary with time, and the interactions of the concrete with the steel reinforcement.

The creep behavior of the concrete structures may be of particular importance in mitigating effects of foundation settlements. For example, consider a simple two-span continuous beam as shown in Figure 24. If the center support settles vertically, curvatures and reactions will develop as shown. The magnitudes of the support reactions will be proportional to the settlement which occurs. If the beam is made of a "noncreeping" material such as structural steel (at normal temperatures), the reactions must have a time history with the same shape as that for the settlement. Thus, if the structure of Figure 24a is a steel beam subjected to a foundation movement with possible time-settlement relationships as shown in Figure 24b, the variation of the reaction caused by settlement with time will correspond to the curves labelled P_{1S} and P_{2S} in Figure 24c. On the other hand, if the material is likely to creep with time, the stresses will be redistributed within the structure, maintaining equilibrium and compatibility. Possible variations of reaction with time for a creeping material are shown in Figure 24c as curves P_{1C} and P_{2C} for the cases of instantaneous and gradual settlement, respectively. In the course of this investigation, Nikjeh (62) determined that creep may reduce settlement stresses to one-third of the values resulting from a theoretical instantaneous settlement. Because of the interactions of creep and settlement rate, analytical methods must also account for settlement rate, although the case of instantaneous settlement may provide useful comparisons.

To develop time-dependent constitutive relationships, techniques must be available to estimate concrete shrinkage or creep, based on a knowledge of concrete mix parameters and environmental factors. Recommendations for estimating creep and shrinkage are found in reports of ACI Committee 209 (2), and a joint CEB-FIP Committee (19). On the basis of further research, this latter report was revised and incorporated in the CEB Model Code for Structures (18). Since these predictive recommendations are based, in part, on experimental studies, it should be recognized that their use will lead to nominal, or mean values, and that some errors may result.

As a further illustration of the effects of various factors on creep, the ACI Committee 209 (2) standard creep equation will be presented. In this case, the variation of the creep coefficient, ν, with time is given as:

$$\nu = \frac{t^{0.6}}{10 + t^{0.6}} \nu_u, \tag{6}$$

where t is the time after loading, in days, and ν_u is the ultimate creep coefficient. The coefficient ν_u may be defined as:

$$\nu_u = 2.35\alpha_{LA}\alpha_H\alpha_T\alpha_S\alpha_F\alpha_A, \tag{7}$$

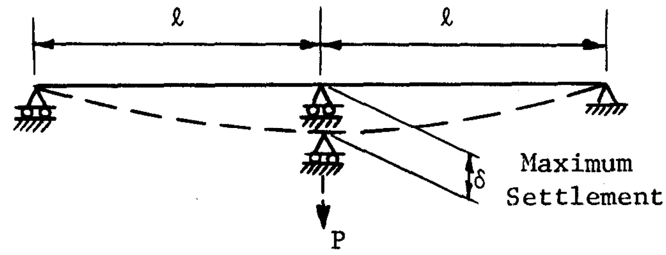
Where α_{LA} = $1.25\ t^{-0.118}$ for moist-cured concrete, and

α_{LA} = $1.13\ t^{-0.095}$ for stream-cured concrete, where t is the age at loading in days;

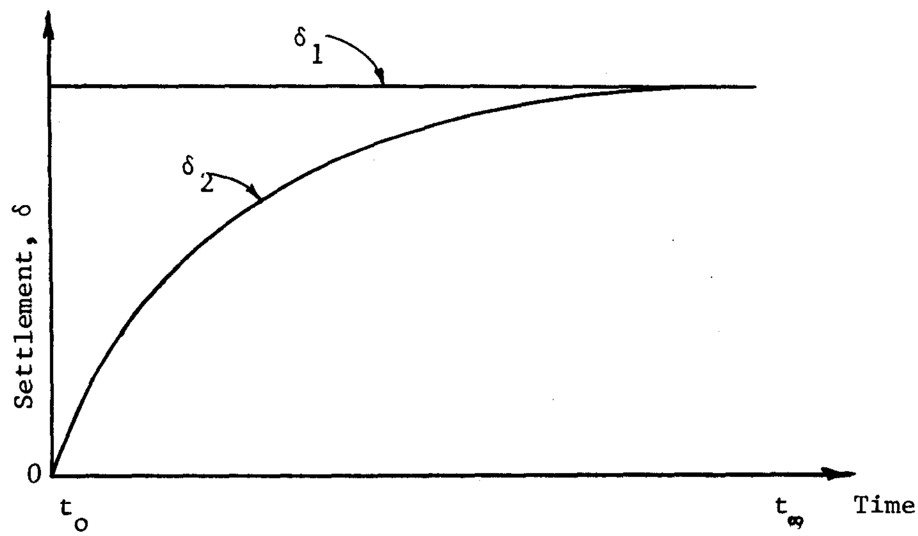
α_H = 1.27 - 0.0067H, where H is the ambient relative humidity in percent (α_H = 1.0 when H is less than 40);

α_T = 1.10 - 0.017 T, where T is the minimum thickness of the cross section in inches;

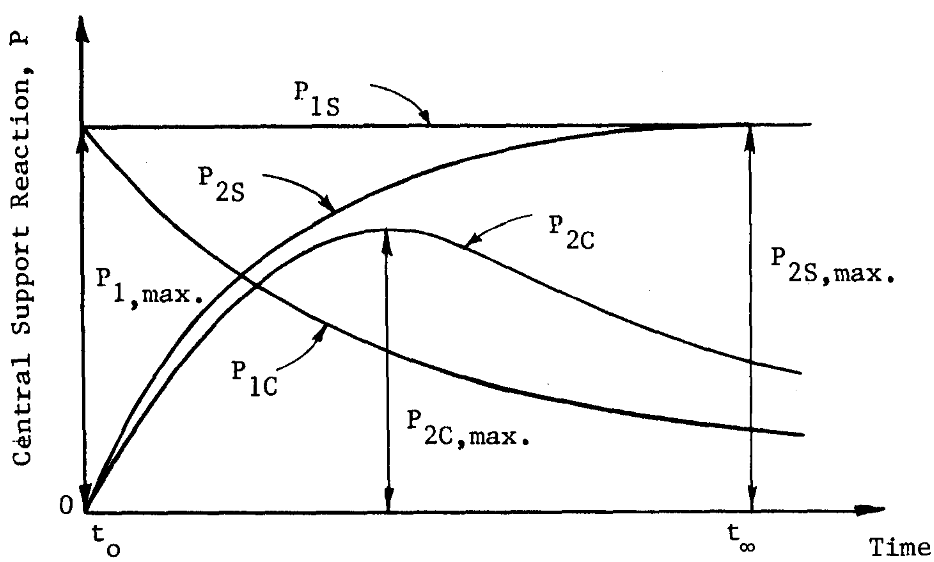
α_S = 0.82 + 0.67 S, where S is the concrete slump in inches;



(a) Two-Span Continuous Beam



(b) Time-Settlement Relation



(c) Support Reaction for Creeping Material "C", e.g. Concrete, and Noncreeping Material "S", e.g. Steel

Figure 24 - Time-Rate Effect on Settlement Induced Forces

α_F = 0.88 + 0.0024 F, where F is the amount of fine aggregate, as a percentage of the total amount of aggregate; and

α_A = 0.46 + 0.09 A, where A is the air content in percent (α_A = 1.0 for A less than 6.0 percent) .

The simplest time-dependent constitutive relationship is developed for the conditions of constant compressive stress. Let a constant compressive stress, f, be applied at age t_o, and held constant until some later time t_1. If the stress, f, is lower than 40 to 50 percent of the concrete compressive strength, creep strains will be proportional to the applied stress, and the principle of linear superposition will apply. At the time of the loading, t_o, an instantaneous strain, ε_e, occurs and the creep strain, $\varepsilon(t_1)$, at any later time can be written in terms of the applied stress, modulus of elasticity and creep and shrinkage properties, as follows:

$$\varepsilon(t_1) = \frac{f(t_1)}{E_c(t_o)} [1 + \nu(t_1,t_o)] + \varepsilon_{sh}(t_1,t_o), \qquad (8)$$

where E_c (t_o) is the modulus of elasticity at time t_o; ν (t_1, t_o) is a creep coefficient, depending on t_o and $(t_1 - t_o)$; and, ε_{sh} (t_1, t_o) is the shrinkage strain, depending on t_o and $(t_1 - t_o)$.

If the stress is a function of time, f(t), an infinitesimal stress increment, df, will produce instantaneous strain, $df/E_c(t)$, and cause a creep strain, $\frac{df(t)}{E_c(t)} \nu$ (t_1, t), during the period $(t_1 - t)$.

The total strain, $\varepsilon(t_1, t)$ is therefore:

$$\varepsilon(t_1) = \int_{t = t_o}^{t_1} \frac{df(t)}{E_c(t)} [1 + \nu(t_1,t_o)] + \varepsilon_{sh}(t_1,t_o) \qquad (9)$$

For the case of a time-variant stress, several approximate methods of analysis have been proposed, with various simplifications. These methods include the effective modulus method, the rate of creep method, the rate of flow method, and the relaxation method. In a review of these approximate methods, Tadros (70) concluded that the relaxation method is the most accurate of the simple methods. In the following discussion, some background will be provided for use of the relaxation method, which is suitable for hand or computer calculations, and a general step-by-step method, which rquires use of a digital computer. For both of these methods, it is assumed that Hooke's Law is valid, so that instantaneous and creep strains are linearly proportional to the applied stress, and Bernoulli's assumption applies (in a flexural member, plane sections remain plane during and after deformation). Each of these methods was employed in the analysis of the various concrete bridges where its use was felt to be most advantageous.

5.1.1 Relaxation Method

This method was originated by Trost (73) and further developed by Bazant (10). The basis of this method is that the creep strain at time, t_1, is greater for applied stress, f, held constant since time, t_o, than would be expected if the stress increased monotonically from zero at the time, t_o, to a stress of f, at time, t. Mathematically, this reduction can be modelled by introducing a term, η, the relaxation- or aging-coefficient, which reduces strains from those computed from the assumption of constant stress. This coefficient is a function of the creep coefficient, age of concrete at loading, and to a lesser degree, loading duration. For concretes loaded at an age of 20 to 30 days, and having creep coefficients in the range of 1.5 to 2.5, η does not vary appreciably, and a mean value of 0.85 may be used. Values of the relaxation coefficient as determined by Dilger and Neville (23) are shown in Figure 25.

To illustrate the application of the relaxation method, once again consider the two-span continuous beam shown in Figure 26a. A sudden settlement, δ, of the central support would cause an instantaneous moment, $M_{e\ell}$. The magnitude of $M_{e\ell}$ can be computed to be $3EI\delta/\ell^2$, by application of the flexibility method. In Figure 26b, the determinate primary structure is shown to have a pin over the central support. When the central support settles an amount, δ, a relative rotation $\theta_{e\ell} = 2\delta/\ell$ occurs at the pin. To achieve compatibility, this rotation must be overcome by the rotation due to $M_{e\ell}$, i.e. $M_{e\ell}\ \theta_{BB,e\ell}$ (Figure 26c), where $\theta_{BB,e\ell} = 2\ell/3EI$, is the total angle change at B which results from a unit redundant moment, $2\ell/3EI$.

The creep rotation that would occur at any time, due to $M_{e\ell}$, is given by $\overline{\theta}(t) = \nu\theta_{e\ell}$ (Figure 26d). To restore compatibility at support B, a rotation $\overline{M}\ (t)\ \theta_{BB}(t)$ is required to oppose the creep rotation (Figure 26e). $\overline{M}(t)$ is the change in moment at the support, while $\theta_{BB}(t)$ is the rotation which would occur at any time, due to a moment increasing from zero at time zero to a unit value at time infinity. By the relaxation method, $\theta_{BB}(t) = \theta_{BB,e\ell}\ (1+\eta\nu)$. Consequently,

$$\overline{M}(t) = -\frac{\theta_{e\ell}\nu}{\theta_{BB,e\ell}(1 + \eta\nu)}\ , \tag{11}$$

or

$$\overline{M}(t) = -\frac{M_{e\ell}\nu}{1 + \eta\nu} \tag{12}$$

To find the total moment, M(t), at any time, the instantaneous and time-dependent moments are superimposed as follows:

$$\begin{aligned} M(t) &= M_{e\ell} + \overline{M}(t) \\ &= M_{e\ell} - \frac{M_{e\ell}\nu}{1 + \eta\nu} \\ &= M_{e\ell}(1 - \frac{\nu}{1 + \eta\nu}) \end{aligned} \tag{13}$$

