FHWA-NJ-2002-19

Cause and Control of Transverse Cracking in Concrete Bridge Decks

Final Report
December 2002

Submitted by

Dr. M Ala Saadeghvaziri
Professor

Rambod Hadidi
Research Assistant

Department of Civil and Environmental Engineering
New Jersey Institute of Technology
Newark, NJ 07102

NJDOT Research Project Manager
Mr. Raj Chawla

In cooperation with

New Jersey
Department of Transportation
Division of Research and Technology
and
U.S. Department of Transportation
Federal Highway Administration

DISCLAIMER STATEMENT

TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No. FHWA-NJ-2002-019	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Cause and Control of Transverse Cracking in Concrete Bridge Decks		5. Report Date December 2002
		6. Performing Organization Code
7. Author(s) M. Ala Saadeghvaziri and Rambod Hadidi		8. Performing Organization Report No.
9. Performing Organization Name and Address Department of Civil and Environmental Engineering, New Jersey Institute of Technology, Newark, NJ 07102-1982		10. Work Unit No.
		11. Contract or Grant No.
12. Sponsoring Agency Name and Address New Jersey Department of Transportation, PO 600, Trenton, NJ 08625	Federal Highway Administration, U.S. Department of Transportation, Washington, D.C.	13. Type of Report and Period Covered March 2000 to December 2002
		14. Sponsoring Agency Code

15. Supplementary Notes

16. Abstract

Many concrete bridge decks develop transverse cracking and most of these cracks develop at early ages, some right after construction, and some after the bridge has been opened to traffic for a period of time. Transverse cracks usually occur when concrete is set and widen with time. These cracks have been observed in most geographical locations, and on many superstructure types. It is estimated that more than 100,000 bridges in the United States develop early transverse cracks. These cracks are typically full depth located 1-3 m (4-12 ft) apart along the length of the span and are usually observed over transverse reinforcement. It has been reported that predominant form of deck cracking is transverse cracking. These cracks reduce the service life of the structure and increase maintenance costs, which is of paramount importance in highway maintenance activities. Transverse cracks accelerate reinforcement corrosion, especially in regions where deicing chemical are applied. Corrosion damage has been observed even on epoxy coated reinforcing bars. Freeze-thaw cycles of water in cracks and leakage of water to supporting structures may also reduce service life of structure.

Cracks in concrete occur when a *restraint* mass of concrete tends to *change volume*. Volume change in concrete depends on the properties of its constituents and their proportions as well as environmental conditions such as ambient temperature changes and humidity. Restraint, which is basically due to composite action of deck and girder, depends on design characteristics of the bridge (i.e., structural design factors). Construction techniques also contribute to volume change and/or to degree of restraint of concrete mass.

Factors associated with mix design/material and construction procedures have been the subject of a significant number of research studies over the past several decades. Structural design factors, however, have not been the subject of much research in the past and they were the main thrust of this research study. Using 2-D and 3-D linear and nonlinear finite element models many design factors such as girder stiffness, deck thickness, girder spacing, relative stiffness of deck to girder, amount of reinforcements, etc. were studied. The research study also included a comprehensive review of the existing literature as well as survey of 24 bridges in the state of New Jersey. Results of each research task are presented and discussed in details. Furthermore, based on analytical results and literature review effect of various factors are quantified and specific recommendations for possible consideration in design are made. These are classified under the three major categories, namely: 1) material and mix design, 2) construction practice and ambient condition factors, and 3) structural design. A simple window application program to more accurately estimate deck stresses during design is also developed under this study. Future research needs are also identified.

17. Key Words Concrete, Deck, Cracking, Cracks, Transverse, Bridges, Causes of Cracking, Control of Cracking		18. Distribution Statement	
19. Security Class (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages 203	22. Price

Form DOT F 1700.7 (8-69)

ACKNOWLEDGEMENT

This research study was supported by the New Jersey Department of Transportation/FHWA and the National Center for Transportation and Industrial Productivity (NCTIP) at NJIT. The assistance of NJDOT staff, especially Fred Lovett from the Bureau of Materials, Jose Lopez and Gerry Sellner from the Bureau of Structures & Design, and Raj Chawla the project monitor in conducting the field surveys and/or providing material and design data on bridges is greatly appreciated. The results and conclusions are those of the authors and do not necessarily reflect the views of the sponsors.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF SYMBOLS

$A_{girder}, A_{Composite}$	=	Cross sectional area of girder and composite section
$A_{Element}$	=	Area of Element
A_{ri}	=	Total cross sectional area of reinforcement layer i
A_i	=	Cross sectional area of section i
C_{top}, C_{bottom}	=	Top and bottom reinforcement cover
d_{ti}	=	Distance from centroid to top fiber of section i
d_{bi}	=	Distance from centroid to bottom fiber of section i
d_{bg}, d_{tg}	=	Distance of top and bottom fiber of girder from centroid
d_{bc}, d_{tc}	=	Distance of top and bottom fiber of composite section from centroid
d_{rebar}	=	Reinforcement diameter
E_c	=	Modulus of elasticity of concrete
E_i	=	Modulus of elasticity of section i
$E_{Element}$	=	Modulus of elasticity of element
$E_{girder}, E_{composite}$	=	Modulus of elasticity of girder and composite section
F_i	=	Force resultant of stresses at interface section i and i+1
Fr_i	=	Force in reinforcement layer i (positive denotes tensile force)
f_r	=	Modulus of rupture of concrete
f_c'	=	Compressive strength of concrete
h_i	=	Height of section I
$I_{Girder}, I_{Composite}$	=	Girder and transformed composite section moment of Inertia
I_i	=	Moment of inertia of section i
K_{vs}, K_h	=	Size and humidity correction factors
L, L_1, L_2, L_3	=	Span length
$l_{element}$	=	Element length
M, M_1, M_2	=	Redundant Moment
P	=	Redundant force
Q_i	=	Moment resultant of stresses at interface section i and i+1
R_i	=	Curvature of section i
S_{bi}, S_{ti}	=	Bottom and top section modulus of section i
S_{ri}	=	Section modulus at level of reinforcement
$\bar{t}$	=	Time in days after measured after curing
t	=	Deck thickness
ti	=	Time
$T_0, T_1, T_2, ..., T_8$	=	Girder and deck temperature changes

Tr_i	=	Temperature at reinforcement layer i
u	=	Reinforcement bond stress
v/s	=	Volume to surface ration
w	=	Reinforcement slip
α_i	=	Coefficient of thermal expansion of section i
α_{ri}	=	Coefficient of thermal expansion of rebar layer i
$\varepsilon_{Bottomi}$	=	Strain at the bottom of section i (Tensile strain is positive)
ε_{Topi}	=	Strain at the top of the section i (Tensile strain is positive)
$\varepsilon_{sh,t}$	=	Shrinkage strain
$\varepsilon_{sh,u}$	=	Ultimate Shrinkage strain
$\sigma_{DeckBottom}, \sigma_{DeckTop}$	=	Deck bottom and top stresses (Tensile stress is positive)
$\Delta\sigma_{DeckBottom}, \Delta\sigma_{DeckTop}$	=	Additional deck bottom and top stresses (Tensile stress is positive)

SUMMARY

A significant number of concrete bridge decks develop transverse deck cracking, often at early ages. Cracks in concrete occur when a *restraint* mass of concrete tends to *change volume*. Amount of volume change in concrete depends on mix design and construction procedures. Factors associated with mix design/material and construction procedures have been the subject of a significant number of researches over the past several decades. Restraining effect arises from the composite action between the deck and girder, which is mostly controlled by structural design factors although partly dependent on construction practices too. Structural design factors have not been the subject of much research in the past and they were the main thrust of this research study. Using 2-D and 3-D linear and nonlinear finite element models many design factors such as girder stiffness, deck thickness, girder spacing, relative stiffness of deck to girder, amount of reinforcements, etc. were studied. The research study also included a comprehensive review of the existing literature as well as survey of 24 bridges in the state of New Jersey. Based on the results of this research study the following recommendations are made:

Structural Design Factors

- Specify an upper limit on actual concrete strength vs. the design value.
- Minimize the ratio of girder to deck stiffness.
- Boundary restraints should be consistent with design.
- Time-dependent loadings must be considered in design of bridges with integral abutments.
- Employ more flexible superstructures.
- Use uniform reinforcement meshes.
- Design should consider AASHTO Article 3.12 (A simple design tool has been developed under this study to facilitate this).

Mix Design (Material)

- Reduce cement content to 650-660 lb/yd^3, and consider using fly ash.
- Limit water cement (w/c) ratio to 0.4-0.45.
- Use AASHTO specification Type II cement.
- Adopt a restraint shrinkage test.
- Consider using type K shrinkage compensating concrete when available.
- Use aggregate size and shape as discussed in the report.

Construction Practices

- As specified in NJDOT Specs, ensure that curing starts immediately after finishing and wet cure for at least 7 consecutive days.
- If “early-open” is not an issue consider 14-day wet curing.
- Make use of evaporation rate chart proposed by ACI and cast the deck in mild temperatures.
- Record wind speed and humidity during construction for future reference.
- Give consideration to pouring sequence as proposed under this report.

Training and Implementation

Many of the recommendations are straightforward and can be implemented immediately. However, couple of these recommendations may have implications and/or may require some training. NJIT will work with NJDOT, specifically Bureaus of Structures & Design and Materials, to coordinate implementation of these recommendations.

Future Research Needs

Several areas of future research have been identified that are all very important. For example the proposed connector called Controlled Composite Action (CCA), as discussed in last chapter, has great potential and should be seriously considered for future investigation. CCA has the high potential to eliminate, as oppose to minimizing, the problem of deck cracking.

INTRODUCTION

This chapter introduces and explains transverse deck cracking in concrete bridge decks and its effects on the bridge performance. It also discusses the needs for further research, such as this study, by presenting and discussing the results of prior research activities on this subject. The objectives and plan for this research study are also discussed in some details. Finally, the report organization is presented.

Problem Statement

New Jersey Department of Transportation's (NJDOT) Bureau of Construction and Materials have noticed an increase in the number of cracking in concrete bridge decks. Literature indicates that indeed many concrete bridge decks develop transverse cracking and most of these cracks develop at early ages, many, right after construction. These cracks are typically full-depth and spaced 3 to 10 ft apart along the length of the bridge. Transverse cracks can accelerate corrosion of reinforcing steel, deteriorate deck concrete, possibly cause damage to underneath components of the bridge, and damage bridge esthetic. As a result of these adverse effects of transverse cracking, the maintenance costs will increase and ultimately the service life of the bridge system will be shortened.

A picture of a full width transverse crack in a bridge deck is shown in Figure 1 Although not quite clear, this is shown to emphasize the extent of damage that a crack can cause once it is initiated. The crack has propagated all the way into the monolithic parapet. The arrow shows the location of the crack. As indicated by the dark water trace on the underneath girder the reinforcements are also corroded. Maintenance and repair costs associated with damage like this to our infrastructure put a significant burden on highway agencies' resources. Figure 2 shows picture of another bridge deck crack that is quite wide and extends the full width of the deck.

Figure 1. Corrosion due to transverse deck cracks in a bridge in New Jersey.

Figure 2. Transverse deck cracks on a bridge deck in New Jersey.

There have been significant numbers of studies on the cause of transverse deck cracking, as it will be discussed in the next chapter. However, the causes are not yet fully understood and the problem still exists. Previous studies were mostly focused on concrete mix design and improvement through changes to construction practices to alleviate shrinkage problem. In many instances there are major disagreements on the factors affecting transverse cracking indicating the need for further research. For examples, Dakhil et al. (1975) report a direct relationship between an increase in cracking and an increase in concrete slump while Cheng and Johnston (1985) have observed a decrease in transverse cracking in concrete bridge decks with increasing slump. Contradiction on the effect of girder type is another important example on the lack of full understanding of the causes of this phenomenon. Meyers (1982) indicates that structures supported on wide flange and composite steel-plate girders exhibited much more cracking than those constructed on other systems. However, the results of our survey showed that the percentage of the cracked decks supported on prestressed concrete girder are higher. Furthermore, the review of the construction and mix design documents for the bridges surveyed and NJDOT 1996 Standard Specifications for Road and Bridge Construction and 1998 Supplemental Specification (hereafter NJDOT Specs) show that most of the important recommendations of previous studies are already included in mix design and construction of bridge decks in New Jersey. However, transverse cracks are still observed on some of the newly constructed bridge decks indicating the contribution of other factors and the need for further study to identify these factors and propose remedies.

Research Objectives

The overall objective of this research is to investigate the cause(s) of transverse concrete deck cracking and to develop solutions for possible implementation in design and construction of new concrete bridge decks. Thus, the tasks to achieve these objectives are as follow:

- Literature review and evaluation of previous research recommendations along with review of current design and construction practices in New Jersey,
- Development of a database and statistical analysis based upon survey of bridge decks, and evaluation of results in light of literature review,
- Development of 3-D finite element models and parametric study,
- Development of simplified analytical procedures for design purposes,
- In-field evaluation and response measurements bridge,
- Development and formulation of design recommendations.

This report discusses the results of the study. In the following section the research approach for tasks conducted during this study is briefly discussed.

Research Approach

There have been significant numbers of studies on the cause of transverse deck cracking. These studies were reviewed to identify their most important recommendations and findings. These recommendations were compared to NJDOT Specs (1998) to identify any discrepancy and to make recommendation for possible adoption. The literature findings are also critically reviewed to adopt appropriate research approach and to identify areas for further work.

24 bridges were surveyed in New Jersey. Their conditions were not known a priori. The criteria for selecting these bridges were that i) they are built after 1994 (5 year old age at the time of this research), and ii) they span more than 75-ft, single or multiple spans. It turned out that most bridges selected were cracked, although there were a few uncracked bridges among those surveyed. During the surveys crack information such as spacing, width and location are qualitatively recorded. The design and construction documents for the bridges surveyed were also reviewed. Based on the information collected a database was developed and it was used in the subsequent statistical analysis. The objective of the statistical analysis was to identify major factors causing transverse deck cracking in New Jersey. Using these results and the knowledge gained through literature survey a narrower list of factors were selected for a more focused study using finite element analysis.

Several Finite Element (FE) models are developed. A comprehensive linear and nonlinear Finite Element Analysis (FEA) were conducted to study crack patterns on bridge deck and quantify the effect of design factors affecting deck transverse cracking. These factors were selected based on the literature review and field surveys conducted within initial phases of this study. Based on the results of the FEA further recommendations are made to reduce crack tendency in bridge decks.