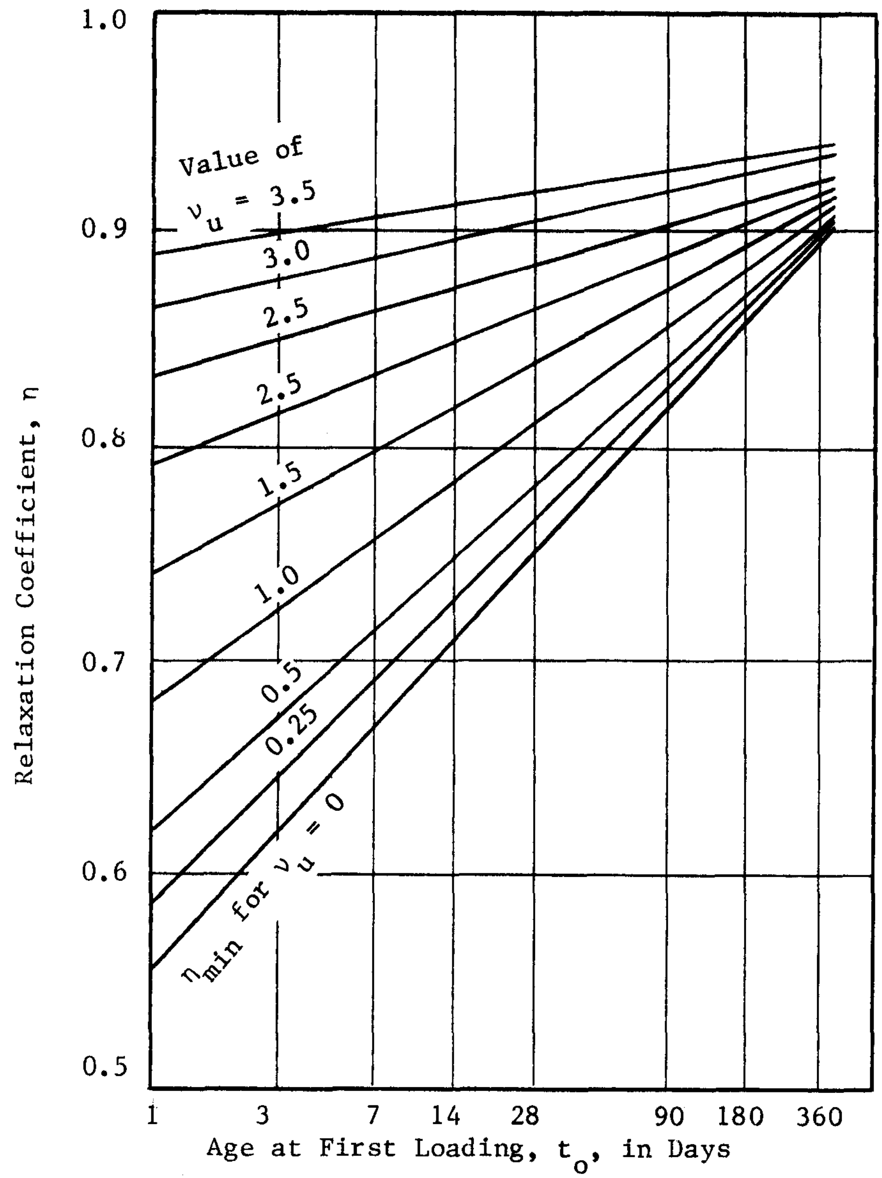
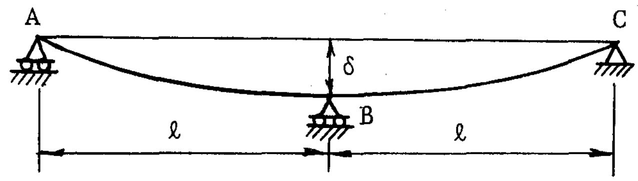
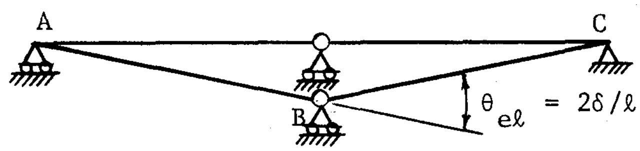


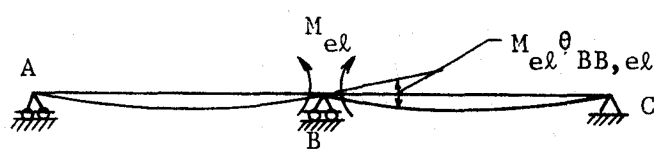
Figure 25 - Relaxation Coefficient, η, as a Function of the Age of First Loading, t_o, for Different Values of Ultimate Creep Coefficient, ν_u(23)



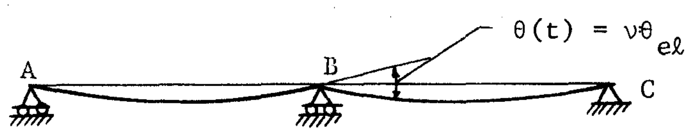
(a) Settlement of Indeterminate Beam



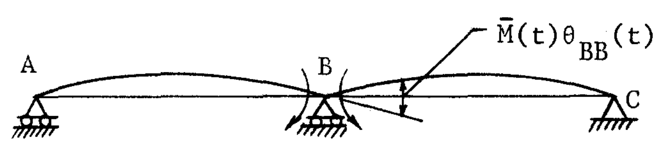
(b) Rotation Caused by Settlement of Statically Determinate (Released) Beam



(c) Rotation Caused by Instantaneous Moment



(d) Rotation Caused by Creep



(e) Rotation Caused by $\bar{M}(t)$

Figure 26 - Analysis of Continuous Beam by the Relaxation Method - Sudden Settlement

A similar analysis may be followed for the case of a gradual support settlement. The time-dependent rotation due to the moment, $M_{e\ell}$, will be $\theta_{e\ell}\eta\nu$, accounting for the gradual application of the stresses. The compatibility equation becomes:

$$\theta_{e\ell}\eta\nu + \overline{M}(t)\theta_{BB}(t) = 0, \tag{14}$$

where $\theta_{BB}(t) = \theta_{BB,e\ell}(1 + \eta\nu)$.

Thus,
$$\overline{M}(t) = M(t) = \frac{\theta_{e\ell}\eta\nu}{\theta_{BB,e\ell}(1 + \eta\nu)} = M_{e\ell}\,\frac{\eta\nu}{1 + \eta\nu} \tag{15}$$

5.1.2 Step-by-step Method

This technique is practical only when implemented by a computer program, since it is based on dividing time into discrete intervals, and a large amount of information must be updated continually. Stresses and deformations at the end of each time interval are calculated in terms of the stress applied in the first interval and the stress increments occurring in successive intervals. Stress variation in any time interval is assumed to occur at its middle. For consistency, instantaneous applied loads such as self-weight, prestressing, and sudden settlement are assumed to occur at the middle of a time interval of zero length. The total concrete strain, instantaneous plus creep and shrinkage, at the end of any interval, i, is:

$$\varepsilon_c(i + 1/2,\ 0) = \sum_{j=1}^{i} \frac{\Delta f_c(j)}{E_c(i)}\,[1 + \nu(i + 1/2,j)] + \varepsilon_{sh}(i + 1/2,0), \tag{16}$$

where i, j refer to the times at the middle of intervals i and j,

i + 1/2 refers to the time at the end of interval i,

0 = time at the beginning of the first interval,

$\Delta f_c(j)$ = concrete stress increment introduced at the middle of interval j,

$E_c(j)$ = the modulus of elasticity of concrete at the middle of interval j,

$\varepsilon(i+1/2,0)$ = the free shrinkage strain in concrete at the end of the i^{th} interval, and

$\nu(i+1/2,j)$ = the creep coefficient, reflecting the creep strain at the end of interval i, due to a stress introduced at the middle of interval j.

Equation 16 may be formulated in terms of member axial strains and curvatures, resulting in a general method of analysis, which can accomodate various relationships of creep and shrinkage with time, loads occurring over many stages and at different concrete ages, and composite action of materials with different creep, shrinkage and elastic properties.

5.1.3 Time-Dependent Settlements

For the purposes of this investigation, it was necessary to establish a range of time-settlement relationships which might represent practical foundation behavior for highway bridges. A total of five different soil types were selected for possible study. The time-settlement relationships for these soils are shown in Table 33, where soil type 'A' might represent a typical fill material, and type 'E' a very cohesive clay. In each case, the ultimate settlement is 3 inches (76.2 mm). The nature of some analyses used in this investigation required that gradual settlements be treated as several equivalent sudden settlements. To achieve this equivalence, the time-settlement relationships of Table 33 were approximated with step functions, using three of more "steps" to attain the ultimate settlement.

5.2 Analysis of AASHTO-PCI Standard I-Girder Bridges

Concrete highway bridges with AASHTO-PCI standard I-girders are very common, being found frequently on interstate-quality highways. Behavior of this type of bridge with respect to foundation settlement is very important information in considering the overall picture of tolerable foundation movements for highway bridges. This section reports the results of the analysis of several I-type cross-sections with spans of 75, 100, and 125 feet (22.9, 30.5 and 38.1 meters). The analyses include time rate effects of settlement, as well as effects of creep and shrinkage. Although the major interest of this section is a study of precast girders made continuous and composite for live loads, some other types of construction are included to illustrate specific effects. Analyses were made using relaxation principles, and the step-by-step computer method. In all cases, results assume that an uncracked gross concrete section is maintained.

5.2.1 Analysis of Continuous I-Girder Bridges

The analysis of a two-span continuous I-type girder provides 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 a relaxation analysis and 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 increase as 1:2:6 and the relative section depths as 1:1.2:1.6.

Table 34 reports time-dependent moments and stresses in these continuous I-girder bridges for both sudden and gradual settlement. In this case, a settlement relation of the form

Table 33 - Time-Settlement Relationships for Various Soil Types

Settlement in Inches (1)	Time Required for Settlement, in Days, for Given Soil Type				
	A (2)	B (3)	C (4)	D (5)	E (6)
0.0	0	0	0	0	0
0.3	0.33	1.7	3.3	6.6	16.6
0.6	1.4	6.8	13.5	27.0	67.6
0.9	3.0	15.2	30.4	60.8	152.1
1.2	5.4	27.1	54.2	108.4	271.2
1.5	8.4	42.2	84.4	168.8	421.8
1.8	12.3	61.6	123.1	246.2	615.5
2.1	17.3	86.7	173.4	346.8	867.3
2.4	24.4	121.9	244.0	488.0	1219.2
2.7	36.5	182.5	364.9	729.8	1824.8
2.85	48.6	243.0	485.9	971.8	2429.0

Note: 1 inch = 25.4 mm

Table 34.-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) (1)	Location of Moments and Stresses (2)	Settlement Rate (3)	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 (4)	180 Days (5)	1800 Days (6)	Top (7)	Bottom (8)	Top (9)	Bottom (10)	Top (11)	Bottom (12)
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	-829	-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.

$$\delta(t) = \frac{t^{0.6}}{10 + t^{0.6}} \cdot \delta_{ult.}$$

was used. This settlement relationship is somewhat different than the rates shown previously in Table 33. Shortly after the settlement begins the relationship behaves more like rate B or C, in Table 33, then the rate of settlement drops off, and it becomes more like soil types D or E.

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

For the 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, an artifact of the particular choice of girder section.

5.2.2 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 35.

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.

5.2.3 Girder Composite with Cast-in-Place Deck

In the analyses reported in this section, the step-by-step computer

Table 35.--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) (1)	Location of Moments and Stresses (2)	Settlement Rate (3)	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 (4)	180 Days (5)	1800 Days (6)	Zero Days		180 Days		1800 Days	
						Top (7)	Bottom (8)	Top (9)	Bottom (10)	Top (11)	Bottom (12)
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

method was used to analyze the composite section made 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 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 36. 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 the cast-in-place bridges of Section 5.3.1. Creep and shrinkage reduce the effects of settlement. Time-dependent stresses at midspan and the central support of the 100 foot (30.5 meter) span bridge are shown in Figures 27 and 28, respectively.

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 the time-settlement variation of soil type E in Table 33. Equal increments of 1 inch (25.4 mm) were applied at 93 days, 453 days and 1553 days. Time-dependent stresses for gradual settlement are shown in Figures 29 and 30 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 ultimately be higher than those caused by sudden settlement.**

5.2.4 Composite Section with Prestressing

To supplement the studies of section 5.2.3, 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. A relaxation analysis was performed, assuming girder and deck to have identical properties and that the settlement occurred just after continuity was imposed.

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 Table 37. 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 witin the allowable range for dead load settlement and prestresses, but live load will cause the

Table 36.--Long-term Stresses in Two-Span Continuous Cast-In-Place Composite Bridges Caused by Dead Load and Settlement

Span Length in Feet (Girder Type) (1)	Assumed Settlement of Central Support (2)	Assumed Behavior with Respect to Creep and Shrinkage (3)	Stresses in ksi in Given Member at Given Location							
			At Central Support				At Mid Span			
			Slab		Girder		Slab		Girder	
			Top (4)	Bottom (5)	Top (6)	Bottom (7)	Top (8)	Bottom (9)	Top (10)	Bottom (11)
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

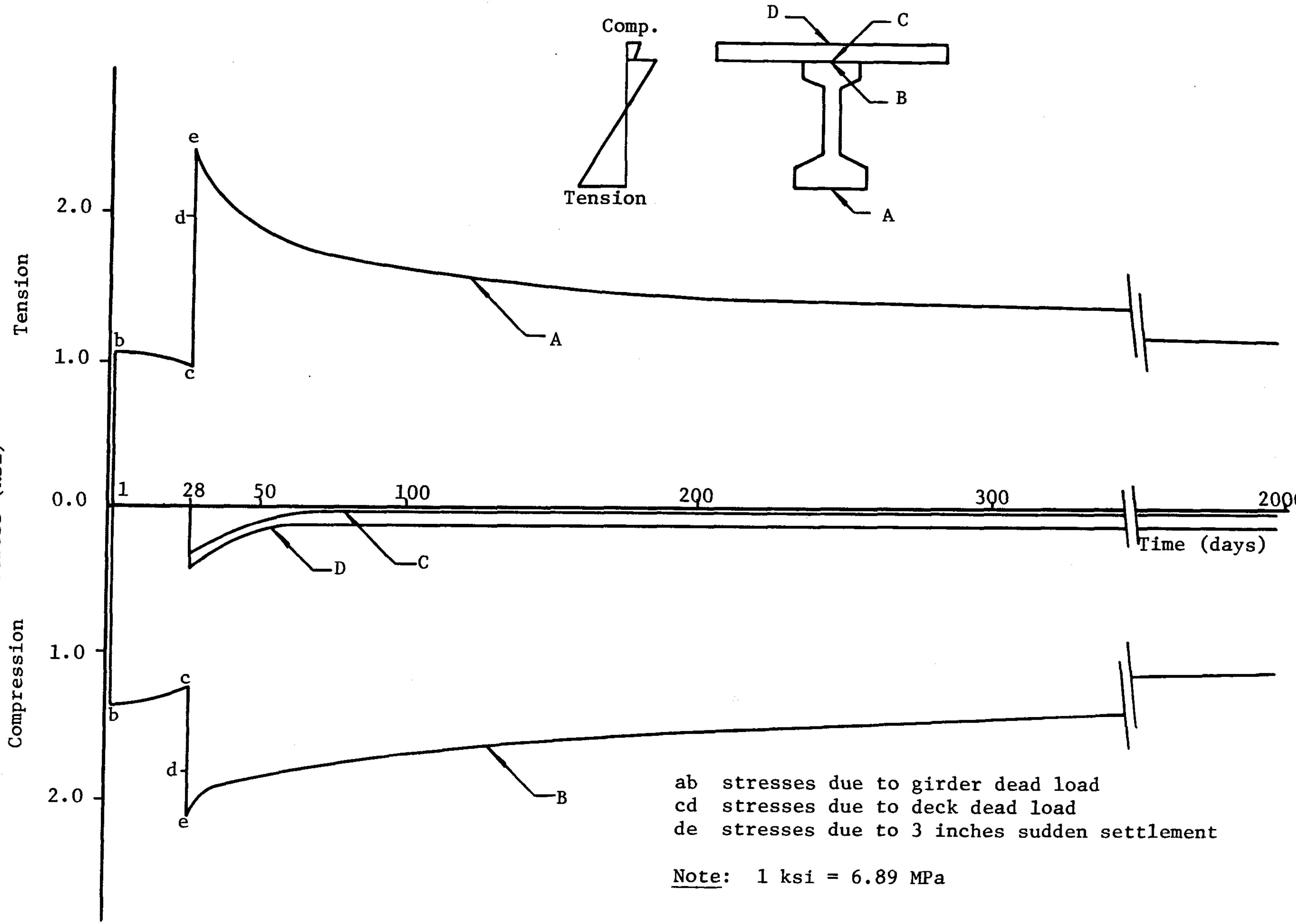


Figure 27 - Time-Dependent Variation of Stresses at Midspan for Two-Span Continuous Concrete Bridge With 100 Foot Spans - Sudden Settlement of Center Support