To study the effect of ambient and hydration temperature as well as shrinkage on deck strains and stresses, four bridge decks were instrumented. Temperature and strains were monitored and recorded during each test. These data are presented and analyzed to identify the effect of hydration, daily temperature, and shrinkage on transverse deck cracking.

Finally, a simplified method to evaluate stresses in concrete bridge deck due to shrinkage, hydration effects and ambient temperature changes was developed. It is proposed that this method be used during design stage to evaluate different superstructure design parameters with respect to transverse deck cracking. To further facilitate use of this method, a user-friendly windows based application is developed, which implements this method.

Report Organization

Results of this study are organized in seven chapters and one appendix. Following this introductory chapter on problem statement and research objective, the literature reviews are presented and discussed in chapter two. Chapter three discusses the details of field study, data gathering, database development, and discussion of field study results. Chapter four examines the causes of volume change in concrete and relevant test methods. 2D and 3D finite element study results are presented in chapter five. Details of the simple method developed to estimate deck stresses are discussed in chapter six. Finally, chapter seven presents recommendations of this study. There are also an appendix, which contains details of the information collected for each bridge during field surveys.

LITERATURE REVIEW

This chapter presents the results of a comprehensive literature review on the cause of transverse deck cracking. It includes experimental and analytical research results as well as survey studies on the effects of different factors on concrete deck cracking. Consistent with past work on the subject, causes are classified under three categories, namely: 1) material and mix design, 2) construction practice and ambient condition factors, and 3) structural design. Finally areas for further research are identified

Background

Many concrete bridge decks develop transverse cracking and most of these cracks develop at early ages, some right after construction, and some after the bridge has been opened to traffic for a period of time. Transverse cracks usually occur when concrete is set (Iowa DOT, 1986; Kosel et al., 1985; PCA, 1970) and widen with time (Cady et al., 1971; Horn et al., 1972; Ramey et al., 1997). These cracks have been observed in most geographical locations, and on many superstructure types (Krauss and Rogalla, 1996). It is estimated that more than 100,000 bridges in the United States develop early transverse cracks (Krauss and Rogalla, 1996). These cracks are typically full depth (Krauss and Rogalla, 1996; La Fraugh, 1989; Mc Keel, 1985) located 1-3 m (4-12 ft) apart along the length of the span (Cheng and Johnson, 1985; Kosel et al., 1985; PCA, 1970) and are usually observed over transverse reinforcement (Krauss and Rogalla, 1996; Iwoa DOT, 1986; Mc Keel, 1985; PCA, 1970; Ramey et al., 1997). It has been reported that predominant form of deck cracking is transverse cracking (Cady et al., 1971; Mc Keel, 1985; PCA, 1970; Ramey et al., 1975). These cracks reduce the service life of the structure and increase maintenance costs, which is of paramount importance in highway maintenance activities. Transverse cracks accelerate reinforcement corrosion, especially in regions where deicing chemical are applied (Mc Keel, 1985; Perfetti et al., 1985). Corrosion damage has been observed even on epoxy coated reinforcing bars (Perfetti et al., 1985). Freeze-thaw cycles of water in cracks and leakage of water to supporting structures may also reduce service life of structure.

Although transverse cracking in bridge decks has been one of the main concerns of designers and researchers for decades, effect of contributing factors on transverse cracking is not fully understood yet. This chapter examines the state of knowledge on transverse deck cracking, discusses areas for further research, and presents a set of recommendations for reducing possibility of transverse deck cracking.

Causes of Transverse Deck Cracking

Cracks in concrete occur when a *restraint* mass of concrete tends to *change volume*. Volume change in concrete depends on the properties of its constituents and their proportions as well as environmental conditions such as ambient temperature changes and humidity. Restraint, which is basically due to composite action of deck and girder, depends on design characteristics of the bridge. Construction techniques also contribute to volume change and/or to degree of restraint of concrete mass. Mechanism of transverse cracking is shown in Figure 3.

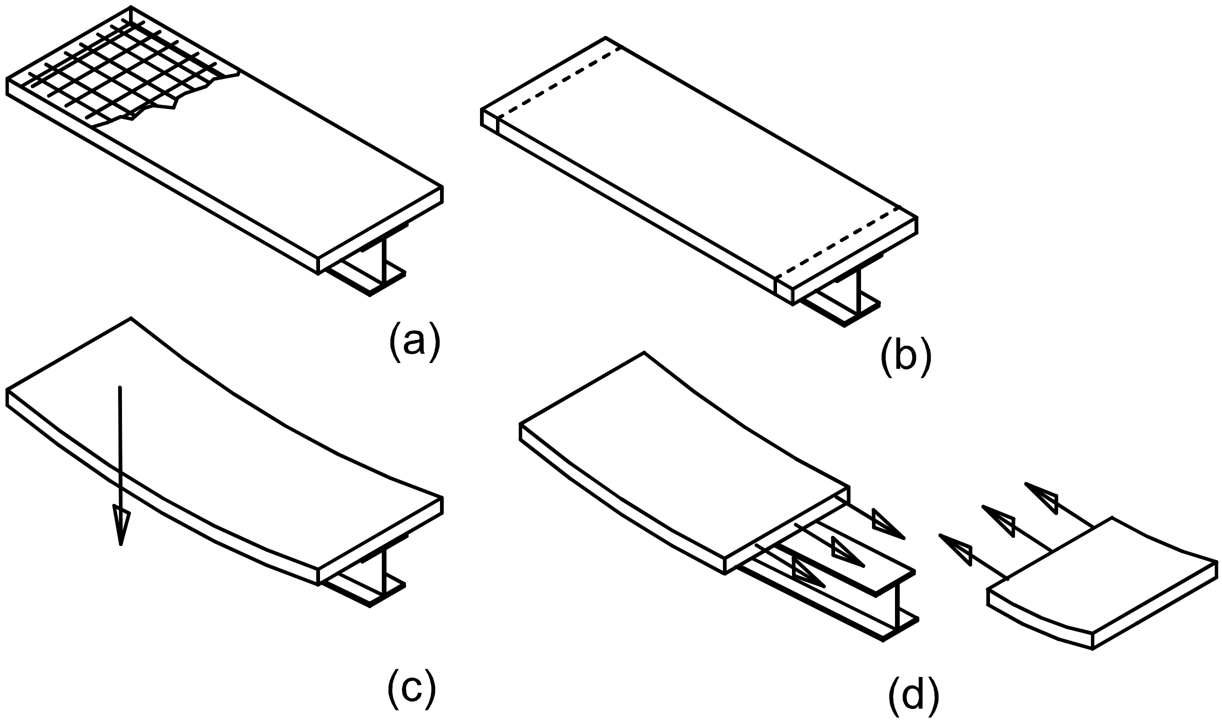


Figure 3. Mechanism of transverse cracking. (a) Concrete is poured. (b) Concrete shrinks, (c) Due to restraint from girder, concrete shrinkage produces downward deflection. (d) Tensile stress is developed in deck, which causes transverse cracks.

Volume change in concrete bridge deck, as influenced by the properties of concrete, is caused by drying-shrinkage, autogenous shrinkage, plastic shrinkage, thermal shrinkage, and creep. Drying shrinkage is change in concrete volume due the change in water content during the time after exposure to atmosphere. Autogenous shrinkage is the change in concrete volume without change in its water content and usually occurs in very low w/c ratios. Plastic shrinkage is referred to shrinkage caused by excessive evaporation of surface water. Cooling of concrete after initial hydration is the cause of thermal shrinkage. Creep strains, on the other hand, tend to counteract the effect of shrinkage. Although relative importance of these factors is not completely quantified yet, drying shrinkage and thermal shrinkage are considered by many studies to be the major cause of concrete deck cracking cracking (Babaei et al., 1997, 1987; French et al., 1999; Krauss and Rogalla, 1996; La Fraugh et al., 1985; PCA, 1970). Earlier studies reported correlation between deck cracking, drying shrinkage (Babaei and Hawkins, 1987; La Fraugh, 1985; PCA, 1970) and higher placement temperatures (PCA, 1970). Babaei and Purvis (*1994*) measured thermal and drying shrinkage of different mixes and showed that mixes with higher thermal and drying shrinkage values tend to crack more. Krauss and Rogalla (1996) through analytical study and further instrumentations of bridge decks showed that drying shrinkage and temperature changes through the section are responsible for deck cracking. In another study, Ducret et al. (1997) have measured that mixes with lower peak hydration temperature produces less residual stress in concrete. In a recent study, Frosch et al. (2002) through field instrumentation and test of constructed deck in laboratory have conclusively shown that drying shrinkage is the most important cause of transverse cracking.

Restraint of deck by girder against its volume change provides the condition for cracking. Ducret et al. (*1997*) have measured that reducing the ratio of cross sectional area of girder to deck reduces risk of cracking. However, relative effect of different factors on the restraint of the composite system is not fully understood yet. Very little, if any, effort has been done to reduce the restraint of deck girder system by changes in design.

Construction practice, such as curing procedures, pouring sequence, and form type can also affect deck cracking. Cady et al. (*1971*), in their study of 249 bridges in Pennsylvania, have shown that the bridge decks constructed by certain contractors have more transverse deck cracking than other decks in the study and concluded that construction practice plays a major role in cracking of concrete bridge decks. Several researchers (e.g. Horn et al., 1975; Kochanski et al., 1990; PCA, 1970) have emphasized effect of curing and weather. Although construction methods may increase or decrease risk of cracking but cracking have been observed on decks built with different construction techniques. Consequently, transverse deck cracking cannot be solely attributed to a certain type of construction technique.

In the following sections, the state-of-the-art on the causes and control of transverse deck cracking are presented under three main categories of material and mix design, construction practice and ambient conditions, and structural design factors.

Material and Mix Design Factors

Aggregate

Type, size, volume and properties of aggregate have pronounced effects on concrete properties. Suggestion on aggregate from prior studies include using largest possible size of aggregate (Babaei and Purvis, 1994; Kosel et al., 1985; Krauss and Rogalla, 1996; PCA, 1970), maximizing aggregate volume (French et al., 1999; Kochanski, 1990; Kosel et al., 1985), and using low shrinkage aggregate (Horn et al., 1972; Krauss and Rogalla, 1996; PCA, 1970) to reduce cracking. To identify low shrinkage aggregate Babaei and Purvis (1994) further recommended that coarse aggregate absorption should not be more that 0.5 percent and the fine aggregate absorption should not be more that 1.5 percent and that aggregates should have high specific gravity. In general, concrete mixes with good quality, clean, low shrinkage aggregate with high aggregate to paste ratio have been observed to perform better.

Water Content

While Horn et al. (1975) noticed little correlation between cracking and water content, Schmitt and Darwin (1999) found increased cracking with increased water content and recommended reducing water content. Other researchers (Babaei and Hawkins, 1985; Babaei and Purvis, 1994; Issa, 1999) have also made similar recommendation. Schmitt and Darwin (1999) further suggested that volume of water and cement should not exceed 27% of total volume of concrete, and Babaei and Purvis (1994) recommended the maximum water content of 192 kg/m^3 (323 lb/yd^3).

Cement Type

Several studies have found that cement type has a significant effect on cracking. It is accepted among researchers that use of type II cement reduces cracking and several studies recommended the use of type II cement in bridge deck construction (Horn et al., 1975; Krauss and Rogalla, 1996; Babaei and Purvis, 1994; Kosel et al., 1985; La Fraugh, 1989). The better performance of type II cement is usually attributed to reduced early thermal gradient and shrinkage of type II cement Figure 4 shows the effect of cement type and source on curing temperature as reported by Babaei and Purvis (1994).

Cement Content

Many studies have observed more cracking with higher amount of cement in the concrete mix (French et al., 1999; Krauss and Rogalla, 1996; La Fraugh, 1989; Iwoa DOT, 1986; Kochanski et al., 1990; Kosel et al., 1985; La Fraugh, 1989; Schmitt and Darwin, 1999). The adverse effect of higher cement content is usually related to higher drying shrinkage, higher temperature rise during hydration and higher early modulus of elasticity of concrete. Different amounts of cement have been recommended as the maximum acceptable cement content in concrete mixes:

- 360 kg/m^3 (611 lb/m^3) (Iwoa DOT, 1986)
- 370 kg/ m^3 (620 lb/yd3) (La Fraugh, 1989)
- 446 kg/ m^3 (725 lb/yd3) (Babaei and Purvis, 1994)
- 385-390 kg/ m^3 (650-660 lb/yd3) (French et al., 1999)

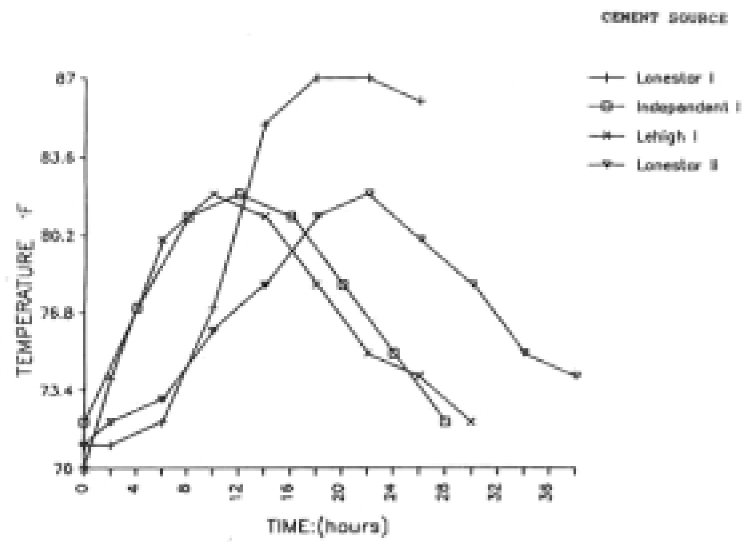


Figure 4. Effect of source and type of concrete on curing temperature (Babaei and Purvis, 1994).

Water/Cement Ratio

Schmitt and Darwin (1999) noticed reduced cracking with reduction in water cement ratio. Indeed other studies also recommend reduction of water cement ratio in concrete mix to reduce cracking (French et al., 1999; Iowa DOT, 1986; Kochanski et al., 1999; PCA, 1970). Reducing water cement ratio of concrete is believed to reduce shrinkage of concrete. However Stewart and Gunderson (1969) found no relationship between high w/c ratio and cracking. The following maximum water cement ratios have been recommended:

- 0.48 (PCA, 1970)
- 0.41 (Iowa DOT, 1986)
- 0.40 (Kochanski et al., 1990)
- 0.40-0.45 (Ramey et al., 1997)

It is also recommended to reduce water cement ratio using water reducers and pozzolans (La Fraugh, 1989).

Concrete Strength

There has been significant increase in concrete strength during the past decades. Increased strength, which is usually accompanied by increase in cement content, increase in paste volume and higher hydration temperatures, is blamed to cause more cracking in concrete decks. In fact, Krauss and Rogalla (1996) related the increase in deck cracking since 1970s to AASHTO's 1973 increase of minimum strength from 3000 psi to 4500 psi and lowering w/c from 0.53 to 0.445. It may seem that high early strength of concrete may reduce cracking but since the strength gain of concrete is usually accompanied by a gain in modulus of elasticity, it can't be easily said that higher strength reduces cracking. There are no general agreements among studies that considered this factor. Schmitt and Darwin (1999) noticed increased cracking with increased compressive strength. However, Ramey et al. (1997) recommended increasing compressive strength. On the other hand, Krauss and Rogalla (1996) recommended the use of concrete with low early strength (i.e. 60 –90 day strength should be specified).

Slump

There are many contradictions in results of the studies performed so far on the effect of slump on deck cracking. Dakhil et al. (1975) in their experimental study reported increased cracking with increasing slump (Figure 5). Some studies have recommended reducing the slump (PCA, 1970; Babaei and Hawkins, 1987; Isaa, 1999; Kosel, 1985; Schmitt and Darwin, 1999) and proposed values for maximum slump:

- 50 + 12 mm (2 + ½ in.) (PCA, 1970)
- 60+12 mm (2 ½ + ½ in.) (Iowa DOT, 1986)

However, Stewart and Gunderson (1969) found no relationship between high slump and cracking, and Krauss and Rogalla (1996) mentioned that there is no relation between slump and cracking tendency. On the other hand, Cheng and Johnson (1985) even noticed a decrease in transverse cracking with an increase in slump.

Air Content

Air content is usually used to increase freeze thaw durability of concrete. But it may be advantageous to use high values of air content in moderate and warm climates. Cheng and Johnson (1985) observed that increase in air content reduces cracking. Schmitt and Darwin (1999) even noticed significant decreases in cracking with air content more than 6% and recommend at least 6% air content. French et al. (1999) recommend air content of 5.5-6%. In contrary, Stewart and Gunderson (1969) found no relationship between air content and cracking.

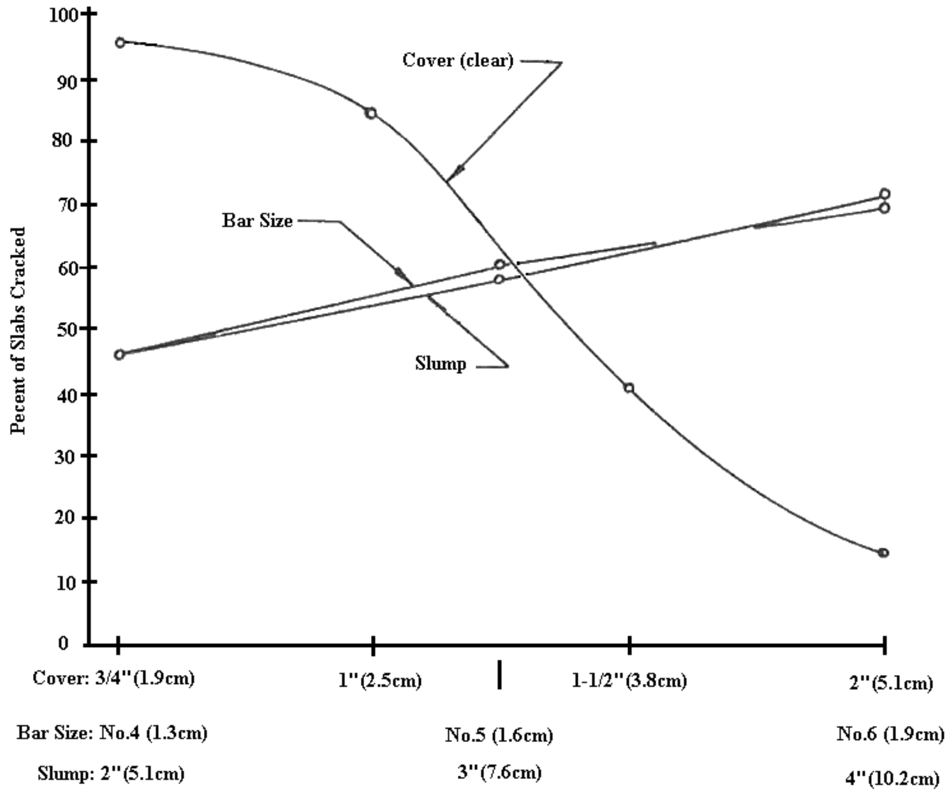


Figure 5. Cracking as a function of bar size, slump and cover (Dakhil et al., 1975).

<u>Admixtures</u>

Effect of different types of admixtures on cracking is not completely understood yet. Cady et al. (1971) reported that the use of retarder is not an important factor. Other researchers (Horn et al., 1975) have reported the same observation. However, some studies (Krauss and Rogalla, 1996; La Fraugh, 1989) encourage the use of retarders, as they believe reduced rate of early temperature rise and early gain of modulus of elasticity would reduce deck cracking. Studies (Krauss and Rogalla, 1996; Schmitt and Darwin, 1999) have shown that use of silica fume may significantly increase cracking if precautions are not taken to prevent plastic cracking. Krauss and Rogalla (1996) also discourage use of calcium chloride and triethanolamine admixtures.

Construction practice and Ambient Condition Factors

Weather Condition and Concrete Temperature

Weather condition during placement of concrete and relative concrete temperature can greatly affect deck cracking. While hot and cold weather conditions may result in poor quality concrete, it is also believed that restraint to thermal variations contributes to cracking (PCA, 1970). Thermal stresses developed at early age in concrete deck depend greatly on concrete temperature and weather conditions. Concrete temperature rises as a result of hydration while girder temperatures remains relatively unchanged. This temperature change will cause thermal stresses in the section. Weather condition and solar radiation may also increase these stresses resulting in more cracking. Studies (Cheng and Johnson, 1985; Mayers, 1982; Schmitt and Darwin, 1999) have shown that hot and cold weather increases cracking and several values for allowable ambient temperatures, and concrete temperature during placement are proposed such as:

- Maximum concrete placement temperature of 27^{o}C (80^{o}F) (PCA, 1970)
- Minimum ambient temperature of 7.2^{o}C (45^{o}F) (Cheng and Johnson, 1985)
- Minimum and maximum ambient temperature of 4 and 32^{o}C (40 and 90^{o}F) and reducing temperature difference between deck and girder. (French et al., 1999)
- Maximum concrete placement temperatures 27^{o}C (80^{o}F) (Krauss and Rogalla, 1996)
- Concrete temperature of at least 5-10^{o}C (10-20^{o}F) cooler that ambient temperature (Krauss and Rogalla, 1996)
- Girder temperature of 12-24^{o}C (55-75^{o}F) should be maintained in clod weather (Babaei and Purvis, 1994)

Some other studies specified the allowable differential temperature of deck and girder, for example:

- Temperature difference of at least 12^{o}C (22^{o}F) for at least 24 hours is recommended by Babaei and Purvis (1994)

Low levels of humidity and high wind speed can also increase cracking. Plastic shrinkage cracks are often attributed to higher evaporation rates than concrete bleeding, where evaporation rates increase with high temperatures, low humidity, and high wind speed. Evaporation rates of concrete under different conditions can be found using evaporation chart. Krauss and Rogalla (1996) recommended that special consideration should be taken when evaporation rates are more that 1.0 kg/m^2/hr (0.2 lb/ft^2/hr) for normal concrete and 0.5 kg/m^2/hr (0.1 lb/ft^2/hr) for low w/c ratio concrete. PCA (1970) recommends testing mixes for bleeding. Kochanski et al. (1990) recommend estimating evaporation rate and reducing it to a maximum of 1.25 kg/m^2/hr (0.25 lb/ft^2/hr).

Curing

Curing has a pronounced effect on the properties of hardened concrete such as durability, and strength. Adequate and timely curing is a key factor in reducing cracking. Importance of cracking is emphasized by almost all studies. Initial fogging (Horn et al., 1975; Stewart et al., 1969), early curing (Horn et al., 1975; PCA, 1970; Babaei and Hawkins, 1987), sprinkling water on concrete surface (PCA, 1970), applying wet burlaps, and applying curing compounds (Stewart et al., 1969) are among the recommendations proposed in literature. Krauss and Rogalla (1996) recommended following procedure for curing:

- Use of fog nozzle water spray in hot weather to cool concrete and to cool the steel and forms immediately ahead of placement – ponding of water on the forms or plastic concrete should not be allowed.
- Use of wind breaks and enclosures when the evaporation rates exceed 1 kg/m2/hr (0.2 lb/ft^2/hr) for normal concrete or 0.5 kg/m^2/hr (0.1 lb/ft^2/hr) for low water cement ratio concretes susceptible to plastic cracking
- Application of water mist or monomolecular film immediately after strike-off or early finishing.
- Application of white-pigmented curing compound as soon as bleed water diminishes.
- Application of prewetted burlap as soon as concrete resist indentation – the burlap must be kept continuously wet by continuous sprinkling or by covering the burlap with plastic sheeting and periodic sprinkling.
- Continuation of wet curing for a minimum of 7 days, preferably 14 days – curing should be extended in cold weather until the concrete has gained adequate strength.

Extended curing time is also suggested by La Fraugh (1989). Kosel and Michols (1985) and Frosh et al. (2002) recommended minimum curing of 7 days for type I and 14 days for type II cement. To reduce temperature, Kochanski et al. (1990) further recommended that decks be covered with permeable membranes rather than impermeable ones.

Pour length and Sequence

Although earlier studies (Cheng and Johnson, 1985; Perfetti et al., 1985) reported that puor length and sequence do not seem to influence cracking, however later studies suggested that pour length, sequence, and rate may have some effects on deck cracking and recommended specifying pouring sequence (Iwoa DOT, 1996) and avoiding pouring irregularities (Horn et al., 1975). Kochanski et al. (1990) recommend pouring concrete at a rate faster than 0.6 span lengths per hour. In his analytical study, Issa (1999) attributes cracking to sequence of pour and recommends placing concrete first in positive moment regions. Ramey et al. (1997) recommend a detailed pouring procedure as follows:

- Place complete deck at one time when possible.
- Place simple span bridges one span per placement or if span is long place divide deck longitudinally and place each stripe at one time. If this cannot be done too, then place the center of span first and then place other portions.
- If multiple placements should be made on continuous beams, place middle spans first and observe 72 h delay between placements. Use bonding agent to enhance bond at joint.

Time of Casting

There is an indication that evening and nighttime casting can reduce the extent of cracking. PCA (1970) recommended nighttime casting. Krauss and Rogalla (1996) also recommended early or mid evening placing.

Finishing

It has been reported that early finishing reduces cracking (Horn et al., 1975; Krauss and Rogalla, 1996). Horn et al. (1975) noticed that hand finishing increases cracking. However, mechanical grooving is recommended by Krauss and Rogalla (1996). Stewart and Gunderson (1969) also found that applying water or grout to concrete surface during finishing operation has adverse effects on cracking.

Revolutions in Concrete Truck

Horn et al. (1975) noticed that excess revolution in truck do not affect cracking.

Vibration of Fresh Concrete

Sufficient vibration of concrete is essential to good concrete. Issa (1999) considers insufficient vibration of fresh a contributing factor in concrete cracking. Horn et al. (1975) noticed that under-vibrated areas tend to develop more cracks, however, over vibration of concrete don’t cause any noticeable effect.

Construction Loads

Effect of traffic and construction loads on deck cracking is also not completely known. Furr and Fuad (1982) found that no deterioration can be attributed to traffic in adjacent lanes during construction, and Manning (1981) showed that good quality plain and reinforced concrete is not adversely affected by jarring and vibrations of low frequency and amplitude during the period of setting and early strength development. However, Issa (1999) attributes cracking to weight and vibration of machinery

Dead load of structure also affects concrete deck cracking (Krauss and Rogalla, 1996). It has been suggested that shoring girders may reduce deck cracking (Babaei and Hawkins, 1987)

Form Type

Inconsistent results have been reported on the effect of form type on deck cracking. Issa (1999) attributes cracking to weight of the forms and their deflection. Based on survey results, Cady et al. (1971) reported that Stay-In-Place (SIP) forms perform better than removable forms. Cheng and Johnson (1985) reported that use of SIP or conventional forms have little effect on transverse deck cracking. While Krauss and Rogalla (1996) have found that SIP forms sometimes increase cracking. Through an experimental study, Frosh et al. (2002) have concluded that additional restraint from stay-in-place forms contribute to cracking and recommended that some other type of forms be used.

Structural Design Factors

It should be noted that research on design factors in general, is very limited. However there are a few studies, which have considered these factors qualitatively. These are discussed in following sections.

Girder Type, Boundary condition, and Spacing

Several studies (Krauss and Rogalla, 1996; PCA, 1970; Cheng and Johnson, 1985; Mayers, 1982; Frosh et al., 2002) have found that decks on steel girders tend to crack more when compared to deck on concrete girders. It is believed that since concrete girders conduct heat slower than steel girders (i.e. lower temperature gradients), thermal stresses in concrete girder bridges are lower than steel girder bridges, consequently, less cracking tendency is expected. Krauss and Rogalla (1996) found that cast in place concrete girders and young prestressed girders have the best performance while deep steel beams have performed worse. They also discouraged design of prestressed composite bridges.

Girder end condition also has pronounced effect on deck cracking (French et al., 1999). Cracking is more prevalent on continuous spans when compared to simple spans (Krauss and Rogalla, 1996; Mayers, 1982; Cady et al., 1971; Cheng and Johnson, 1985), but cracks are observed on both types of spans. There seems to be no significant difference among transverse cracks patterns on different type of structures. Portland Cement Association study (PCA, 1970) indicated that regardless of type of span the same pattern of uniformly spaced cracks are observed on decks supported on steel girder.

Stud Configuration and Properties

Composite action of the deck and girder is basically due to response of shear studs. However, there is no significant effort to reduce deck restraint through changes in stud configuration. Although Krauss and Rogalla (1996) have found that girder restraint and studs cause significant cracking, they don't provide any suggestion on how to reduce girder restraint through change in stud configuration and properties. The only recommendation comes from French et al. (1999) where they have recommended fewer studs with smaller rows and lengths but they don't specify any practical guidelines.