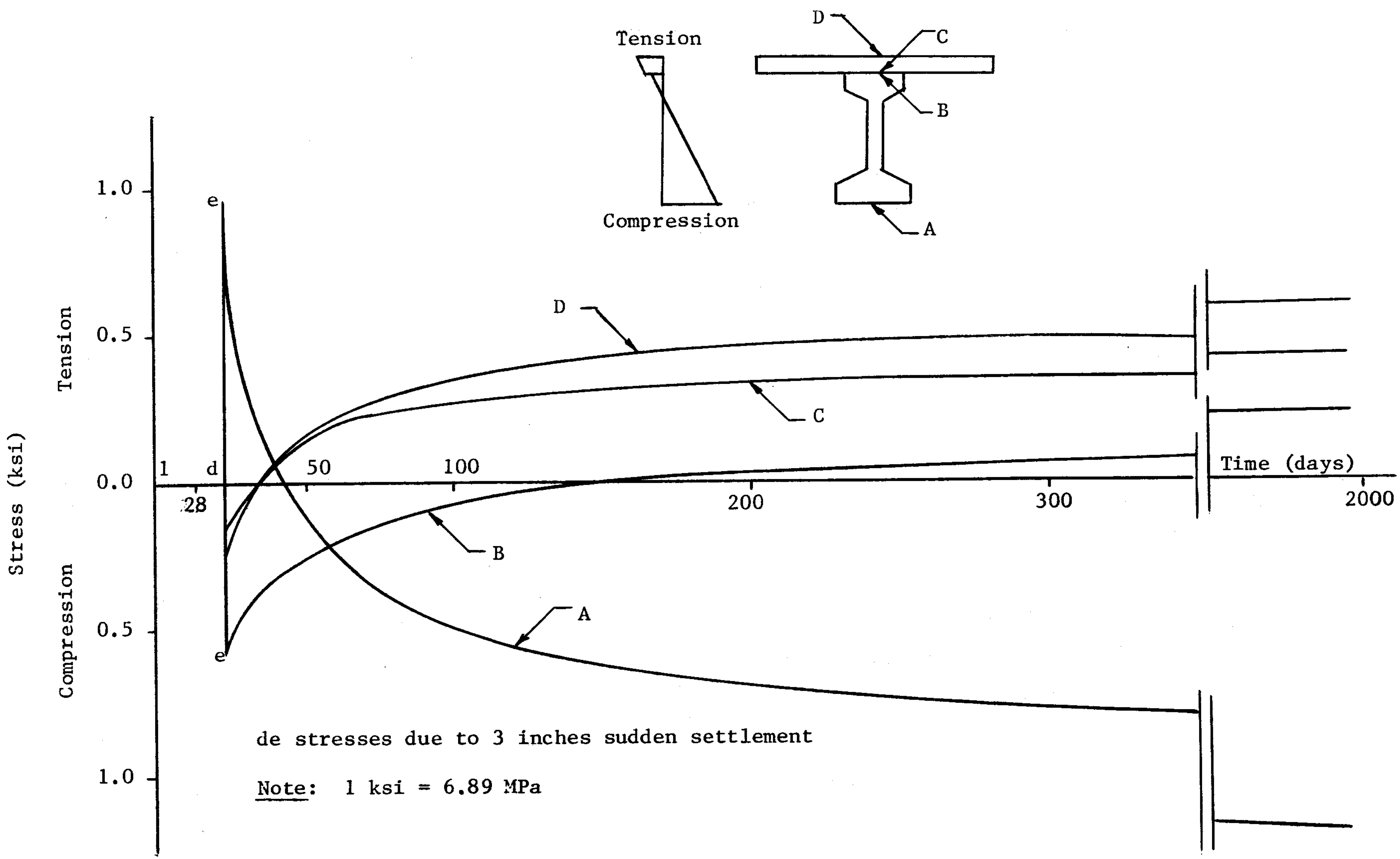


Figure 28 - Time-Dependent Variation of Stresses at Center Support for Two-Span Continuous Concrete Bridge With 100 Foot Spans - Sudden Settlement of Center Support

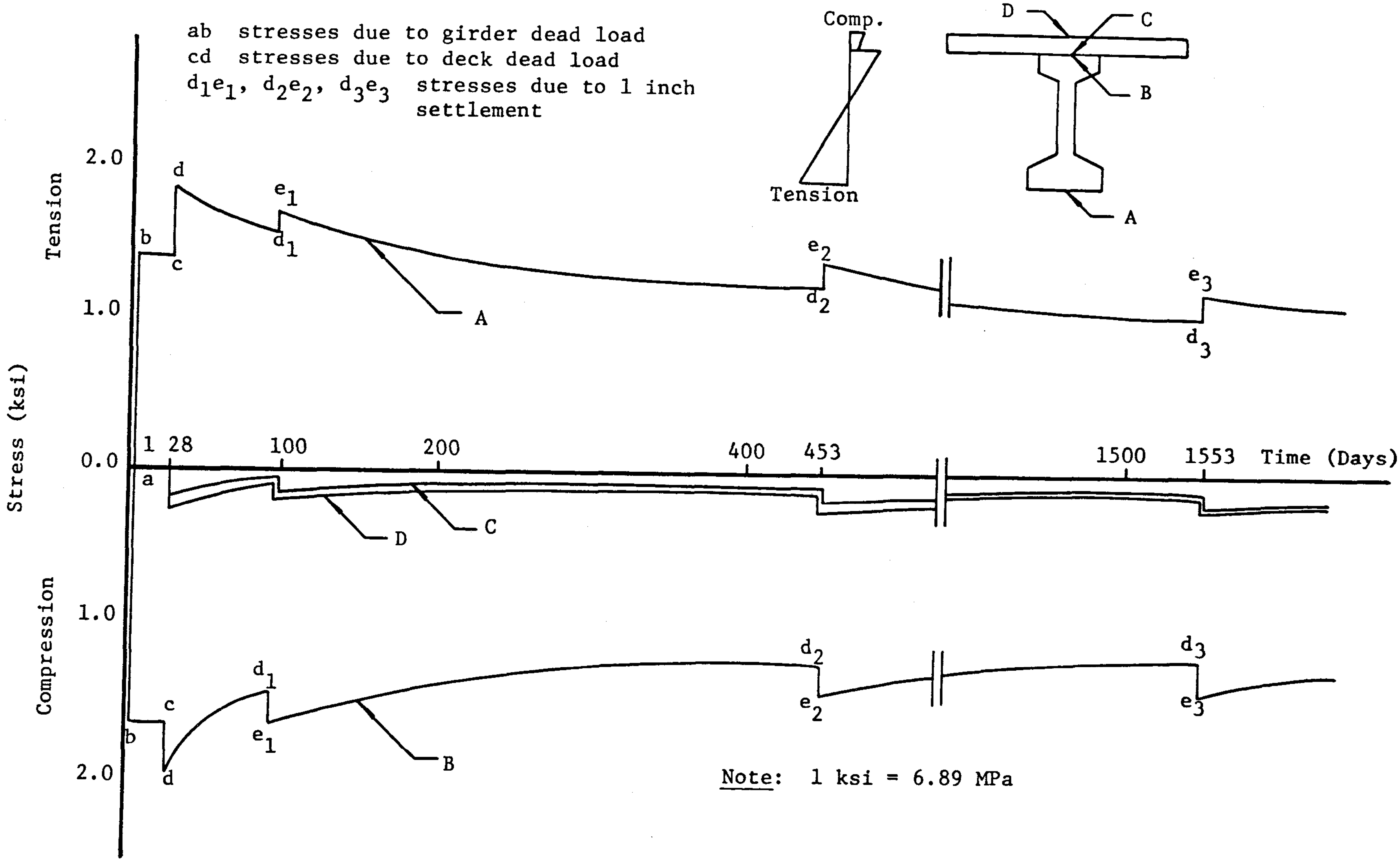
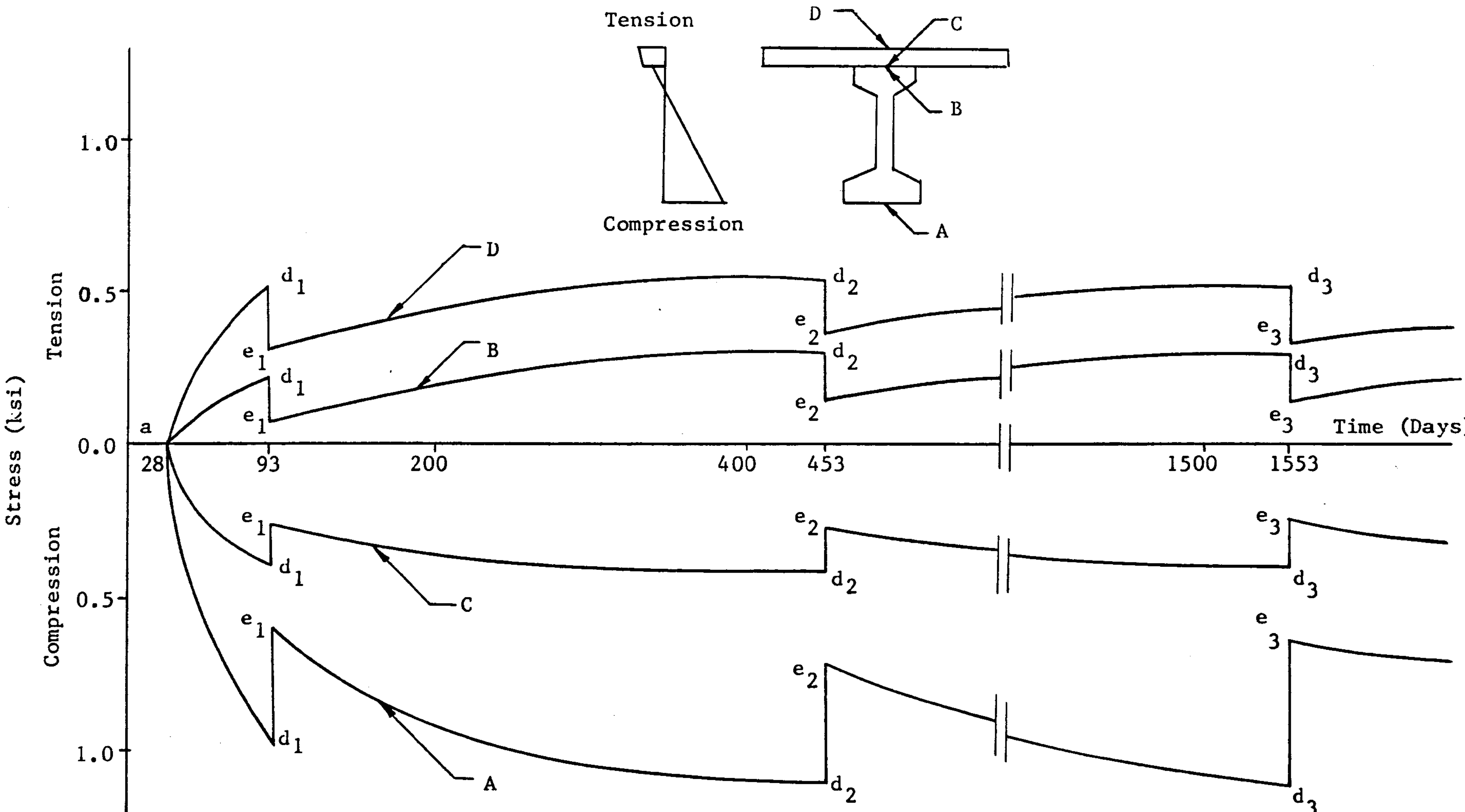


Figure 29 - Time-Dependent Variation of Stresses at Midspan for Two-Span Continuous Concrete Bridge with 100 Foot Spans - Gradual Settlement of Center Support



ad stresses due to dead loads

d_1e_1, d_2e_2, d_3e_3 stresses due to 1 inch settlement

<u>Note:</u> 1 ksi = 6.89 MPa

Figure 30 - Time-Dependent Variation of Stresses at Center Support For Continuous Concrete Bridge with 100 Foot Spans - Gradual Settlement of Center Support

Table 37.--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
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
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
125 (VI)	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

allowable compressive stress to be exceeded.