Concrete Cover

Based on their experimental study, Dakhil et al. (1975) reported that concrete cover over reinforcement is the most important factor affecting crack formation (See Figure 5). Increased cover depth reduces risk of cracking, however, excessive increase in cover depth increases probability of settlement cracks over reinforcement. Different values are proposed as the optimum value of the cover depth over top reinforcing bars:

- Minimum of 38 mm. (1.5 in.) (PCA, 1970)
- 88 mm (3.5 in.) (Babaei and Hawkins, 1987)
- 50 mm (2 in.), where deicing chemicals are used use 64 mm (2 ½ in.) and maintain 76 mm (3 in.) limit (Ramey et al., 1997)
- 38 - 76 mm (1.5 - 3 in.)

Contrary to these studies, Meyers (1982) found that decks with cover of 76 mm (3 in) and more seems to be more susceptible to cracking.

Deck Thickness

Increase in deck thickness reduces deck cracking (French et al., 1999; Krauss and Rogalla, 1996; Kochanski et al., 1990; Ramey et al., 1997; Horn et al., 1972; Mayers, 1982). Meyers (1982) observed that bridge decks with deck thickness of 25 cm (10 in.) and more are less susceptible to cracking. Kochanski et al. (1990) recommends deck thickness of 8 ½ to 9 in. French et al. (1999) recommended decks with thickness greater than 16 cm (6 ¼ in.).

Reinforcement Type, Spacing, Size and Distribution

Reinforcing detail is of paramount importance in controlling cracks in concrete structures. Bar size, type, spacing, and distribution affects cracking tendency of concrete decks greatly. Dakhil et al. (1975) in their experimental study reported that cracking increases with an increase in bar size (See Figure 5). Some other studies (Babaei and Hawkins, 1987; Schmitt and Darwin, 1999) also observed the same behavior and recommended limiting the bar size of deck. Since longitudinal bars control deck stresses and reduce cracking tendency, an increase in the amount of longitudinal reinforcement without increasing bar size is another recommendation proposed in some studies (Krauss and Rogalla, 1996; PCA, 1970; Kochanski et al., 1990; Horn et al., 1972; Frosh et al., 2002). Kochanski et al. (1990) as well as Ramey et al. (1997) recommended the maximum top transverse bar size of No.5 and an increase in longitudinal reinforcement. Krauss and Rogalla (1996) recommend use of No. 4 bars with maximum spacing of 15 cm (6 in.). It is also suspected that due to subsidence of fresh concrete over the reinforcing bars and formation of weak plane, deck tends to crack over transverse reinforcing bars. So, French et al. (1999) also recommended limiting transverse bar size and/or maximize transverse bar spacing.

Epoxy coated bars are used to control corrosion of reinforcing bars. Meyers (1982) found that decks with epoxy bars tend to show more cracking. The same finding is reported by Krass and Rogalla (1996) However contrary to their findings a study by Iowa DOT (1986) recommended use of epoxy coated rebars to control cracking.

Details of construction are also important. Horn et al. (1975) noticed that tightly tied reinforcements develop more small cracks initially than loosely tied reinforcements but ultimately cracking was the same. Issa (1999) attributed some cracking to insufficient reinforcing detail at joints between new and old decks

Ramey et al. (1997) suggest following recommendation for reducing deck cracking:

- Limiting the size of deck reinforcement to No. 5
- Reversing lying transverse and longitudinal rebars in the top mat and staggering top and bottom rebars so as not to create significant plane of weakness and using higher percentage of longitudinal steel
- Using ρ =0.002 for top mat longitudinal steel and using the same for bottom mat and trying to use No. 4 bars
- Reducing splices
- Extending deck transverse steel full width

Section Stiffness

Results of the research studies on the effect of section stiffness on deck cracking are a bit confusing. While Babaei and Hawkins (1987) suggested minimizing the flexibility of structure, Ducret et al. (1997) showed in their study that with an increase in the ratio of girder to deck area (which can be related to reducing flexibility) cracking tendency increases. This finding is in agreement with the findings of French et al. (1999) and Krauss and Rogalla (1996) who also reported that reducing deck stiffness reduces cracking.

Since restraint volume change of deck is the principal cause of deck cracking, reducing section stiffness seems to decrease deck cracking, however, this statement needs to be verified with further research.

Vibration and Impact Characteristics

Perfetti et al. (1985) found no relationship between frequency of vibration of superstructure, speed and impact parameters and transverse cracking. However, Babaei and Hawkins (1987) suggested reducing the amplitude and frequency of structure vibration under live load.

Traffic

Although some studies (Krauss and Rogalla, 1996; Stewart et al., 1969; Cady et al., 1971) reported no relationship between daily traffic of bridge and tendency for deck cracking, others (Mc Keel, 1985) observed that bridges that carry fewer trucks at lower speeds exhibit less cracking than those that carry large number of truck at higher speeds.

Research Needs

Despite of the large number of studies on concrete deck cracking, transverse deck cracking is still a problem faced by many transportation agencies worldwide. There are still areas on cause and control of concrete deck cracking that need to be investigated:

Many studies have considered material factors. However, there is no general consensus on the range of different mix design factors (i.e. w/c ratio, cement content, water content, etc.). More research in needed to quantify the acceptable range of different factors.

Although research have shown that restraint of deck due to composite action of deck and girder is the main cause of cracking, there is no significant effort to quantify and/or reduce this restraint through design recommendations. Particularly, effect of shear studs characteristics and section stiffness on deck cracking is almost unknown.

There seems to be more agreement among different researchers on the effect of construction factors, however, due to constant change in construction techniques and introduction of new concrete products like HPC, research is needed to identify appropriate construction procedure for these products.
Standard repair procedures need to be developed and employed by transportation agencies to repair already cracked decks. These procedures can be incorporated into contract specification (Krauss and Rogalla, 1996).

A promising approach to reduce deck cracking is the use of shrinkage compensating cement and/or fiber reinforcement as well as prestressed deck systems (Krauss and Rogalla, 1996). However, more research is required before successful applications of them.

Design tools for evaluation of deck stresses for various geometries, boundary conditions, concrete mix and ambient condition are required (as specified by AASHRO LRFD (1998) article 3.12.4). Such tools may be based on the earlier work of Krauss and Rogalla (1996). One such a tool has been developed in this study and presented in chapter 6.

FIELD STUDY

This chapter presents scope and objectives of the field study and provides the details of data collection, data base development, statistical analysis, and the features of the database. In the following sections, field study is discussed and the methods and sources for different types of data in the database are explained. This chapter also presents the results of field study and the statistical analysis of based on the data, which were collected during the field study. Results and their limitations are presented under three major categories: structural design, material properties and mix design, and construction. In each part, results are compared to similar studies performed by other researchers. These results are followed by discussions of research findings and some interesting observations that were made during the field study

Objectives and Scope

A field study was initiated with the survey of 24 bridges in the state of New Jersey and by collecting design and construction data on these bridges for subsequent statistical analysis (Appendix A contains the data collected for all bridges). In the following section the objectives and scope of the field study are discussed.

Objective

The main objective of these surveys was to identify factors that affect transverse deck cracking in bridges in the state of New Jersey. Another objective of these surveys was the evaluation of current deck mix design and construction practices in the state of New Jersey based on the survey results and the results reported in the literature. Based on this evaluation, recommendations with regard to material and mix design as well as construction practices were made to improve bridge deck performance. Another important objective of this field study was to help to focus the efforts of the second part of the research study by narrowing down the list of important factors that need to be investigated in more details. These are factors that either have not received proper attention during past research, as discussed, or because these factors are related to particular design and/or construction practice unique to the state of New Jersey. As it will be discussed, the surveys and subsequent evaluation of the data do indeed support the initial thrust of the research endeavor with regard to more emphasis on design factors. These factors were investigated in details under this study using analytical and finite element analyses.

Scope

The field surveys included 24 bridges from central and northern New Jersey. 20 of those bridges surveyed were located in the Mercer County while Bergen, Essex, Hunterdon and Monmouth Counties each had one bridge among the bridges surveyed. Figure 6 shows the geographical distribution of the surveyed bridges.

The bridge condition with regard to transverse deck cracking was not know prior to their selection. The only criteria used for selecting these bridges were span length (longer than 85-ft) and age (built after 1994 and considered new). Both prestressed and steel girder bridges were considered. Results showed that the inventory included both cracked and uncracked bridges. However, majority of the bridges, 18 bridges or 75%, were cracked. Table 1 shows the construction year of the bridges included in the survey.

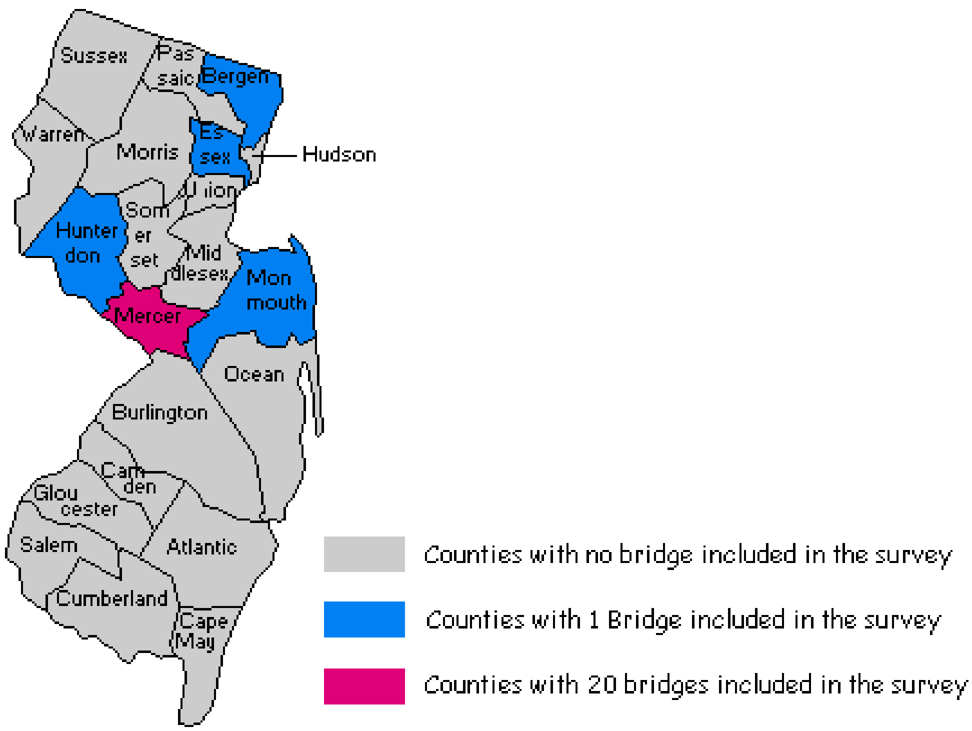


Figure 6. Geographical distribution of bridges surveyed.

Table 1. Construction data for bridges surveyed.

Year Built	**1995**	**1996**	**1997**	**1998**	**1999**	**2000**
No of Bridges	6	2	0	13	2	1

Figure 3.

All bridges considered have composite decks. 8 of them have steel girders while the rest were supported on prestressed concrete girders. The bridges surveyed bridges have various span lengths with the maximum span length of 175 ft. Most of them consisted of 1, 2 or 3 spans (83 percent), but it also included bridges with up to 12 spans.

There were different support conditions among the bridges surveyed. There were 8 spans with simply supported steel girders and 17 continuous multiple spans with steel girders. Considering prestressed concrete girders, 11 spans were simply supported and the rest of the spans (i.e., 43 spans) were continuous multiple spans. Note that continuous spans are continuous in interior spans only and the exterior supports are simply supported.

Except for one bridge, all of the bridges were open to traffic at the time of field survey. Field survey included visual walk-by evaluation mostly of the top of the deck. During the field surveys, bridge decks were visually evaluated and the crack information (crack type, spacing, size and approximate location in the deck) were recorded qualitatively for each bridge. The survey logs also included information regarding bridge location, type, span number, span type, wearing surface and type of bearing. Furthermore, structural plans and construction and mix design information for the bridges were collected from NJDOT and important aspects of design and construction were determined for each bridge. The survey observations and collected data are reported individually for each bridge in appendix A.

Data Sources

There were three major source of data employed in development of the data base that was consequently used for statistical analysis. These are:

- Field survey forms
- Structural plans
- NJDOT Inspection/Testing datasheets

Field Survey Form

Figure 7 shows the visual bridge evaluation form, which was specifically developed for the field survey. Additional information, such as photos of cracks and structural components, were also collected for some of the bridges in conjunction with completing this form, which are all included in Appendix A. However, the crack data and general information about the bridge were two main parts of information collected during the field surveys.

Structural Plans

Structural plans were another source of information. In coordination with NJDOT staff at the Bureau of Bridge Design, structural plans for all bridges were obtained and important design information such as bridge dimensions, deck details, and girder details were extracted.