These effects should be investigated more fully, including the effects of other strand profiles, gradual settlements, and settlements occurring after the superstructure concrete is more mature.

5.2.5 Summary

This section has 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.

5.3 Analysis of 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 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 by Nikjeh (62) 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, additional 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).

5.3.1 Two-Span Box Girder Bridges

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 using the step-by-step analysis procedure described in Section 5.1.2 implemented 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 31 and 32 for the bridge with 100 foot (30.5 meter) spans, and Figures 33 and 34 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 32. 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

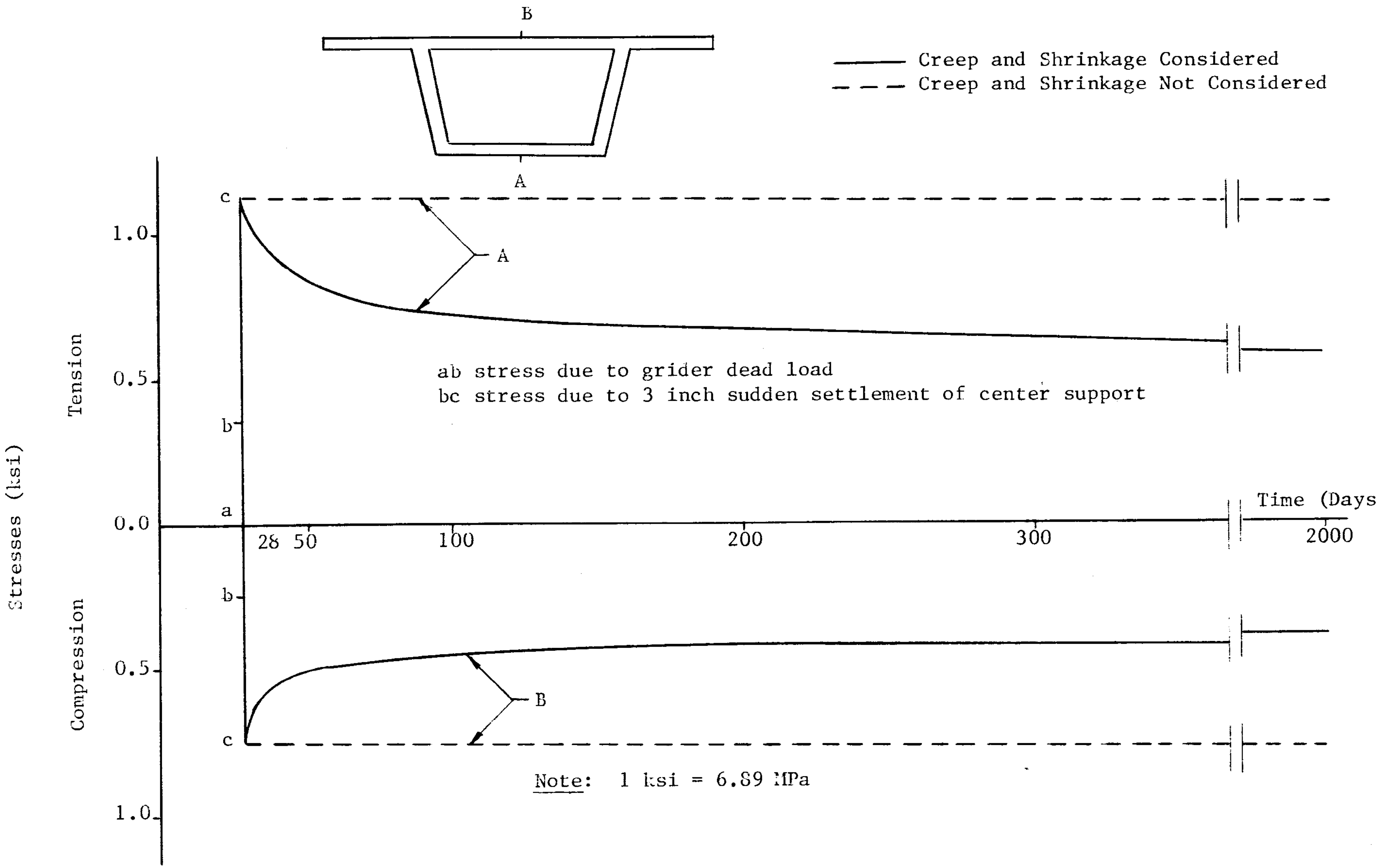


Figure 31 - Time-Dependent Stresses at Midspan for Two-Span Continuous Concrete Box Girder Bridge with 100 Foot Spans - Sudden Settlement of Center Support

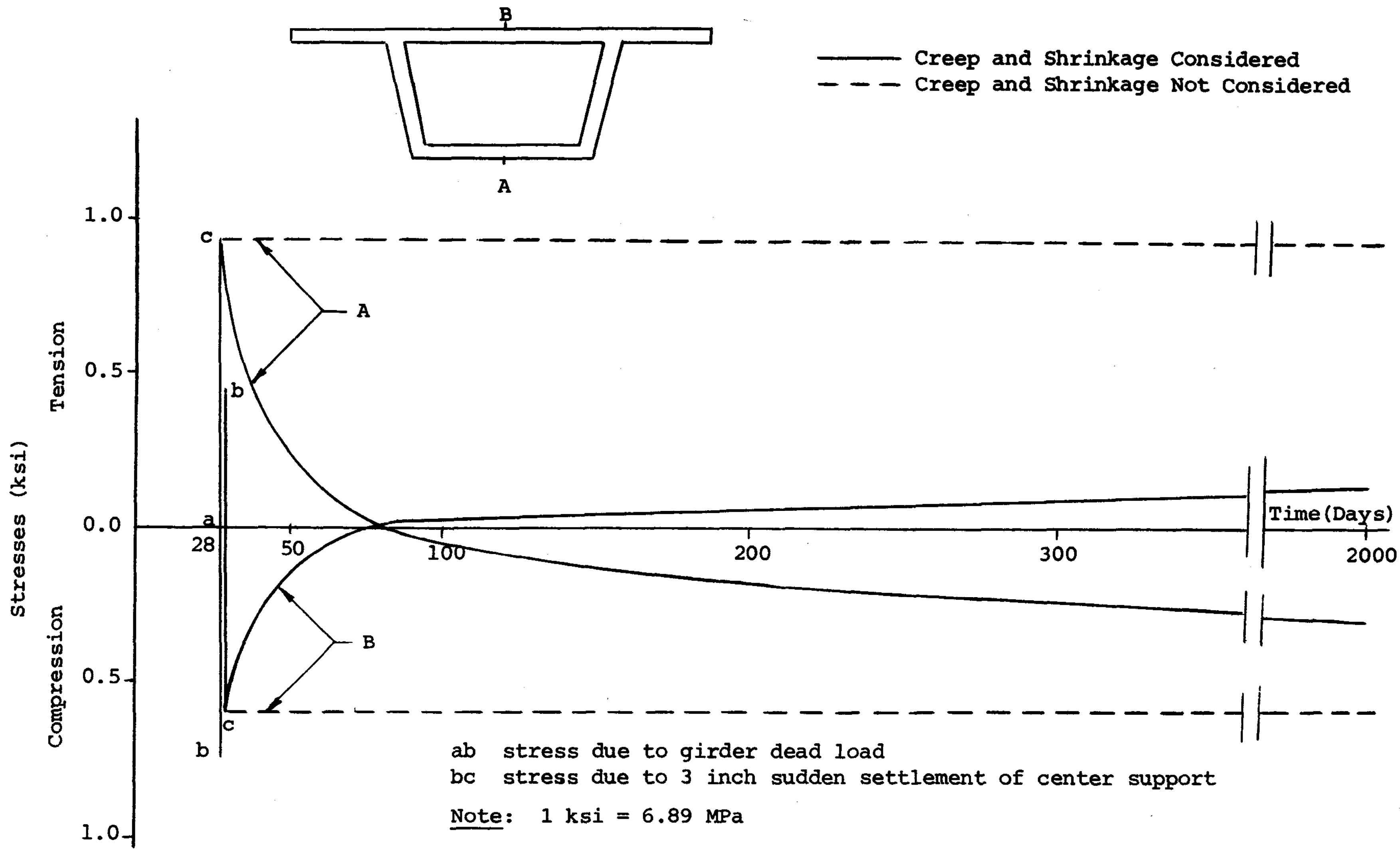
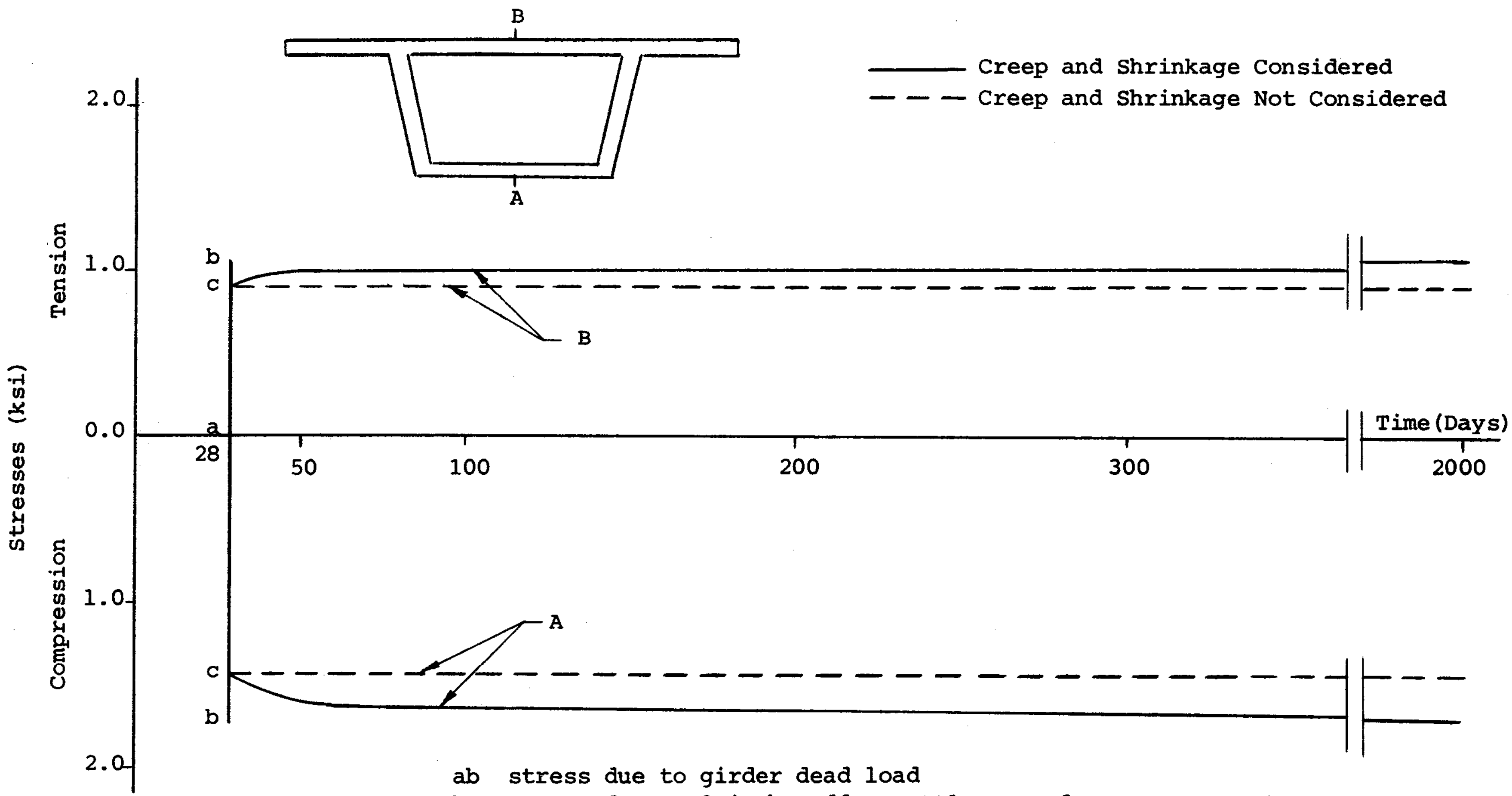


Figure 32 - Time-Dependent Stresses at Center Support for Two-Span Continuous Concrete Box Girder Bridge with 100 Foot Spans - Sudden Settlement of Center Support



ab stress due to girder dead load
bc stress due to 3 inch sudden settlement of center support

Note: 1 ksi = 6.89 MPa

Figure 33 - Time-Dependent Stresses at Center Support for Two-Span Concrete Box Girder Bridge with 200 Foot Spans - Sudden Settlement of Center Support