Date of Evaluation:
Region:
Date Constructed:
Number of Spans:

- ❑ Single
- ❑ Multiple Span
  - ❑ Simply Supported
  - ❑ Continuous

Degree of Skewness (quantify if possible):

- ❑ None (straight)
- ❑ Mild
- ❑ Severe

Type of Beam:

- ❑ Prestressed
- ❑ Steel
- ❑ Others

Deck Surface:

- ❑ Concrete
- ❑ Latex
- ❑ Unknown

Deck Surface Texture:

- ❑ Sawcut
- ❑ Turf Drug
- ❑ None – New Construction

Type of Bearings:

- ❑ Steel Bearings
- ❑ Elastomeric Pads
- ❑ Others:

Pictures/Video Available:

- ❑ Yes
- ❑ No

Additional Note:

Structure Location:

Type of Cracks (use additional forms for > 3 span):

	Span 1	Span 2	Span 3
Transverse			
Longitudinal			
Others			

Location of Cracks within the Deck:

	Span 1	Span 2	Span 3
Center			
End(s)			
Others			

Spacing of Cracks:

	Span 1	Span 2	Span 3
0 – 2’			
3’ – 5’			
> 5’			
Irregular			

Size of Cracks (quantify if possible):

	Span 1	Span 2	Span 3
Hair Line			
Typical			
Wide			

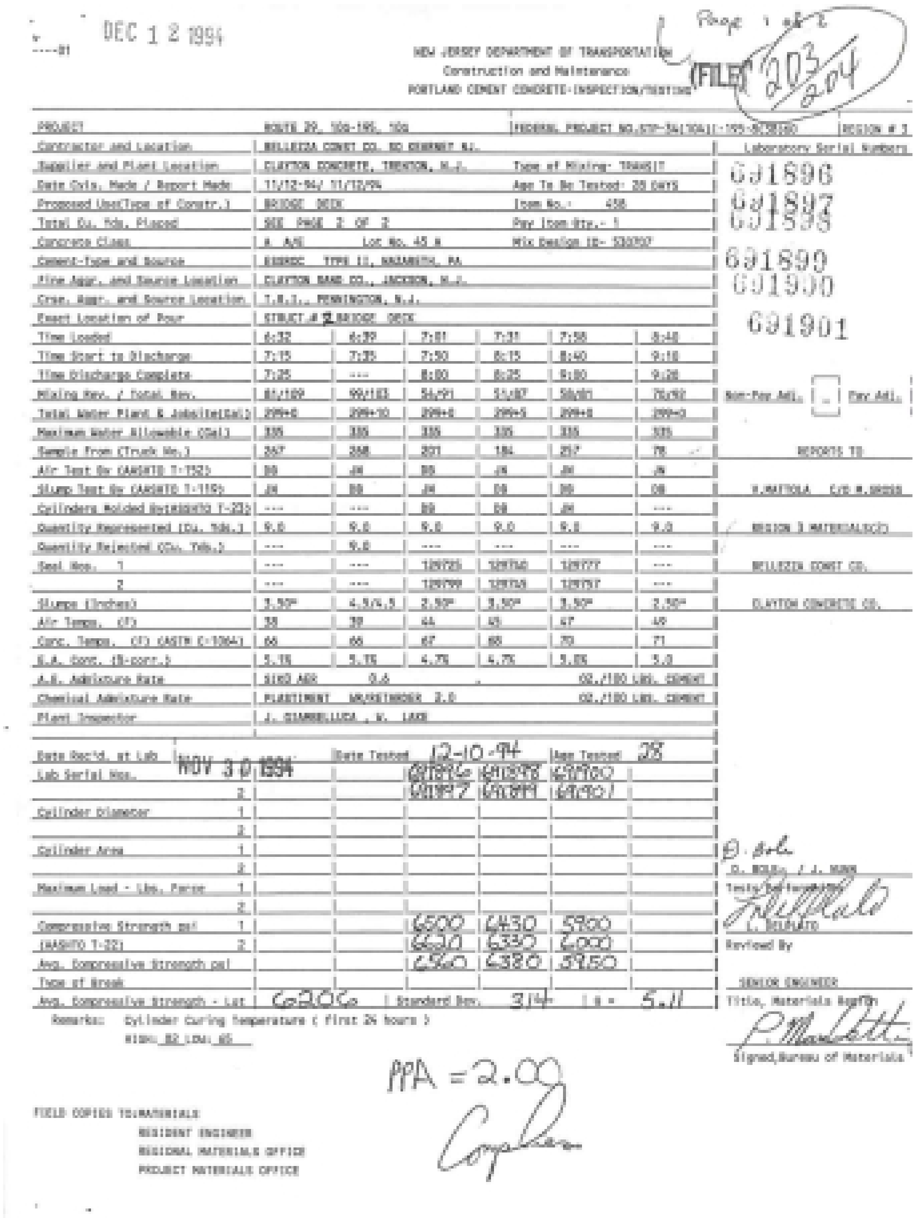
Team Members:

1. 2.
3. 4.

Figure 7. Visual bridge evaluation form.

NJDOT Inspection/Testing Datasheets

These datasheets, which are completed during the construction, contain different data regarding construction and the mix design for each bridge. The results of the strength and slump tests are also reported on these forms. These forms
are part of the documents that NJDOT holds for each bridge. Figure 8 shows a typical datasheet. One part of these forms is completed on the day of casting and the other parts are completed after conducting strength tests. Mix design, strength test results and temperature measurements are three important parts of these datasheets that were used in this study.



DEC 1 2 1994

Page 1 of 2

NEW JERSEY DEPARTMENT OF TRANSPORTATION
Construction and Maintenance
PORTLAND CEMENT CONCRETE-INSPECTION/TESTING

(FILE) 203/204

PROJECT	ROUTE 29, 100-195, 100			FEDERAL PROJECT NO.			REGION # 3
Contractor and Location	BELLEZZA CONST CO. SO KEARNEY N.J.						
Supplier and Plant Location	CLAYTON CONCRETE, TRENTON, N.J.			Type of Mixing- TRANSIT			
Date Cyls. Made / Report Made	11/12/94/ 11/12/94			Age To Be Tested: 28 DAYS			
Proposed Use(Type of Constr.)	BRIDGE DECK			Item No.- 458			
Total Cu. Yds. Placed	SEE PAGE 2 OF 2			Pay Item Qty.- 1			
Concrete Class	A A/E Lot No. 45 A			Mix Design ID- 530707			
Cement-Type and Source	ESSROC TYPE II, NAZARETH, PA						
Fine Aggr. and Source Location	CLAYTON SAND CO., JACKSON, N.J.						
Crse. Aggr. and Source Location	T.R.I., PENNINGTON, N.J.						
Exact Location of Pour	STRUCT.# 2 BRIDGE DECK						
Time Loaded	6:32	6:39	7:01	7:31	7:58	8:40	
Time Start to Discharge	7:15	7:35	7:50	8:15	8:40	9:10	
Time Discharge Complete	7:25	---	8:00	8:25	9:00	9:20	
Mixing Rev. / Total Rev.	81/109	99/123	54/91	51/87	58/81	76/92	
Total Water Plant & Jobsite(Gal)	299+6	299+10	299+6	299+5	299+8	299+3	
Maximum Water Allowable (Gal)	335	335	335	335	335	335	
Sample From (Truck No.)	267	268	201	184	257	78	
Air Test By (AASHTO T-152)	DB	JH	DB	JH	JH	JH	
Slump Test By (AASHTO T-119)	JH	DB	JH	DB	DB	DB	
Cylinders Molded By(AASHTO T-23)	---	---	DB	DB	JH	---	
Quantity Represented (Cu. Yds.)	9.0	9.0	9.0	9.0	9.0	9.0	
Quantity Rejected (Cu. Yds.)	---	9.0	---	---	---	---	
Seal Nos. 1	---	---	[illegible]	[illegible]	[illegible]	---	
2	---	---	[illegible]	[illegible]	[illegible]	---	
Slumps (Inches)	3.50"	4.5/4.5	2.50"	3.50"	3.50"	2.50"	
Air Temps. (F)	38	39	44	45	47	49	
Conc. Temps. (F) (ASTM C-1064)	66	66	67	68	70	71	
E.A. Cont. (%-corr.)	5.75	5.75	4.75	4.75	5.05	5.0	
A.E. Admixture Rate	SIKO AER 0.6				OZ./100 LBS. CEMENT		
Chemical Admixture Rate	PLASTIMENT GR/RETARDER 2.0				OZ./100 LBS. CEMENT		
Plant Inspector	J. GIAMBELLUCA , W. LAKE						
Date Rec'd. at Lab	NOV 3 0 1994	Date Tested	12-10-94		Age Tested 28		
Lab Serial Nos. 1			691896	691898	691900		
2			691897	691899	691901		
Cylinder Diameter 1							
2							
Cylinder Area 1							
2							
Maximum Load - Lbs. Force 1							
2							
Compressive Strength psi 1			6500	6430	5900		
(AASHTO T-22) 2			6620	6330	6000		
Avg. Compressive Strength psi			6560	6380	5950		
Type of Break							
Avg. Compressive Strength - Lot	6206	Standard Dev.	314		s = 5.11		

Laboratory Serial Numbers
691896
691897
691898
691899
691900
691901

Non-Pay Adj. Pay Adj.

REPORTS TO
V.MATTOLA C/O M.GROSS
REGION 3 MATERIALS(C2)
BELLEZZA CONST CO.
CLAYTON CONCRETE CO.

D. BOLS, / J. HUNN
Tests Performed By
L. DELPLATO
Reviewed By
SENIOR ENGINEER
Title, Materials Region
Signed,Bureau of Materials

Remarks: Cylinder Curing Temperature (first 24 hours)
HIGH: 82 LOW: 65

PPA = 2.00

FIELD COPIES TO:MATERIALS
RESIDENT ENGINEER
REGIONAL MATERIALS OFFICE
PROJECT MATERIALS OFFICE

Figure 8. Typical NJDOT Inspection/Testing datasheet.

Database

The information in the database (see Appendix A) is divided into five major categories for each bridge. These are: general information, structural design information, material properties and mix design information, construction information, and crack information.

General Information

This part of the database contains information regarding bridge location and construction year. This information is derived from the survey forms.

Structural Design Information

Using the structural plans, detailed information about the bridge design is extracted. These data include:

- Bridge dimensions
- Number of spans
- Traffic direction
- Girder type
- Span type (e.g., if continuous at interior spans or simply supported)
- Span length and width
- Framing information (e.g., spacing of girders in each span)
- Deck design information (i.e. rebar details, thickness, cover depth and wearing surface)
- Girder properties (e.g., area, depth, and moment of inertial)
- Shear studs spacing

Material Properties and Mix Design Information

This information is derived from NJDOT Inspection/Testing datasheets. Important mix design information like water content, cement content, cement type, admixtures and slump are reflected in the database. Note that since during the construction slump in measured several times, the average value of these tests are taken as the representative value of the slump for the deck concrete during casting. Similarly, average compressive strength of all concrete deck specimens are reported as the compressive strength of the deck.

Construction Information

In this part, based on the NJDOT Inspection/Testing forms the average air temperature during the casting period and the average value of concrete temperature at the time of casting for all the measurement is reported as the representative values.

Crack Information

Crack information, which was recorded during the field surveys, is reported in this part. This information includes: crack type, approximate crack location on the deck, crack spacing and its size.

The database program Microsoft Access (MS Access, 2000) was used to store and organize the data. Microsoft Excel (MS Excel, 2000) program was mostly used for presentation and analysis of the data. These two software packages are also employed for statistical analyses. Charts are extensively used in presenting the result of analyses.

Evaluation of Survey Results

Factors identified relevant to deck cracking are evaluated for all bridges surveyed and are compared with the results of other important researches (for an extensive literature review please refer to chapter two). Results of literature review were used in the selection of these factors. Of course, availability of data was another criterion in determining the factors considered. In the following sections these results are discussed and summarized using tables and graphs. However, the raw data for all bridges are reported in Appendix A.

Structural Design Factors

Design factors included in the survey are detailed in the Appendix. These factors are as follow:

- Girder type
- End Condition
- Skewness
- Type of bearing
- Surface texture
- Wearing surface
- Deck thickness
- Bar size and spacing

Girder Type

Girder type is considered an important factor by various researchers. Some of the pervious works such as NCHRP Report 380 (Krauss and Rogalla,1996) and Minnesota DOT's research (French et al., 1999), show that decks supported on steel girders are more likely to crack than those supported on concrete girders. However, results of this study show that 94 percent of decks supported by prestressed concrete girders were cracked whereas only 38 percent of steel girder bridge decks cracked. The bias in this case is more likely due to the fact that a great number of bridges with prestressed concrete girders had more restraint (fixity) at their supports. This is a very important factor, as it will be discussed throughout this report. Nevertheless, the contradiction highlights the fact that there are other factors that play a significant role that complicate the problem. As it will be discussed, it appears that it is the relative stiffness of the deck with respect to the girder stiffness that is more important than the girder type.

End Condition

This study shows a good correlation between the end fixity and the cracking tendency of the bridge decks. As it is shown in Figure 9, by increasing the fixity of the end supports the percentage of cracked-decks increases. Note that with reference to this figure the continuous end condition refers to situations where the girder is continuous over internal supports. Fixed condition is when the abutment end is fixed (i.e., the end of the girder is built into the abutment wall or integral abutment). The term fixed end should be used with some cautions here. Within the context of these data, this end condition refers to a situation as shown in Figure 10. This type of construction, where the end diaphragm is cast around the girders, is quite rigid. This type of construction is typical construction for bridges built on Route 133. This type of construction is considered simply supported in design stage. As long as the diaphragm is uncracked, i.e., under low level of forces, the connection will act like fixed support. Probably for higher level of forces, such as ultimate, the diaphragm will crack and the girders will behave like simply supported beam as intended. It is difficult to determine the actual rotational stiffness provided by this type of diaphragm. However, under shrinkage stresses, which are much smaller than ultimate stresses, the connection quite likely acts like a fixed support. The effect of end condition will be discussed in more detail later in the report. Nevertheless, widespread cracking of bridges with similar end conditions, which actually prompted this research investigation, supports the pronounced effect of end conditions and rotational rigidity on transverse deck cracking.

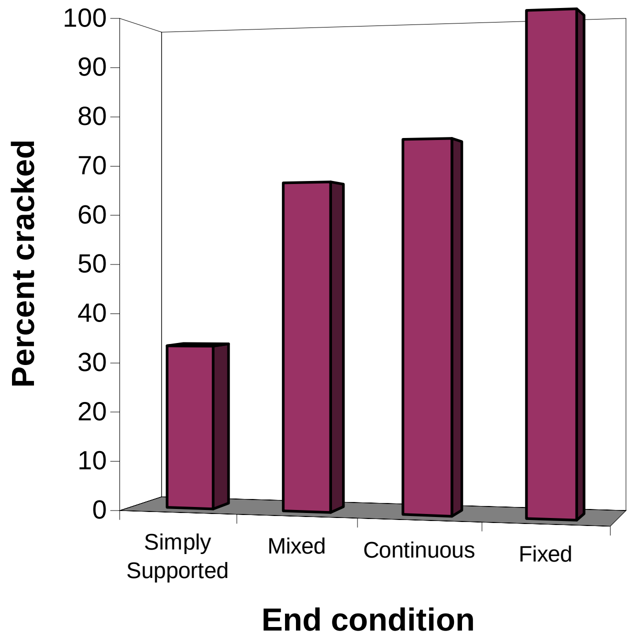


Figure 9. Percent of cracked bridge decks with different end condition.

Figure 10. Boundary condition: girders are bounded.

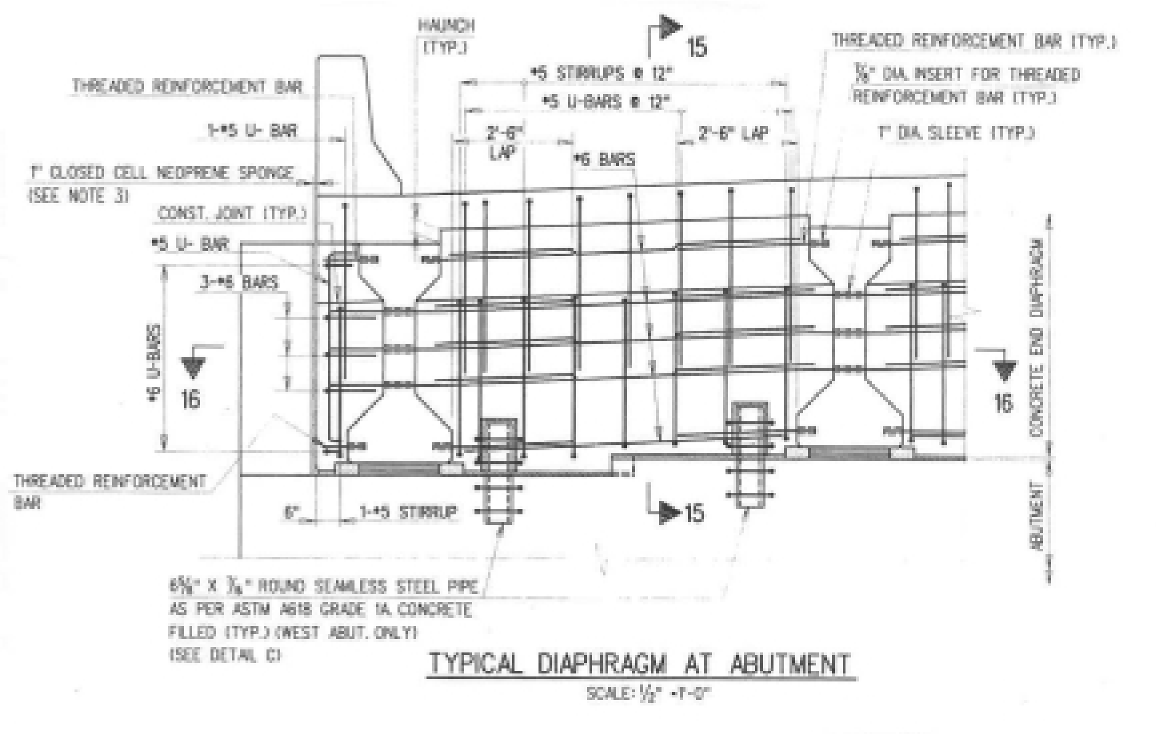


Figure 11. Structural detail of the end condition shown in Figure 10.

NCHRP report 380 (Krauss and Rogalla, 1996), K-Tran (Schmitt and Darwin, 1999) and Minnesota DOT report (French et al., 1999) are among research studies that report increased cracking for fixed girders compared to those with pin-ended girders. Since the number of bridges surveyed is not enough, the effect of the end condition can't be evaluated for different girder types but comparing the number of the continuous steel and prestressed concrete girder bridges shows that 100 percent of prestressed concrete girder bridges cracked whereas only 25 percent of those continuous bridges with steel girder cracked. However, this result is not reliable due to small number of bridges in the sample.

Skewness

There seems to be no direct relationship between the degree of skewness and the potential for transverse deck cracking. Figure 12 shows that the percentage of cracked bridge decks do not follow a consistent trend with respect to skewness. Considering the overall percentage of cracked bridge decks in this study which is equal to 75 percent, it seems that the data on the graph is just some variation with respect to this number which exists in any statistical analysis. Similar result is reported in NCHRP Report 380 (Krauss and Rogalla, 1996).

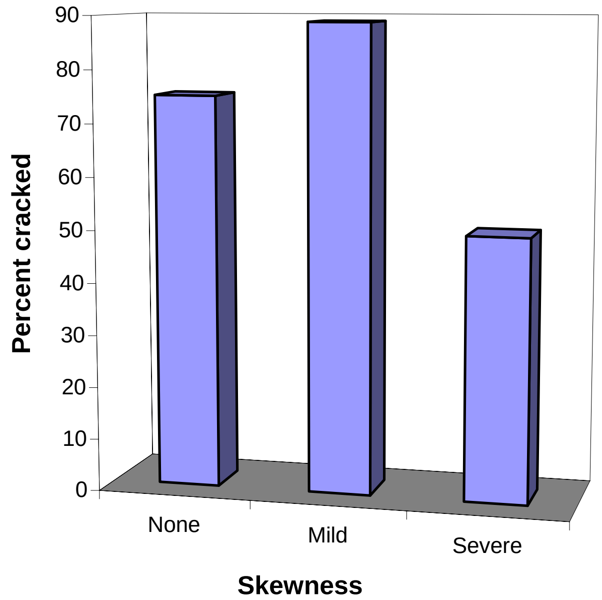


Figure 12. Percent of cracked bridge decks with different skewness.

Type of Bearing

There were two types of bearings in bridges surveyed. Steel bearing and elastomeric pads. Elastomeric pads where only used with girders on route 133 bridges, where as discussed, the end diaphragm where cast around the girders (see Figure 10, 11). The survey shows that all bridges with elastomeric pads are cracked but for steel bearings the percent of cracked bridge decks is 56. This could be due to the fact that probably steel bearings allow rotation more freely. Thus, it seems that the type of bearing may have an affect on transverse deck cracking. However, the result should be viewed with cautions because the elastomeric pads were only used in bridges that also had a different end diaphragm as discussed under end condition.

Surface Texture

Since the dominant texture for bridges surveyed is saw-cut texture and there is only one bridge decks with other type of surface texture, comparison between different textures is not possible.

Wearing Surface

80 percent of the surveyed bridges had concrete wearing surface whereas latex concrete was the wearing surface for the remaining bridge decks. 84 percent of the bridge decks with the concrete wearing surface were cracked while only 20 percent of the bridge decks with the latex concrete surface developed cracks. Thus, there is an indication that latex concrete can reduce the cracking but due to the small number of

bridges in the latex sample (5 bridges), this result should be treated carefully and one cannot draw a general conclusion.

Deck Thickness

The average thickness of the cracked decks was about 8.75 in while this average for the un-cracked bridge decks is around 9 in. This shows that an increase in the deck thickness reduces cracking. Similar results are also reported by other researchers (e.g., Krauss and Rogalla, 1996).

Bar Size and Spacing

All of the bridges surveyed use a mesh of #5 or #6 bars spaced 5 to 7 in as the top mesh (except in the negative moment areas). This study can't identify any significant relationship between the bar size and bar spacing and transverse deck cracking in the range of available data. It is generally accepted that smaller bar size and closer spacing can reduce cracking.

One Dimensional Model of Deck and Girder

Based on the survey results it appears that the total stiffness of bridge and relative stiffness of deck and girder are important factors in transverse deck cracking. To further investigate the effect of bridge stiffness on transverse cracking, a one dimensional finite element model is developed. Structural plan and design sheets for surveyed bridges were also reviewed. The results show that increasing the stiffness of bridge increases the possibility of deck cracking. The details of Finite element study and its results are presented below.

*Objective***:** One-dimensional model is used to evaluate the response of the bridges under service condition. The most important response parameter of interest in this model is the maximum deflection under service condition. A one-dimensional ***linear*** model of one girder and effective portion of deck (AASHTO, 1998) is developed with ANSYS (V5.5, 1998) to investigate service response of bridges. Based on the structural plans of the bridge, this model is dimensioned and analyzed.

*Mesh and Elements***:** Beam elements are used to model the bridge. The span length is divided into 100 portions and each portion is modeled with beam element. Based on structural plans of bridge, composite moment of inertia of girder and effective portion of deck (AASHTO, 1998) are evaluated and specified through out the span length. Figure 13 shows the model.

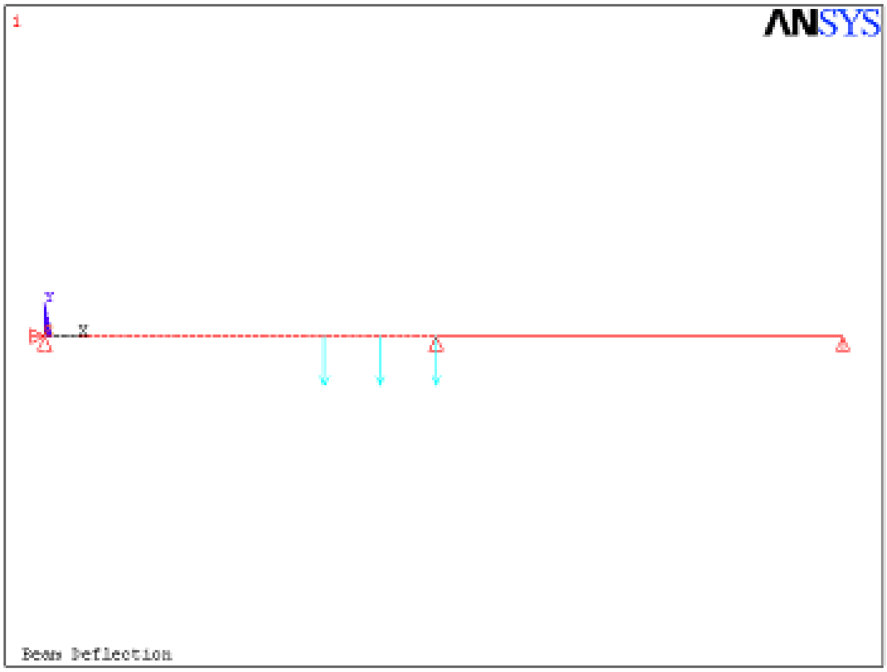


Figure 13. One dimensional finite element model.

Boundary Condition: Based on the bridge being modeled, pin or roller boundary condition is applied on different nodes (Figure 13).

Material Properties: The only material property used in the one-dimensional model is elastic modulus. Elastic modulus of steel and concrete are considered to be 29E6 and 3.83E6 psi respectively.

Loading: Based on AASHTO (LRFD, 1998) provisions, two types of loading should be considered for calculating maximum service load deflection; moving truckload and 25% of moving truckload plus lane load (Figure 14). Based on AASHTO code, distribution factors equal to number of lanes divided by number of girders are multiplied to each loading case.

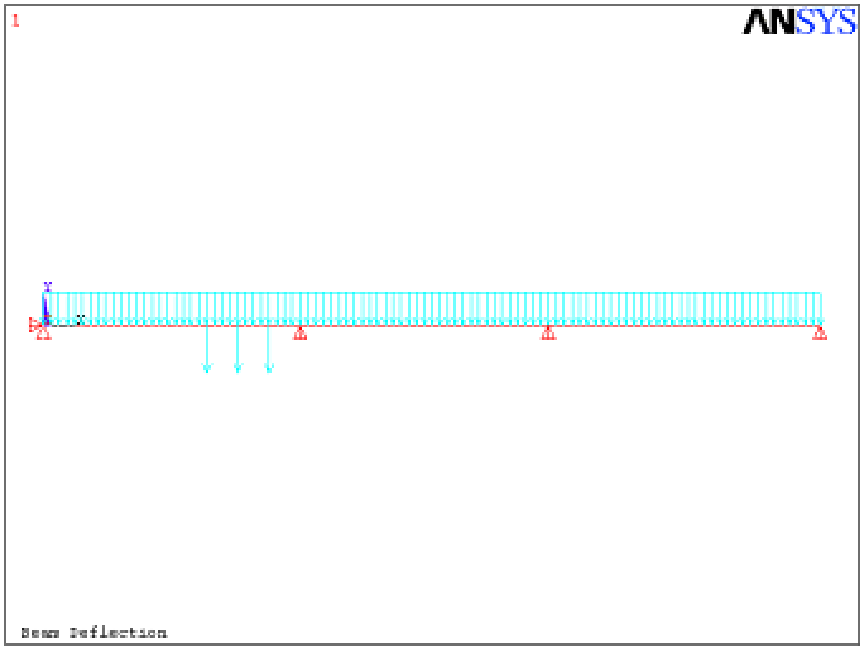


Figure 14. Modeling of traffic loads for 1D model.

The values of truck and lane loads used in analysis are based on AASHTO Articles 3.6.1.2.2 and 3.6.1.2.4. The truckload is moved throughout the span, using birth and death option of ANSYS to simulate moving truck. No dynamic effects are included directly.

One Dimensional Model Results

The result of the one-dimensional analysis is compared to the AASHTO (LRFD, 1998) optional deflection limit of L/800 to study the effect of bridge stiffness on deck cracking. Thirteen bridges among surveyed bridges are modeled and analyzed in this part of study. Where available, the FEA calculated deflections are also compared to the deflections calculated on design sheets at design time of the bridge. Unfortunately the deflection check for all the bridges could not be retrieved, however there is a good agreement between FEA results and calculated values where available. Figure 15 shows a microfilm picture of design sheets reviewed in this study to obtain design deflection values.

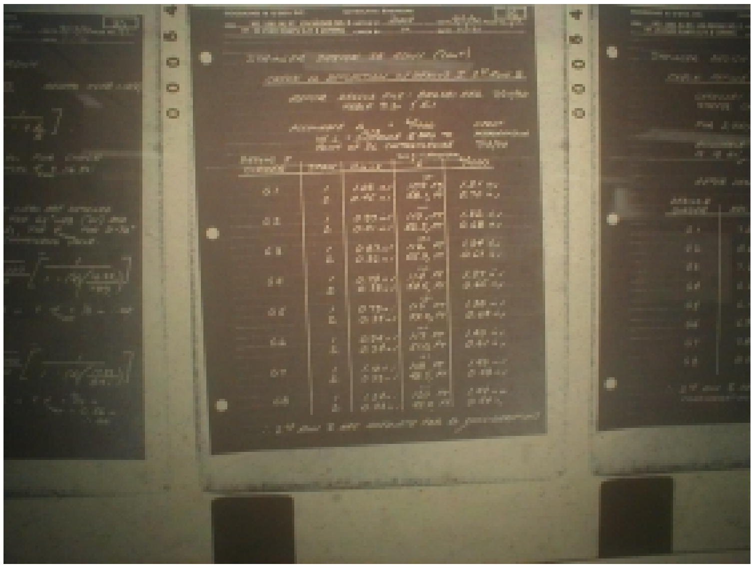


Figure 15. A picture of microfilm of design sheets.

Table 2 shows the summary of the analysis. Based on this table, the average ratio of actual to limit deflections for cracked bridges is 0.09 while this ratio for bridges without is 0.20. This result suggests that as the stiffness of bridge increases, the possibility of cracking also increases. The deflection L/800 limit can be set as benchmark to compare the stiffness of bridges, however since the study is based on a limited number of bridges no particular ratio can be proposed for comparison.