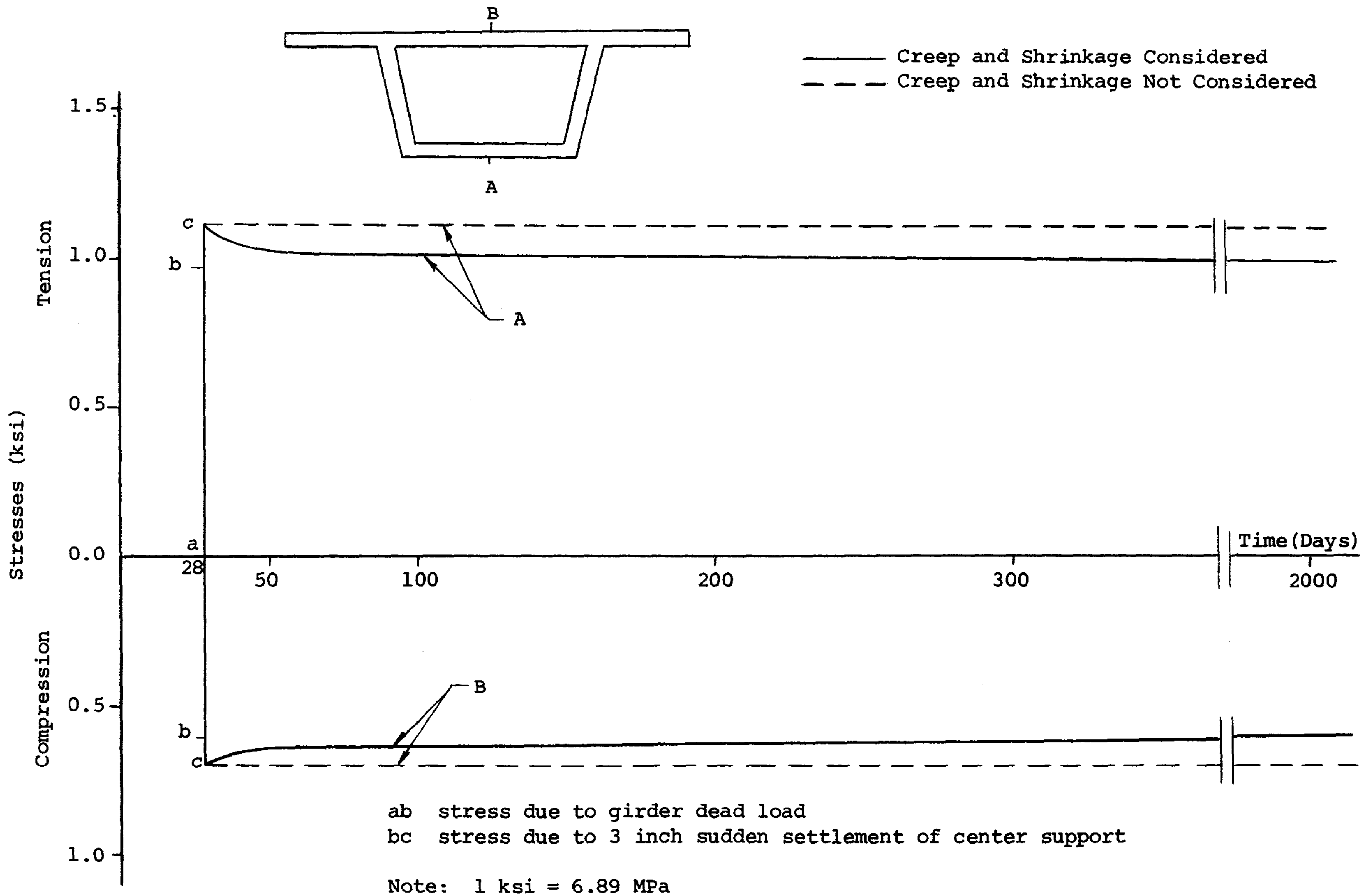


Figure 34 - Time-Dependent Stresses at Midspan for Two-Span Concrete Box Girder Bridge with 200 Foot Spans - Sudden Settlement of Center Support

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.

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 approximatey those due to dead load alone.**

5.3.2 Analysis of 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 35, 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.**

5.3.3 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.

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).

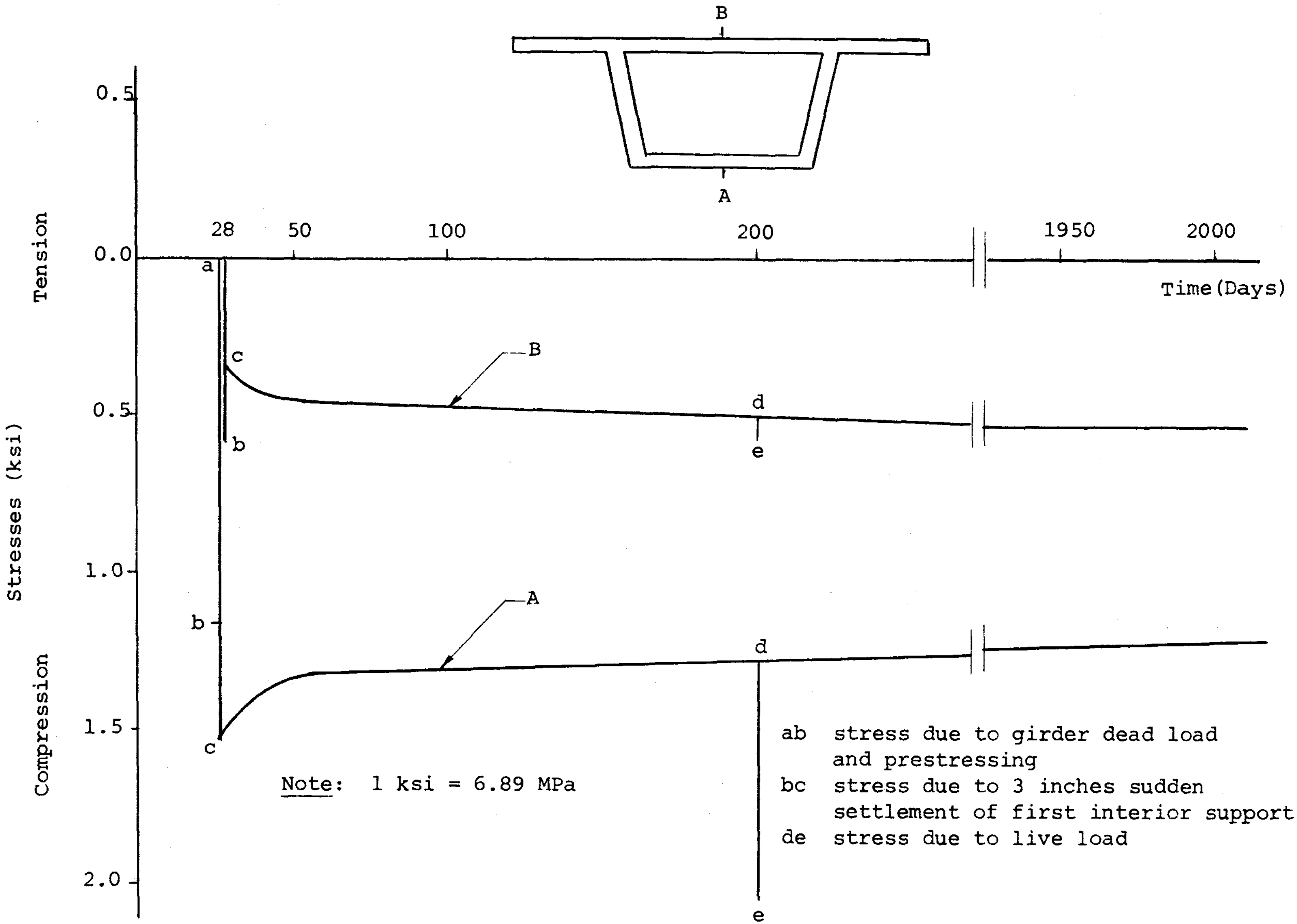


Figure 35 - Time-Dependent Stresses at Center Support of Four-Span Continuous Post-Tensioned Concrete Box Girder Bridge with 200 Foot Spans - Sudden Settlement of First Interior Support

5.4 Simplified Method of Analysis

The previous sections have outlined the various complexities of material and structural behavior which are encountered when concrete highway bridges are analyzed for the effects of foundation settlement. Because of these considerations, many practical problems may be handled more expeditiously by a simplified analysis method than by relying on an incomplete parameter study to provide estimates of settlement effects. This section will outline a simplified analysis method which can account for sudden or gradual settlements, as well as combinations of dead, live and prestressing loads, and the effects of creep.

A simple analysis method for determining time-dependent moments in reinforced concrete bridges is based on the relaxation method described in Section 5.1.1. In this method, the constant ν_u, the ultimate creep coefficient, accounts for the effects of creep due to concrete material properties and mix proportions, curing conditions, member thickness, and age of concrete at first loading. The constant ν_u is computed in accordance with ACI Committee 209 Recommendations (2).

In Section 5.2.1, the expression for time-dependent moments in the case of sudden effects is shown to be:

$$M(t) = M_{e\ell} = (1 - \frac{\nu}{1 + \nu}), \tag{18}$$

and the time-dependent moment resulting from gradual effects is:

$$M(t) = M_{e\ell}(1 - \frac{\eta\nu}{1 + \nu}) \tag{19}$$

For simplicity, indices can be defined as:

$$\Omega_s = 1 - \frac{\nu}{1 + \nu}, \tag{20}$$

and

$$\Omega_g = 1 - \frac{\eta\nu}{1 + \nu}, \tag{21}$$

so $M(t) = M_{e\ell}\Omega_s$ in the case of sudden effects, and $M(t) = M_{e\ell}\Omega_g$ in the case of gradual effects. Since η depends on ν_u and the time since loading, the indices can be tabulated as a family of curves in ν_u, plotting the settlement index with time. Since η is in the range of 0.8 to 0.9 for a wide range of loading ages and values for the ultimate creep coefficient (See Figure 25), Ω_s and Ω_g differ only slightly. Figures 36 and 37 show the variation of Ω_s and Ω_g, respectively, for a range of ultimate creep coefficients.

Figure 38 illustrates the bending moment diagram as a function of time for a two-span continuous beam where a sudden settlement occurs at the same time as dead load is applied. Since the effects of settlement and dead

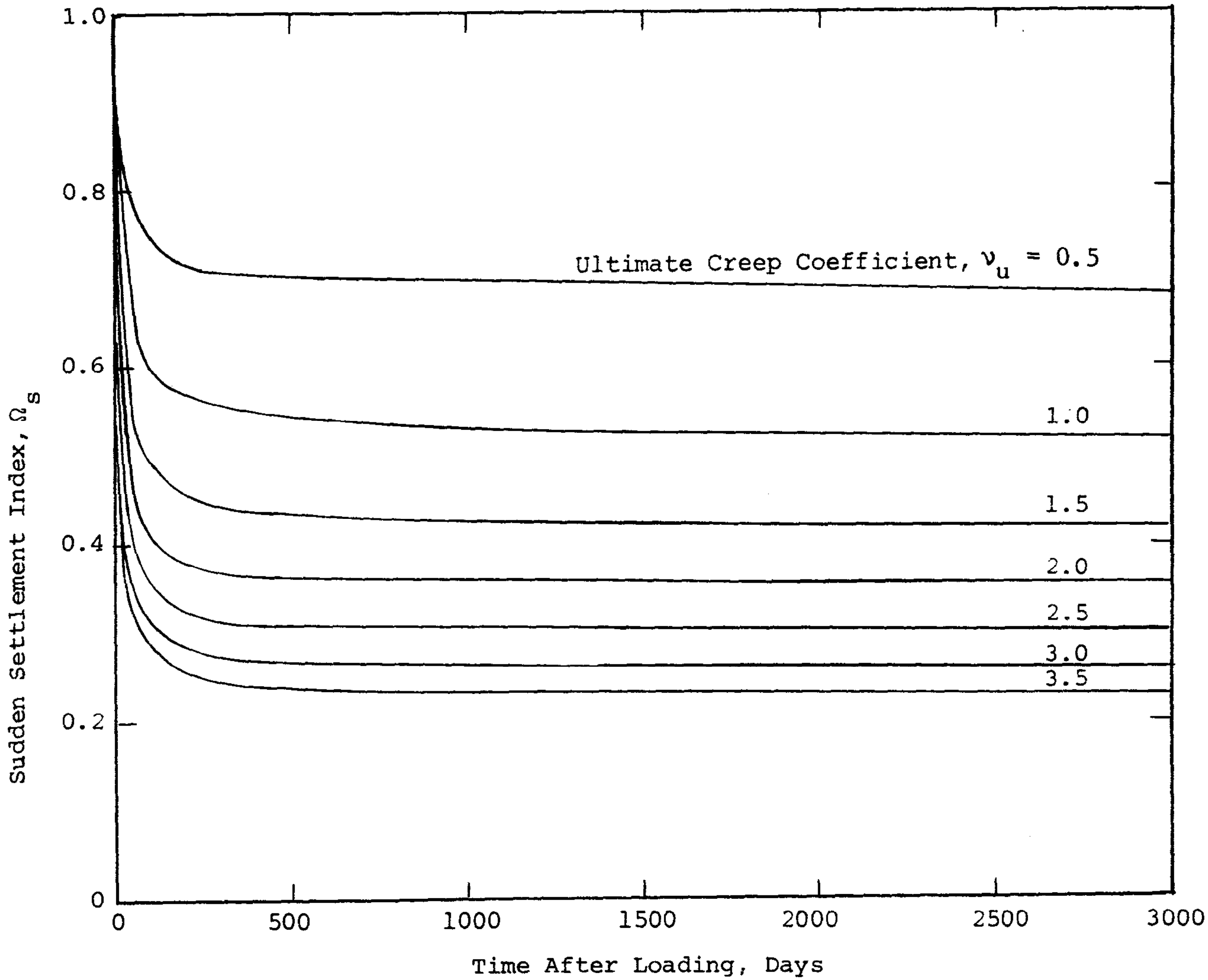


Figure 36 - Sudden Settlement Index Vs. Time After Loading For Various Values of Ultimate Creep Coefficient

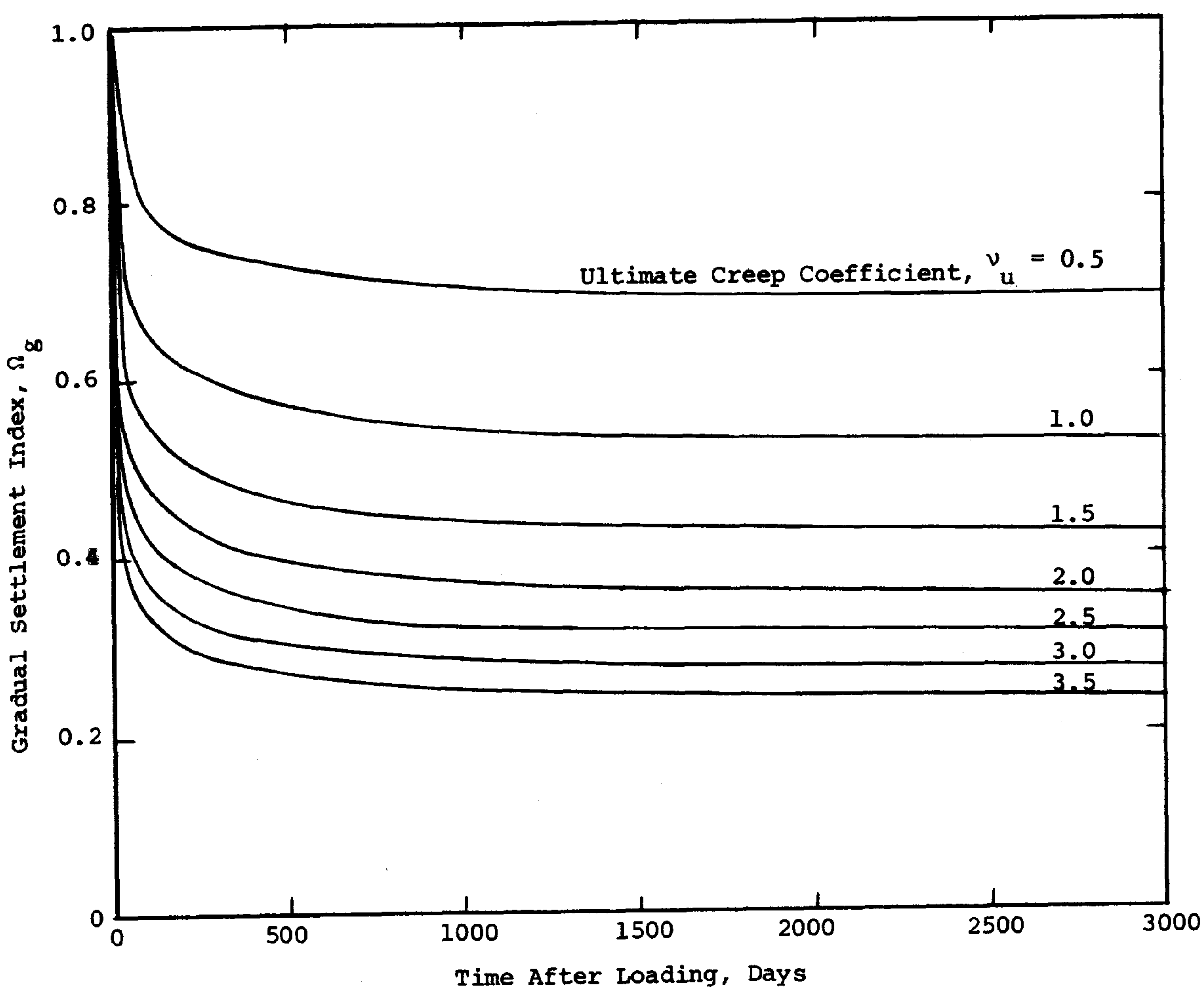


Figure 37 - Gradual Settlement Index Vs. Time After Loading For Various Values of Ultimate Creep Coefficient

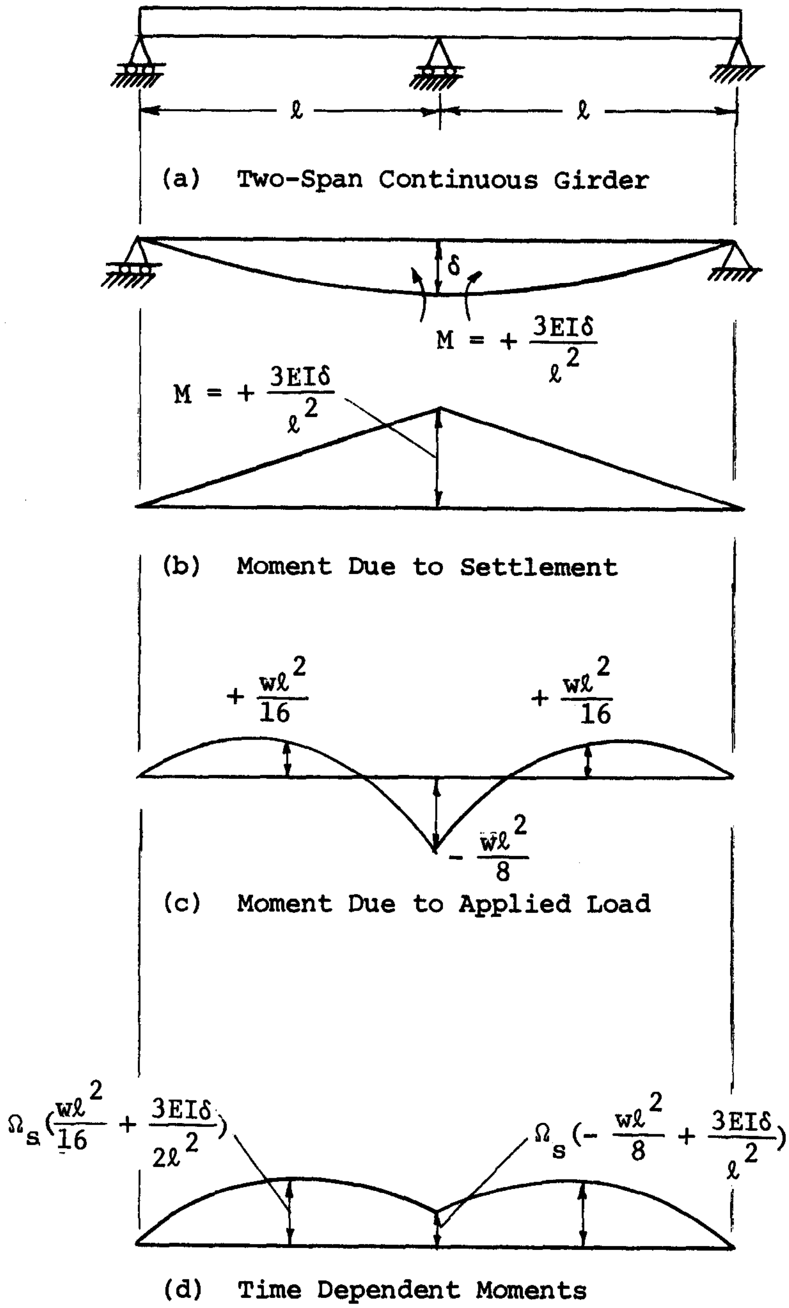


Figure 38 - Bending Moments Due to Applied Load and Settlement for Two-Span Continuous Girder

load are applied suddenly and simultaneously, at any later time the same index will apply to both effects. If the effects occur at different times, each may be considered separately and the results combined by assuming a linear superposition.

Another important case to consider is the problem of moment redistribution in a structure made continuous after some loads are applied, such as two precast girders made continuous by a joint cast-in-place in the field. In this case, the effect of creep is to induce moments over the central support where none existed previusly, as shown in Figure 39. In this case, it is possible to define the time-dependent moment over the center support as:

$$M(t) = M^{*}\Omega_{j} \tag{22}$$

where Ω_j is called the joint continuity index, and is equal to $\nu/(1 + \eta\nu)$. M* is the moment that would have existed at the joint if the structure were initially continuous. The variation of Ω_j is shown in Figure 40.

In using the index coefficients Ω_s, Ω_g, and Ω_j, time-dependent bending moments can be determined at any continuous support. After determining the support moments, bending moments at any other section can be determined from statics. Once bending moments have been established stresses can be determined from the relationship $\sigma = Mc/I$. The method is most applicable to noncomposite structures. Application to composite structures is more complex. Since ages at first loading, material properties and other characteristics of the concretes in the girder and deck are likely to be different, it is necessary to estimate an effective value of ν_u for the entire cross section. Given the overall uncertainties of these analyses, simply averaging the two coefficients should provide a reasonable approximation. In accounting for the effects of gradual settlement, the total settlement should be divided into at least three to five parts.

In the preceding paragraphs, a simplified method for analyzing time-dependent moments in concrete bridges has been presented. The advantage of this method is that it does permit hand calculations, at least for simple structures and loading conditions. It may also be implemented by computer for a more convenient solution. The procedure provides a simple and straight forward alternative to expensive and sophisticated time-step analysis methods. The major liability of the method is the requirement for careful bookkeeping during the analysis. Although this method requires additional development, it does provide a convenient means for bridge and foundation engineers to estimate the effects of foundation movement in concrete highway bridges.

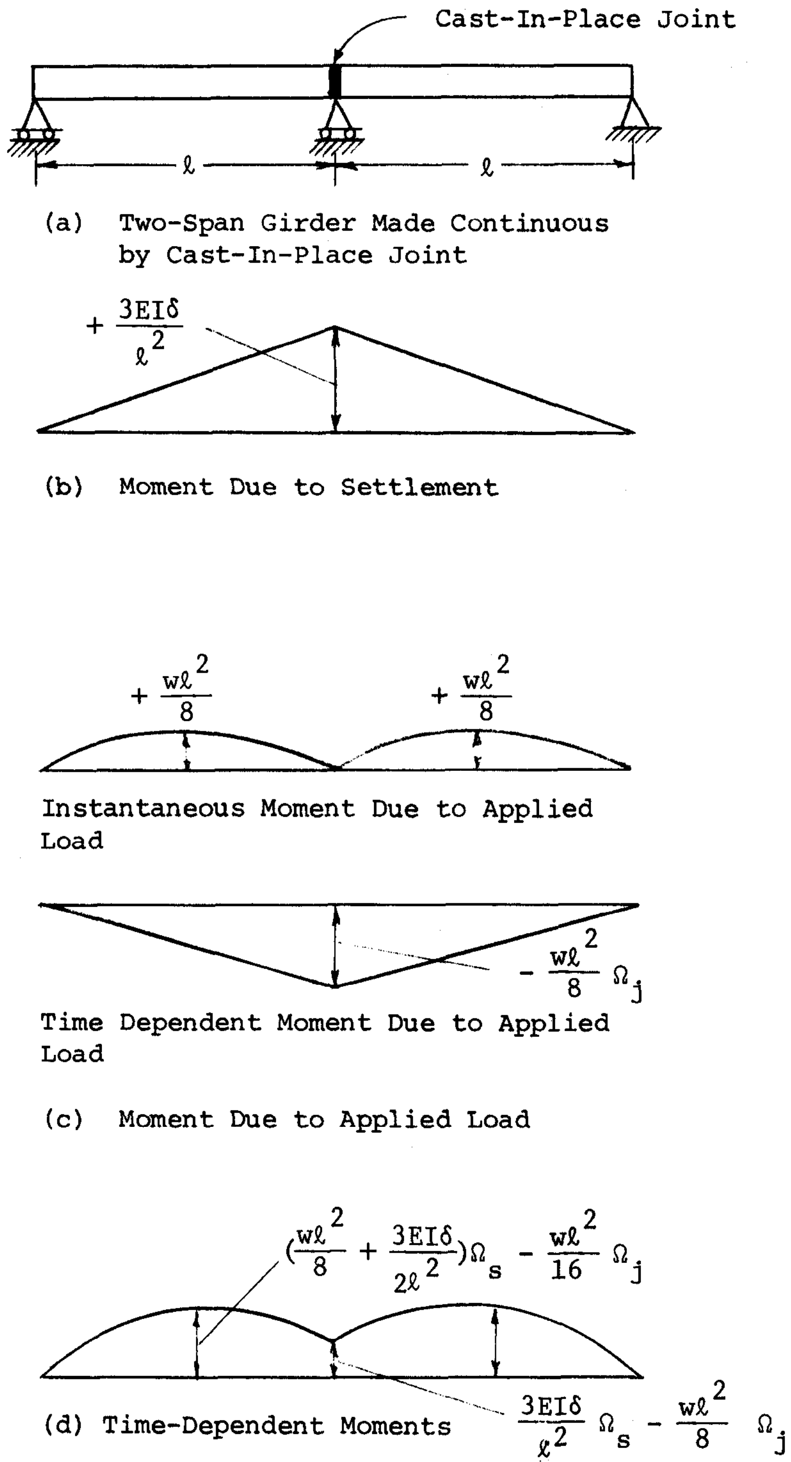


Figure 39 - Bending Moments Due to Applied Load and Settlement for Two-Span Girder Made Continuous by Cast-In-Place Joint