Table 2. Results of 1D FEA

Row No.	Bridge Name	Description	Span Length (mm)	No. of Lanes	No. of Girders	DF
1	ST. # 1130-156 Ramp	Span 1, Simple, G2	16404.19948	1	5	0.2
2	ST. # 1130-156 Ramp	Span 2,3,4, Continuous, G7	46505.91	1	5	0.2
3	ST. #1103-158	Span 1, Simple, G2	31233.60	2	6	0.33
4	ST.#1013-151	Span 1, Simple, S3	40682.4147	2	7	0.29
5	ST.#1136-154	Span 2, Simple, 21S	42322.83465	2	5	0.4
6	ST.#1143-167 &168	Span 1, simple,G2	27279	2	5	0.4
7	ST.#1143-169 &170	Span 1,2,3 Continuous, G2	36576	2	5	0.4
8	ST.#1143-171 &172	Span 1,2 Continuous, G2	32000	3	6	0.5
9	ST#1143-173	Span 1,2 Continuous, G2	24840	2	6	0.33
10	ST#1143-174 &175	Span 1, Simple, G2	27430	2	5	0.4
11	ST#1143-176 & 177	Span 1,2,3, Continuous, G2	40538	2	5	0.4
12	ST#1130-154	Span 1,2 , Continuous, G102	52493.44	3	8	0.375
13	ST#0206-165	Span1, Simple, G112	27482.8	3	7	0.43

Results of 1D FEA (Continued)

Row No.	Ix (Composite) (cm^4)	Truck Δ (mm)	25% Truck + Lane Δ (mm)	Final Δ (mm)	L/800 (mm)	Ratio (actual/Limit)	Bridge Status
1	1029986.27	3.28	1.670	**3.28**	**20.51**	**0.16**	Not Cracked
2	Variable	8.027	5.290	**8.03**	**58.13**	**0.14**	Not Cracked
3	Variable	12.07	8.287	**12.07**	**39.04**	**0.31**	Not Cracked
4	60760442.97	10.64	8.660	**10.64**	**50.85**	**0.21**	Not Cracked
5	Variable	12.3	10.280	**12.30**	**52.90**	**0.23**	Not Cracked
6	3186616.46	2.56	1.480	**2.56**	**34.10**	**0.08**	Cracked
7	6896776.47	5.49	2.910	**5.49**	**45.72**	**0.12**	Cracked
8	5973539.01	4.28	2.260	**4.28**	**40.00**	**0.11**	Cracked
9	3136103.71	2.52	1.170	**2.52**	**31.05**	**0.08**	Cracked
10	4919769.65	1.68	0.980	**1.68**	**34.29**	**0.05**	Cracked
11	7088042.67	4.56	3.880	**4.56**	**50.67**	**0.09**	Cracked
12	Variable	10.96	9.191	**10.96**	**65.62**	**0.17**	Not Cracked
13	Variable	13.66	8.79	**13.66**	**34.35**	**0.40**	Cracked

Material Properties and Mix Design Factors

Based on the NJDOT Inspection/Testing datasheets that contain the information about mix design and construction practice several material properties are extracted and used in the development of the database. The material properties that are recorded in the NJDOT Inspection/Testing datasheets and used in this study are:

- Cement Content
- Water Content
- Water cement ratio
- Air content
- Cement Type
- Slump
- Compressive strength
- Admixture

Note that the range of data for these factors is quite narrow and this fact should be considered in interpreting the results. However, this may be a good thing and supportive of the research thrust on design factors. That is, the data are within a narrow band and most of them well within the recommended range made by other research. Still majority of the bridges have cracked supporting the fact that design factors play a significant role in causing and/or controlling transverse deck cracking.

<u>Cement Content</u>

The cement content in the deck concrete for bridges surveyed is in the range of 611 to 735 lb/yd^3 with most of the bridge decks built with cement content of 700 lb/yd^3 (19 bridges). This study shows that the cracking occurs on both decks with high cement content and low cement content, albeit within that narrow range. 80 percent of decks with cement content of 700 lb/yd^3 and more cracked, whereas 50 percent of decks with lower cement content cracked. As for those 19 bridges with exactly 700 lb/yd^3 cement content 73 percent are cracked. Thus, there is an indication that lower level of cement content reduces cracking but considering the distribution of data (i.e., narrow range and the fact that majority had one cement content) this result cannot be emphasized. Other researchers also reported increased cracking with an increase in cement content. (such as Krauss and Rogalla, 1996; French et al., 1999; Babaei and Purvis, 1996; Shmitt and Darwin, 1999)

Water Content

The water content for the deck concretes is between 31.5 lb/yd^3 and 35 lb/yd^3, where 19 bridge
decks have water content of 31.8 lb/yd^3 and water content for 23 decks is in the range of 31.5 lb/yd^3 to 32.6 lb/yd^3. The ratio of cracked bridges is 77 percent. Previous studies reported that cracking increase with an increase in water content (Krauss and Rogalla, 1996; Shmitt and Darwin, 1999; French et al., 1999; Babaei and Purvis ,1996), but due to the narrow range of data in this part no general conclusion can be made.

Water Cement Ratio

The w/c ratio for the bridges surveyed is between 0.44 and 0.36, where 19 bridges have w/c of 0.38. Again, due to the narrow range of data and their distribution this part is also inconclusive. But it should be noted that the range of the w/c ration in bridges surveyed more or less is in the range recommended by other researchers.

Air Content

The average air content for the bridge decks is in the range of 5.1 to 6.7. There is no indication of the effect of higher or lower air content on deck cracking in the bridges surveyed based on the results.

Cement Type

Type II cement is used in all decks for bridges surveyed. The cement manufacturer is also one company for 92 percent of bridges. Literature also recommends the use of type II cement to minimize transverse deck cracking.

Slump

The average slump of the concrete used in these bridge decks is in the range of 3 to 4 inches. Cracking is observed in decks with both high slump (4 inches) and low slump (3 inches). It seems that for the range of slump observed in these bridges, it has no effect on the transverse deck cracking.

<u>Compressive Strength</u>

The compressive strength of the bridge decks is in the range of 4500 to 6623 psi, which is a broad range. The average compressive strength of un-cracked bridge decks is 5640 psi, whereas this average for cracked bridges is 5730 psi. This shows that cracked bridges have slightly higher compressive strength, which is in agreement with previous studies (e.g., Krauss and Rogalla, 1996). In fact comparing these strengths with the 4500 psi, which is the required strength for design, it is observed that the average compressive strength is about 1200 psi more than that specified in design. *<u>Reducing this margin, which partly means reducing cement content, may reduce the potential for deck cracking.</u>*

<u>Admixture</u>

Water reducer and air entraining agent is used in all bridges surveyed. Also, 79 percent of the deck bridges have retarder agent in their mix. There is no indication of increase or decrease in cracking because of the use of these admixtures.

Construction Techniques and Ambient Condition Factors

All the data that are available and used in the development of the database are those provided on the Inspection/Testing sheets. Unfortunately, no documented data is available about curing method and time of curing, which are among the most important factors with regard to transverse deck cracking. Therefore, it may be a good practice to include more data on construction methods in the inspection and quality control forms. *Among data that may be included are: wind velocity, humidity, curing method, curing period, and placement length*. Note that measurement of humidity is required based on the NJDOT Specs (1998).

Thus, based on the available data, in this study the effect of the following factors are considered:

- Air and concrete temperature
- Month of placement

<u>Air and Concrete Temperature</u>

The study shows that the cracked bridge decks were cast in slightly higher temperatures and with slightly higher concrete temperatures. The average air temperature in the time of casting for cracked bridges is 64^{o}F while this number is 60^{o}F for un-cracked bridges. Also, the average concrete temperature at the time of casting for un-cracked bridges is 76^{o}F, while this number is 73^{o}F for un-cracked bridges.

Literature indicates (e.g., Babaei and Purvis, 1996) that placement of bridge decks in very high and very cold weather increases the possibility of cracking. However, the data show that none of these 24 bridges surveyed were cast in very hot or very cold weather.

Month of Placement

As it was just mentioned, literature indicates that deck placement in very hot and very cold weather increases transverse cracking. The month of placement can be a good indicator of this situation. The decks for none of the bridges surveyed were cast in the winter. Construction season for the bridges surveyed are as follow: 37.5 percent in the spring, 33.3 percent in the summer and 29.2 percent in the fall. 43 percent of bridge decks cast in the fall developed cracking, while this number is 75 and 89 for the bridges cast in the months of summer and spring, respectively. This can indicates that casting the decks in mild weathers can reduce the potential for deck cracking.

Remarks

Review of the data collected and comparison with the literature and with the 1998 NJDOT Specs (1998) show that many material and mix design recommendations are already satisfied. Due to the limitation of the data such a conclusion cannot be made with regard to construction factors. For example, it could not be determined if 7 day wet curing is employed. However, consistent with the literature recommendation, NJDOT Specs (1998) does require 7 day wet curing. This is a very important factor in controlling transverse deck cracking and every effort must be made to adhere to the Specifications. In summary, NJDOT Specs (1998) contains many of the recommendations made as results of prior research. There are also some factors are consistent with research recommendation. For example, none of the bridges surveyed were cast in the winter and all surveyed bridge decks had relatively low w/c ratio.

With regard to design and material property factors, higher compressive strength of the tested specimens compared to design value shows that the structural design requirement on the compressive strength could have been satisfied by the use of lower cement in the mix. Reducing the cement content in turn can reduce drying shrinkage, thermal shrinkage and temperature differentials during casting, which are believed to be the dominant causes of transverse cracking. Therefore, it is strongly recommended to reduce the cement content. Considering the design compressive strength of 4500 psi for bridge decks, this recommendation can still be satisfied quite easily. If the current guideline provides any incentive for higher than design compressive strength it should be revoked. It is also noticeable that except for high amount of cement content, other parameters included in the study satisfies recommendations made in the literature. Use of type II cement, adequate air content (>5%), satisfactory w/c ratio (<4.5) and use of the water reducer and retarder agents are indications of good mix design as required by literature to reduce transverse cracking.

Despite significant research work and enhanced knowledge on the effect of mix design and construction practice on concrete deck cracking, the current knowledge on the effects of structural design factors on deck cracking is limited. The design recommendations proposed in the literature are mostly based on engineering judgment and have not been quantified in details through analytical and/or experimental studies. For example, it is accepted that reducing deck restraint or increasing deck thickness can reduce the possibility of deck cracking, however, no specific guidelines and/or values have been recommended. The observations of bridges on route 133 (see Figure 10, and 11), as mentioned in section 3.7, suggest that the design factors can be important in deck cracking. Note that the end diaphragms on all of these bridges are cast around the girders. As shown in Figure 3.16, the cracks in these bridges are parallel to the bridge longitudinal axis and become normal to the axis as they propagate towards the center of the span. Considering the not so common design of the end diaphragms and its high rigidity compared to a pinned case, the question arises about the possible contribution of design factors to significantly increase transverse deck cracking. On the other hand, the same factors can be employed in a balanced design to provide remedies.

Figure 16. Rt. 133 bridge deck crack.

The observations made from the field survey of two parallel bridges crossing the Watson Creek (EB and WB) further support the need for more knowledge on the effects of structural design factors. These two bridges are almost identical and built on EB and WB side of route I-195. Structural designs as well as the mix designs for these two bridges are very similar to each other (see appendix A). During the surveys transverse cracks were observed on the eastbound side but the westbound side had no transverse deck cracking. Examination of the joint continuity for these two bridges showed that the joints on the westbound side were cracked, while the continuity joints on eastbound side were intact (see Figure 17 and 18). That is, the bridge with cracked joint (less rigidity/constraint) did not develop any cracking in the deck, while the one with uncracked joint (more rigid/restraint) did develop transverse deck cracking. These observations point to the role of structural design factors in deck cracking and the need for further research to enhance or knowledge on these factors.

Figure 17. Watson Creek Bridge (West Bound): Cracked continuity joints (bridge No.1130-152).

Figure 18. Watson Creek Bridge (East Bound): Un-cracked continuity joints (bridge No.1130-153).

CAUSES OF VOLUME CHANGE IN CONCRETE AND RESTRAINED SHRINKAGE TESTS

This chapter presents an overview of the causes of volume change in concrete relevant to bridge deck cracking. Effects of four types of shrinkage, ambient temperature changes, and creep are examined. Laboratory tests currently utilized to measure the cracking tendency of restrained concrete are introduced. The chapter concludes with a summary.

Volume Change and Cracking in Concrete

Concrete cracks as a result of *restraint volume change* of concrete and this section describes the basic mechanisms of the volume change in concrete. There are seven basic causes of volume change in bridge deck concrete relevant to deck cracking; drying shrinkage, autogenous shrinkage, plastic shrinkage, thermal contraction, creep, ambient temperature changes, and solar radiation.

Restraint of concrete during volume change causes cracking. Effect of restraint on bridge decks will be discussed comprehensively in the following chapters. As mentioned previously, this project focuses on alleviating the deck-cracking problem by reducing restraint of concrete deck through changes in design. It should be emphasized again that although the authors recognize the key role and importance of reducing volume change in concrete (as described herein) but they also believe that, as a safeguard, current design of bridge decks can be improved to accommodate volume change of concrete with less cracking.

Drying Shrinkage

Drying shrinkage is the result of water loss of hardened concrete. Volume of concrete reduces as water withdraws from concrete. However, only part of (40-70 percent) the shrinkage is recoverable with future wetting cycles.

Drying shrinkage depends on concrete properties. It is strongly influenced by w/c ratio, volume to surface ratio and cement content. Both ACI (Committee 209, 1978) and AASHTO (LRFD, 1998) codes provide equations for estimating drying shrinkage values as a function of time. These equations do not consider the effect of cement type and its composition. Where, as experimental results show, shrinkage strains are significantly influenced by cement properties (Burrows, 1998).

Drying shrinkage in a restraint concrete can produce significant tensile stresses, which may result in cracking. Figure 19 shows a graph of drying shrinkage strains vs. time for different volume to surface ratios.