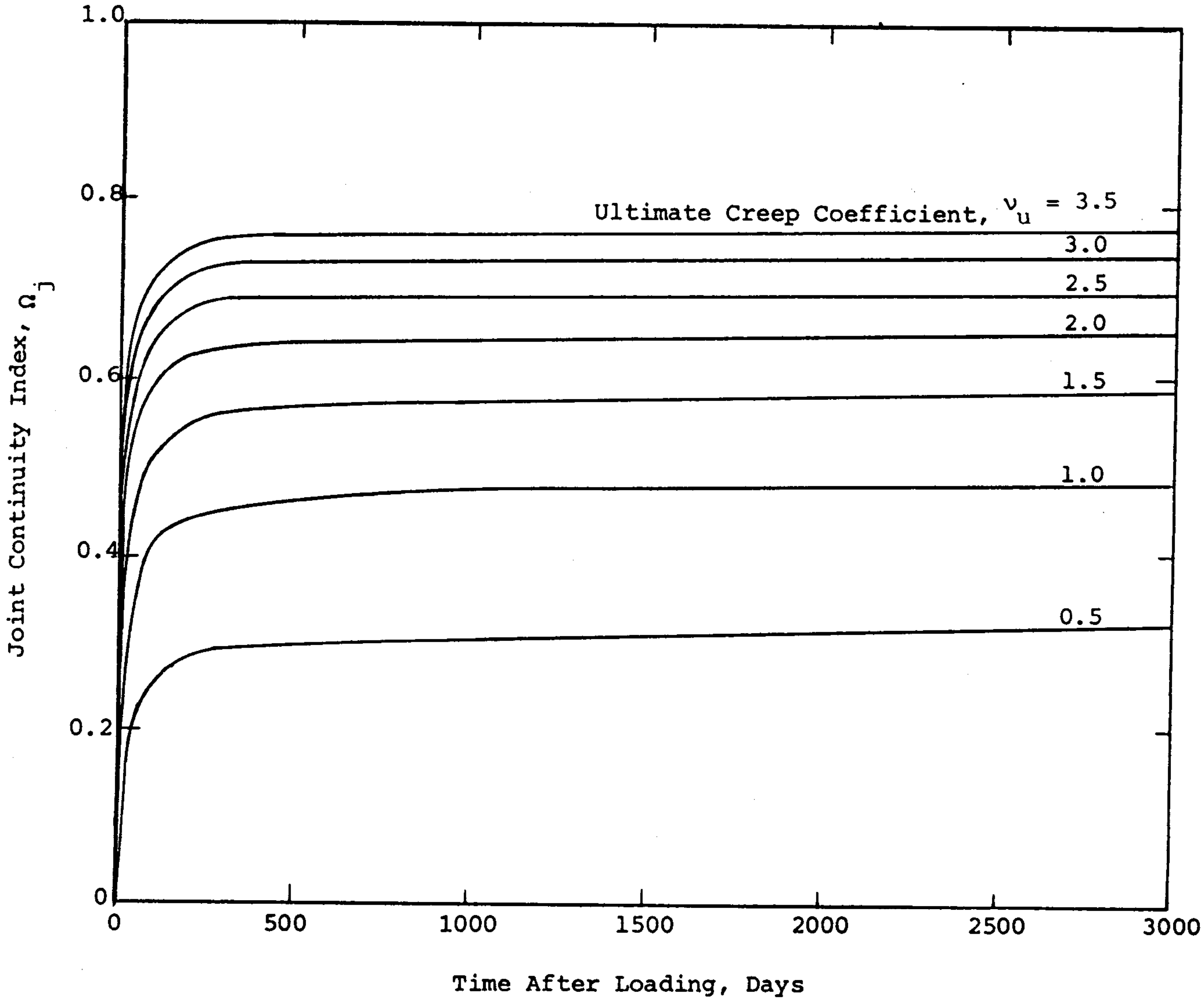


Figure 40 - Joint Continuity Index Vs. Time After Loading For Various Values of Ultimate Creep Coefficient

6. DEVELOPMENT OF DESIGN METHODOLOGY

The results of the field studies and analytical studies described above have been used to study a number of possible methodologies for the design of highway bridges that would embody a rational set of criteria for tolerable bridge movements. Although these studies are not entirely complete at this writing, significant progress has been made in establishing the framework of a proposed design procedure and selecting tolerable movement criteria for use in this procedure. The results of this investigation to date and some recommendations for tolerable movement criteria are presented below. However, these results and recommendations should be considered as preliminary and incomplete, since there is still a substantial amount of research that must be accomplished in order to refine the proposed design procedure and establish complete and reliable design criteria and guidelines for their use.

6.1 Basic Design Procedure

As indicated in the INTRODUCTION, the research described herein is part of a larger effort designed to promote the use of a systems approach to the design of highway bridges, whereby the bridge superstructure and its supporting substructure are not designed separately, but rather as a single integrated system offering the best combination of economy and long-term maintenance-free performance. A proposed design procedure that would accomplish this objective is presented schematically in Figure 41.

It is proposed 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 41. 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 is recommended that spread footing foundations 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 extremely compressible foundation soils that could lead to very large differential settlements.

Appropriate geotechnical analyses would then be conducted, as indicated in Figure 41. In the case of spread footings, it is recommended that these analyses include an evaluation of bearing capacity, estimates of long term 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. In the case of deep foundations, it is recommended that these analyses also include an evaluation of bearing capacity and settlement, as well as some appraisal of the potential for horizontal movements. 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.

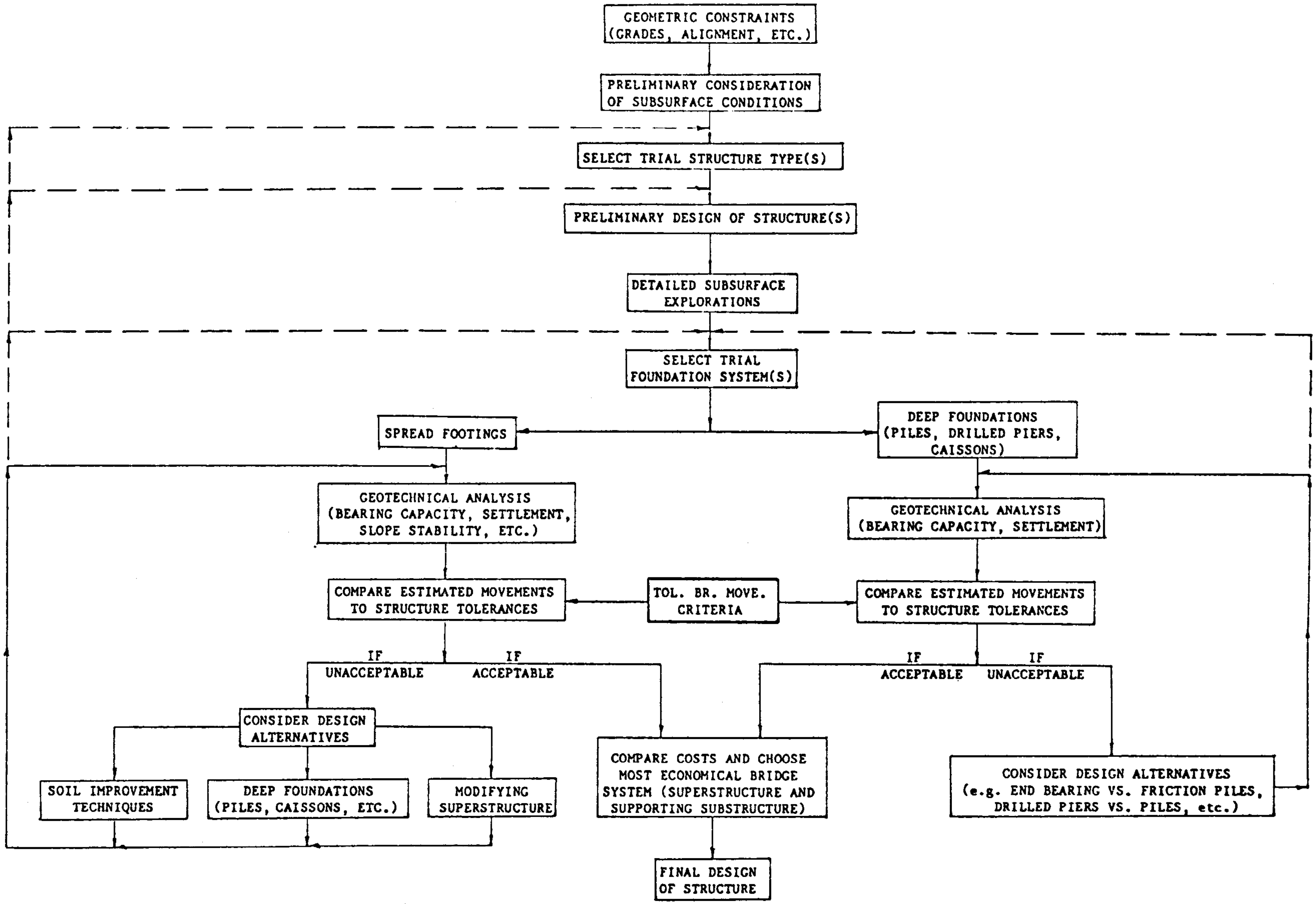


Figure 41 - Schematic Representation of Proposed Methodology For the Design of Highway Bridge Systems

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 it is proposed that the designer would consider a variety of design alternatives, as shown in Figure 41. In the case of spread footings 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 (1,6,8,40,41,57), 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 41. 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 plies, or alternate types of deep foundations, such as drilled piers or caissons rather than some type of 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.

6.2 Tolerable Movement Criteria

As a result of both field and analytical studies, it has become clear that the criteria for tolerable bridge movements must 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 may not immediately jeopardize the load carrying capacity of the bridge does not ncessarily 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 requiring 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.

The following discussion of tolerable movement criteria is limited to steel bridges, and the consideration of tolerable movement criteria for concrete bridges has been deferred until complexities associated with the time-dependent behavior of these structures can be resolved.

6.2.1 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.

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 (4,5) 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 permament, 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. It is envisioned that in practice the procedure would involve the design of the bridge in accordance with the existing AASHTO working stress design criteria assuming zero settlement and then using design aids, such as those shown in Figures 18 through 23, to check whether or not the stress increases caused by the estimated differential settlements are within tolerable limits.

The establishment of tolerable limits on overstress caused by differential settlements is currently being studied. It has been found that there is a substantial body of literature[1] describing measurements of the strains in a wide variety of highway bridges under actual and simulated highway loading conditions. The interpetation of these measured strains in terms of stress history has shown that, under typical highway loading conditions, the peak live load stresses occur relatively infrequently, and often their magnitude is below the level that would be expected based on current design criteria. This suggests that a reasonable basis might exist for the establishment of tolerable movement criteria based upon an allowable overstress. However, additional study will be required in order to resolve this matter.

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[1] For the sake of brevity, the bibliographic references to this literature have been omitted from this report. However, this list of references can be supplied upon request.

Another 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 (5). Although, in the context of the research described herein, this approach establishes one type of tolerable movement criteria based upon strength, it also contitutes one of the design alternatives (modifying superstructure) in the design procedure illustrated in Figure 41. As such, it should be considered in competition with other possible design alternatives in terms of effectiveness and economy.

One method of implementing this approach 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 (5), 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 accomodate the anticipated settlements, although the availability of design aids such as those given in Figures 18 through 23 (26) make the former method somewhat easier. In practice, the designer would use the appropriate design aids, along with predicted values of foundation settlements, to solve for maximum positive and negative settlement stresses. The resulting values would then be subtracted from the AASHTO limit (5) 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.

In an effort to overcome this drawback to some extent, the possible application of a design procedure based upon the load factor concept is presently being studied in some detail. Such procedures have become widely accepted and are recognized as being more realistic than working stress design. The current research efforts in this direction are concentrating on the development of a load factor for settlement stresses. Some consideration was also given to the possible use of "Auto Stress Design" (16,17,35), but this was abandoned because of the inherent danager of the formation of a "collapse mechanism" caused by the combined effects of support settlements and the passage of maximum live loads across the bridge.

6.2.2 Serviceability Criteria

Serviceability criteria deal with the maintenance of rider comfort and the control of functional distress. The types of movements that have been identified (36) 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 existing information, limits can be established on only some of these movements, because of the lack of a wide range of statistically reliable field information. 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 means available for predicting these horizontal movements with reasonable reliability.

On the basis of the data that have been assembled during the course of this project to date, tolerable limits have been established on (a) longitudinal angular distortion (differential settlement/span length) for simple and continuous bridges, (b) horizontal movement of abutments, (c) differential vertical settlements based on cracking of bridge decks, and (d) bridge vibrations.

6.2.2.1 Angular Distortion. 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, 96.0 percent of the angular distortions less than 0.004 were considered to be tolerable. In contrast, for simply supported steel bridges, 97.1 percent of the angular distortions less than 0.005 were reported as being tolerable. 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 the field data showed that there is a 97.9 percent probability that angular distortions less than 0.004 will be tolerable for continuous bridges, and that there is a 99.8 percent probability that angular distortions less than 0.005 will be tolerable for simply supported bridges. On this basis, the tolerable limits for longitudinal angular distortion of continuous and simply supported steel bridges were chosen to be 0.004 and 0.005, respectively.

6.2.2.2 <u>Horizontal Movements of Abutments</u>. 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 were considered to be tolerable in 83.3 percent of the cases. When accompanied by vertical movements, horizontal movements less than 2.0 inches were considered to be tolerable in only 68.2 percent of the cases. However horizontal movements of 1.0 inch and less were almost always reported as being tolerable (44). On the basis of these data, it is tentatively recommended that horizontal movements of abutments be limited to 1.5 inches. However, it is suggested 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 is currently being pursued.

6.2.2.3 <u>Differential Vertical Settlement Based on Deck Cracking</u>. 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 from Equations 3 or 5, respectively, or by the use of appropriate design aids, such as Figures 19, 21 and 23. 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 (5). 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.

6.2.2.4 <u>Bridge Vibrations</u>. As noted in Section 4.2.2 of 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 ratio of the forced (ω_f) to natural (ω_n) frequencies of a bridge were between 0.5 and 1.5. This criterion is embodied in Equation 1. By using Equation 1, a designer can determine if a proposed bridge has sufficient mass and stiffness to prevent excessive dynamic deflections. Special consideration should be given to modifying these parameters if the application of Equation 1 shows that ω_f/ω_n falls between 0.5 and 1.5.

7. SUMMARY AND CONCLUSIONS

7.1 Field Studies

The data resulting from the field studies show 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 spread footing foundations were used slightly more frequently than pile foundations for abutments. However, many more piers were founded on piles than on spread footings. Although the movements of spread footing foundations occurred a little more frequently, the movements of pile foundations had slightly greater magnitudes. This suggests 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.

7.2 Analytical Studies of Steel Bridges

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. 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} (327,741 mm^3) or less.

The stress increases produced in the two-span continuous parallel and non-parallel chord 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.

The results of the dynamic analysis of steel bridges suggested that very careful consideration needs to be given in design to the inclusion of provisions for adequate damping, stiffness and mass to reduce the possibility of intolerable dynamic vibrations. This is particularly critical for span lengths greater than 150 feet (45.7 meters).

7.3 Analytical Studies of Concrete Bridges

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.

A variety of analytical tools has been presented and discussed. The relaxation method provides a simple basis for analysis, and can be used to provide an approximate analysis, even in complex structures. A simplified method of performing relaxation analyses by hand calculations was described. Some of the studies were accomplished by means of a sophisticated, step-by-step computer method. Although this method provides a much more refined estimate of stresses resulting from settlement and creep, it is a complicated procedure to implement and is expensive to use.

"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. 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 occuring 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.

This investigation did not lead to the development of simple design aids for the estimation of time-dependent stresses caused by differential settlement, shrinkage and creep. Such design aids may be feasible to develop, but they do require exhaustive parameter studies, which are time consuming and expensive to perform even using the most sophisticated analytical tools. Additional work in this area should focus on refining the approximate analytical procedures and performing additional parameter studies, which emphasize effects of gradual settlement, prestressing strand profile and settlements occurring several weeks, months, and years after the structure has been erected.

7.4 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.

7.5 Tolerable Movement Criteria

The results of both field and analytical studies have been utilized in an investigation aimed at developing tolerable movement criteria for steel bridges based upon both strength and serviceability. Although these studies are not presently complete, the continuing efforts with respect to the establishment of tolerable movement criteria based on strength are described, and tentative serviceability criteria are presented, based on limiting longitudinal angular distortion, horizontal movements of abutments, deck cracking and bridge vibrations.

Since the results of both field and analytical studies have shown that many continuous bridges may experience relatively modest stress increases as a result of foundation movements, an attempt is currently being made to establish strength criteria 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 has been 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 have been 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 is being studied is the use of load factor design, which has been increasing in popularity in recent years. To this end, studies are currently underway in an effort to develop a load factor for differential settlement stresses.

7.6 Need for Additional Research

It should be noted that there are several additional aspects of bridge design, construction and performance, relating to the tolerance to bridge movements, requiring further research. For example, strength and serviceability criteria for skewed highway bridges subjected to differential movements have yet to be established. Preliminary analytical studies revealed that skewed bridges are more susceptible to damage resulting from differential movements than rectangular ones. In addition, considerable additional research will be required in order to resolve the complexities associated with the time dependent behavior of continuous concrete bridges and to develop tolerable movement criteria for these structures.

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APPENDIX

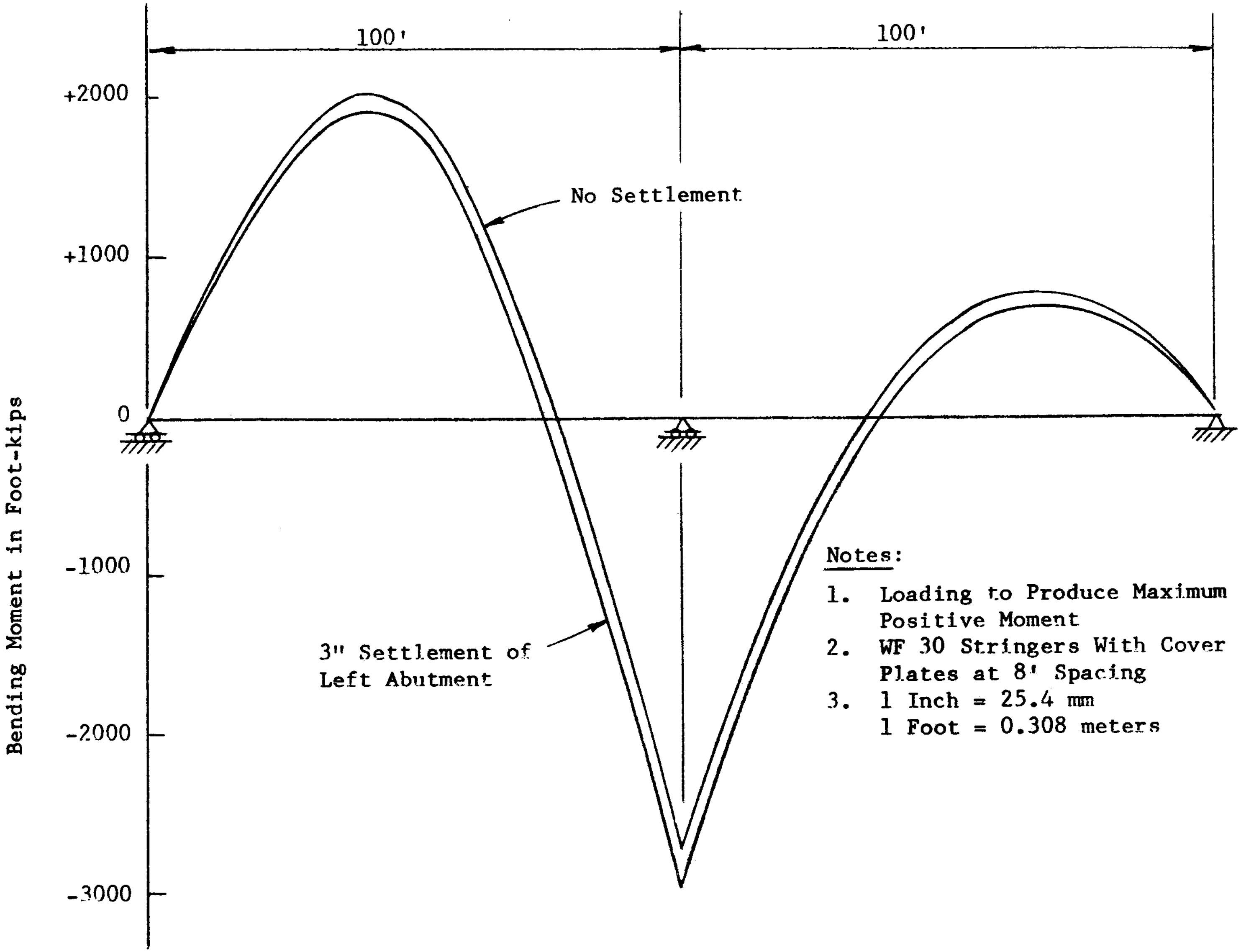

Figure 42. Typical Moment Diagram for Two-Span Continuous Bridge, With 100 Foot Spans, Loaded With Dead Load, Live Load and Settlement of Left Abutment

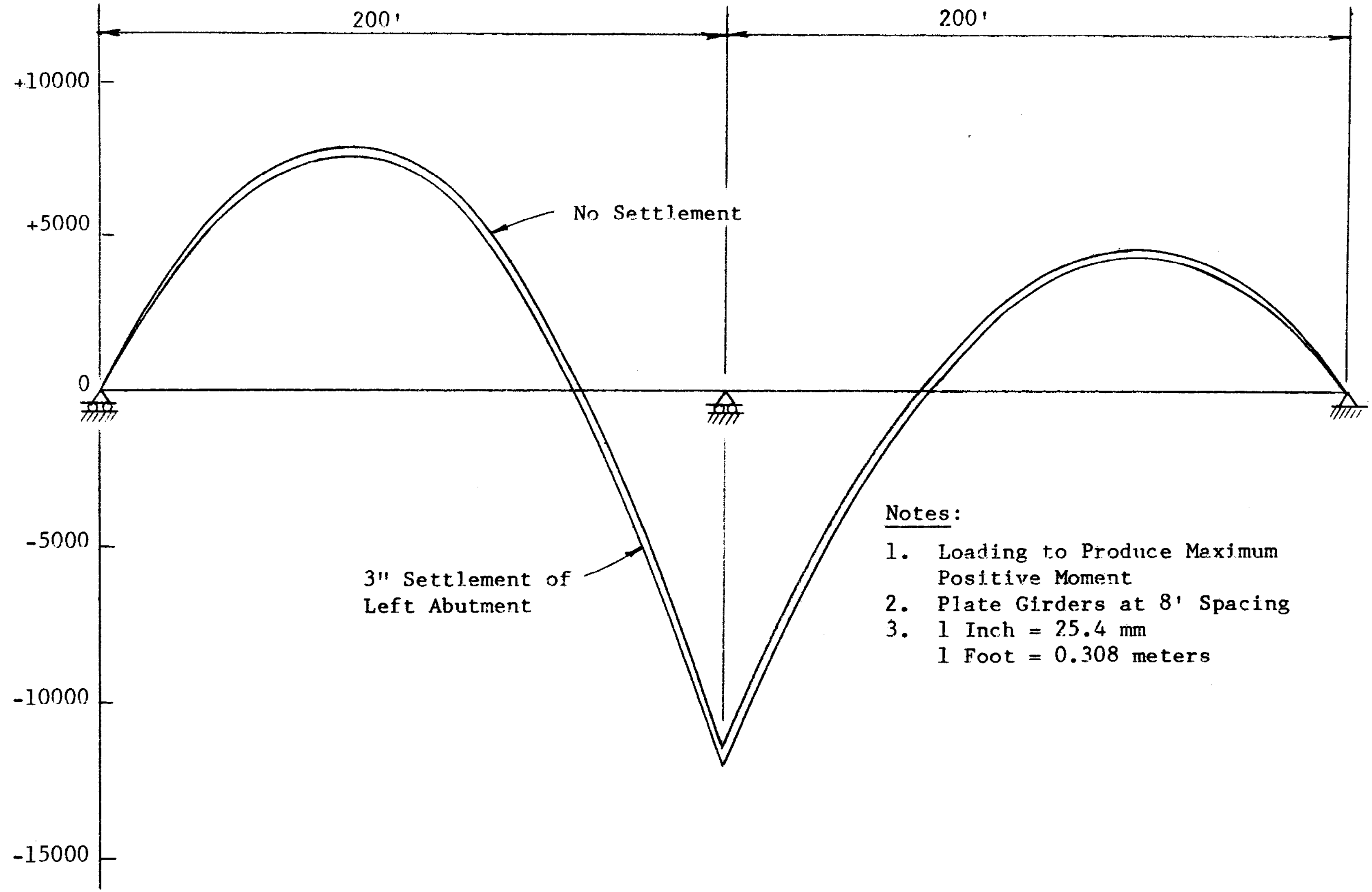

Figure 43. Typical Moment Diagram for Two-Span Continuous Bridge With 200 Foot Spans, Loaded With Dead Load, Live Load and Settlement of Left Abutment

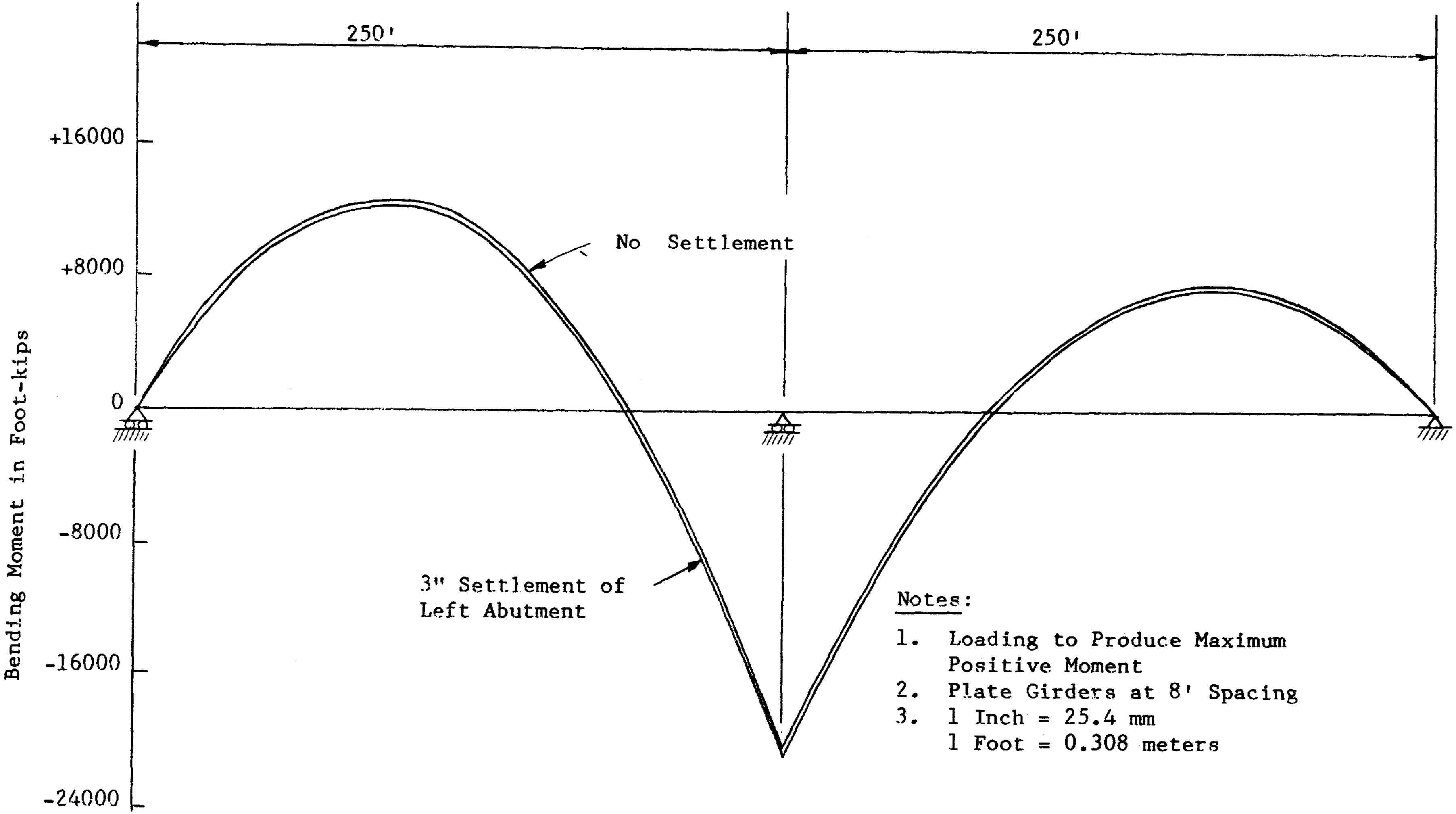

Figure 44. Typical Moment Diagram for Two-Span Continuous Bridge With 250 Foot Spans, Loaded With Dead Load, Live Load and Settlement of Left Abutment

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