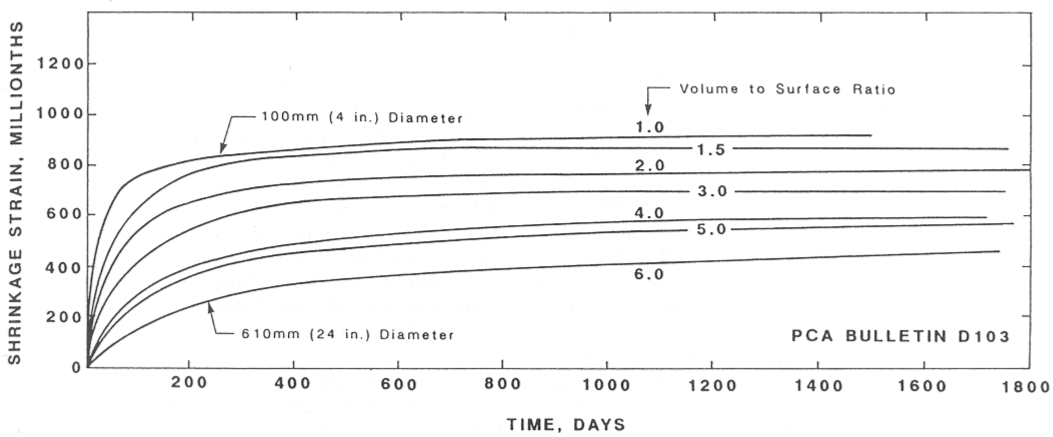


Figure 19. Drying shrinkage of concrete.

ACI (Committee 209, 1978) recommends the following equation for predicting drying shrinkage of concrete.

$$\varepsilon_{sh,t} = \frac{ti}{ti+\alpha}\varepsilon_{sh,u} \tag{1}$$

Where: $\alpha =$ 35 for moist cured concrete and 55 for steam cured concrete
$ti =$ Time in days after measured after curing
$\varepsilon_{sh,u} =$ Ultimate shrinkage=780E-6 in/in in standard condition

The value of the $\varepsilon_{sh,u}$ should be multiplied by following factors when other than standard condition exist:

For other than standard humidity a correction factor should be applies to equation.

For 40<H<80 Percent $\quad K_H = 1.40 - 0.010H \quad$ (2)

For 80<H<100 $\quad K_H = 3.00 - 0.30H \quad$ (3)

For curing period other than 7 days for moist cured concrete and 3 days for steam cured concrete multipliers in Table 3 should be used. For v/s ratios (Volume to surface) other than 1.5" correction factor K_{vs} should be employed.

$$K_{vs} = 1.20\exp(-0.12\frac{v}{s}) < 0.2 \tag{4}$$

Table 3. Shrinkage correction factors for initial moist curing (AASHTO).

Moist Curing Duration (Days)	Factor
1	1.2
3	1.1
7	1.0
14	0.93
28	0.86
90	0.75

ACI (Committee 209, 1978) also suggests a second method for computing K_{vs} where:

$$K_{vs} = 1.23 - 0.038t \quad t>6" \qquad (5)$$

Where t is deck thickness. This equation is valid only during the first year of curing, which is the concern of this study.

There are other factors for Slump, Cement Content, Air Content and Temperature. But ACI (Committee 209, 1978) states that for slump less than 5", fine aggregate between 40 to 60 percent, cement content between 470 and 750 lb/yd^3, and air content less than 8 percent these factors are approximately equal to unity.

<u>Plastic Shrinkage and Plastic Settlement</u>

Plastic shrinkage and plastic settlement occur in plastic concrete. Plastic shrinkage is the result of excessive evaporation of water from concrete before hardening. The rapid evaporation of water from surface layer of concrete and the subsequent contraction of this layer causes tensile stress due to restraint from wet concrete layers below. This may result in plastic shrinkage cracks. Plastic settlement is caused by uneven settlement of fresh concrete over the obstructions like reinforcement (Figure 20).

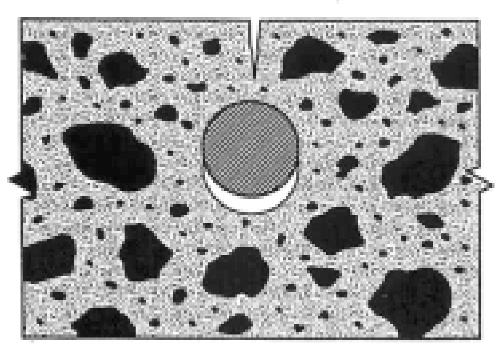


Figure 20. Crack in plastic concrete due to uneven settlement.

Prevention of plastic shrinkage and plastic settlement cracks is well understood. Plastic shrinkage is controlled by controlling the rate of evaporation of water from concrete. The rate of water evaporation basically depends on wind velocity, air temperature, relative humidity, concrete temperature, cement content, and aggregate size. ACI monograph (Committee 308, 1986) can be used to estimate the rate of evaporation (Fig 24). Special curing procedure should be applied when evaporation rate exceeds certain limits.

Plastic settlement cracks can be eliminated by use of smaller reinforcements, and adequate and timely vibration of concrete. It is believed that the plastic settlement in decks produces weak planes over reinforcement, which may later crack by other effects.

Autogenous Shrinkage

Autogenous shrinkage is the concrete shrinkage without loss of water. This kind of shrinkage occurs at low w/c ratios and significantly increases with use of silica fume, HRWRAs (High Range Water Reducing Admixtures) and finer cement. In the past, this type of shrinkage was insignificant. However, with the downward trend of w/c ratio in concrete mixes, use of silica fume, finer cements, and widespread use of HRWRAs this type of shrinkage came into attention. Stresses produced by this type of shrinkage can add up to locked in stresses due to thermal contraction, drying shrinkage and ambient effects and may cause cracking. Paillere et al. (1989) measured the tensile stress produced in a test specimen as a result of autogenous shrinkage. They observed increasing stresses in concrete mixes with the same amount of cement (716 lb) but different w/c ratios and silica fume. Figure 21 shows their result.

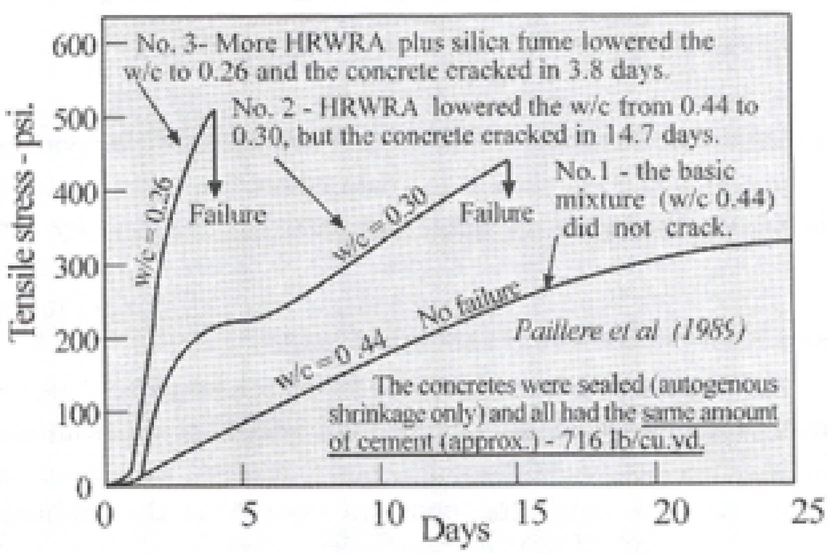


Figure 21. Effect of w/c and silica fume on tensile stress produced by autogenous shrinkage of concrete (Paillere et al., 1989).

Tazawa et al. (1994 and 1995) have clearly shown that autogenous shrinkage increases with lower w/c ratios, higher cement fineness, silica fume and use of HRWRA (Figure 22).

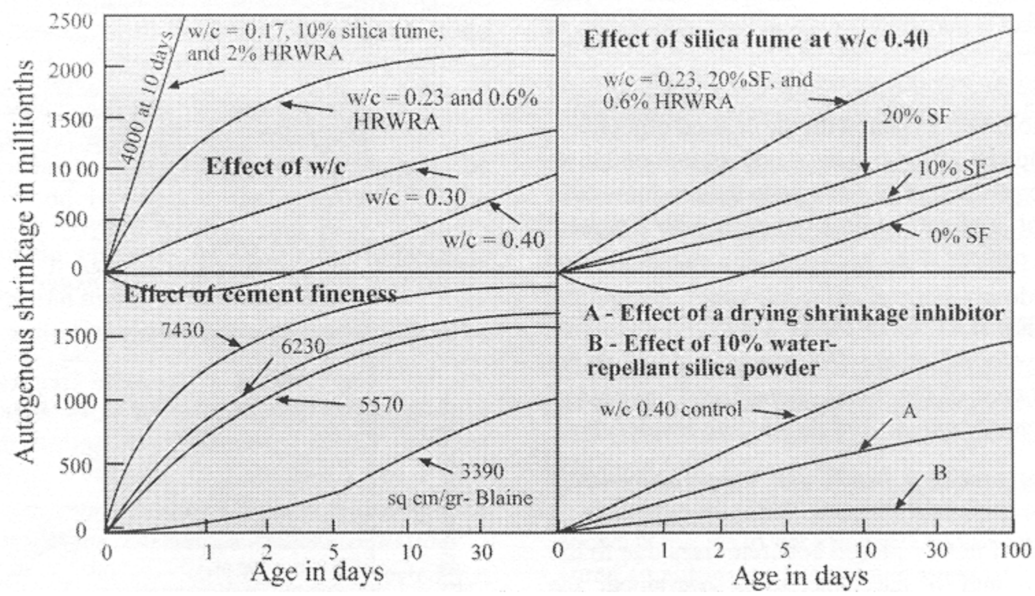


Figure 22. Effect of w/c, cement fineness, silica fume and admixtures on autogenous shrinkage (Tazawa et al., 1995).

Thermal Contraction

Concrete temperature rises after placement due to hydration. If the concrete is restraint, subsequent cooling and increase in modulus of elasticity produces tensile stresses in concrete and may cause cracking a few days after placement. It is also possible that these locked in stresses add up to stresses produced by other causes.

It has been found (Sprinegnschmid et al., 1994) that selecting low heat cement may not completely solve the problem. Because it ignores the effect of rate of increase in modulus of elasticity, and tensile strength. Therefore, a better approach is required for selecting appropriate cement. The test frame developed for thermal contraction tests (Sprinegnschmid et al., 1994) are described in the next section as a possible alternative.

Cement and its chemical composition have strong effect on magnitude of thermal contraction. Based on the work of Sprinegnschmid and Breitenbucher (1990), for a low thermal cracking tendency, cement should have low alkali content, be course grounded, and have high sulfate content.

Lower placement temperatures, higher air content, (Breitenbucher and Mangold, 1994) and slower cooling rates (Chui and Dilger, 1993) also reduces the thermal cracking tendency

Ambient Temperature Changes and Solar Radiation

Daily temperature variations and solar radiation produces temperature differential throughout the section. Stresses produced by this temperature differential add up to the locked in stresses from different types of shrinkage. Similarly, seasonal variation of temperature can produce internal stresses.

Figure 23 shows the temperature distribution across the depth of a composite beam obtained using a finite element analysis for hot weather (Emanuel and Hulsey, 1978). Similar results are also available for cold weather. AASHTO (LRFD, 1998) has recommendation for estimating temperature differentials in the deck-girder section for design purposes.

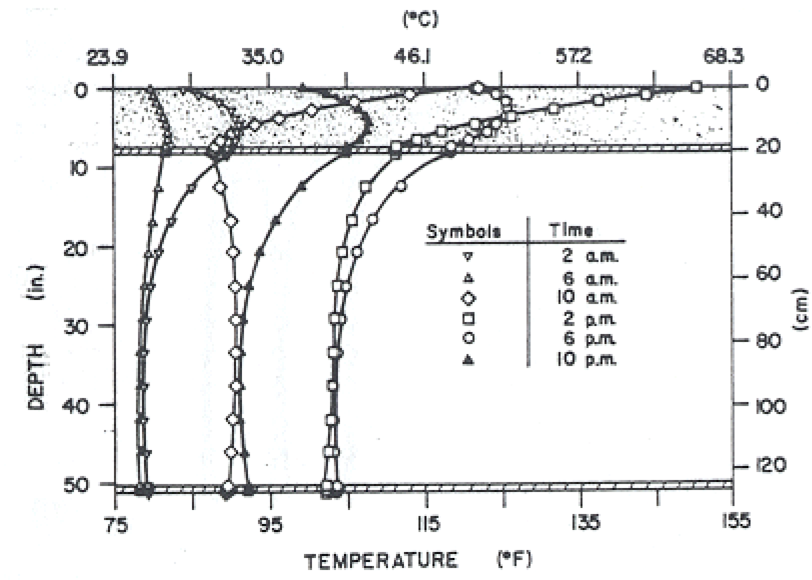


Figure 23. Temperature distribution across the section for hot weather (Emanuel and Hulsey, 1978).

Creep

Creep is the property of concrete to deform with time under sustained stresses. Creep reduces stresses from sustained stresses, thus, reduces deck cracking. So, concrete with high creep tendency, especially during first few months after casting is desirable. High w/c ratio, low strength, and soft aggregates produce concrete with high creep. ACI (Committee 209, 1978) and AASHTO (LRFD, 1998) have equations for estimating creep strains as a function of time.

Bridge Deck Cracking

Magnitude and time variation of different factors mentioned above determines the performance of a bridge deck vis-à-vis of cracking. Burrow (1998) describes the causes of 52 cracked bridge decks out of 100 surveyed by Krauss and Rogalla (1996) as follows:

- “22 cracked during first week: The cause must be thermal contraction plus (if w/c is below 0.5) autogenous shrinkage
- 6 cracked in next six weeks: It is too late for thermal contraction and too early for drying shrinkage. Therefore it must be autogenous shrinkage adding to the locked in stresses from thermal contraction.
- 8 cracked with in a year: Probably stresses from drying shrinkage adding to the locked in stresses from thermal contraction and autogenous shrinkage that have not been relieved by creep.
- 16 cracked after one year: Drying shrinkage is taking its toll.”

Restrained Shrinkage Tests

Restraint shrinkage tests evaluate the cracking resistance of cement or concrete. Any cement or concrete can be evaluated by comparing the test values against an established evaluation criterion. These tests have been used in many countries such as France, Canada, Japan, and Germany to control concrete cracking, but very few have been used in the U.S. Several testing devices with different dimensions, configuration and objectives have been developed so far. However, all of these devices pursue the common goal of classifying cements/concretes based on their cracking tendency. These devices fall into two main categories:

Ring Tests

Ring test was invented by Roy Carlson (1942) of MIT to measure cracking resistance to shrinkage. In this test cement mortar or concrete is cast around a steel ring and the time to occurrence of first crack is measured. The elapsed time untill occurrence of first crack is used as a measure of cracking tendency of cement. Based on the purpose of the test, curing and temperature changes may be introduced. Figure 24 shows different sizes of this device used by different researches. Figure 25 show a picture of the ring used by Krauss and Rogalla (1996)

Different methods are used to record cracking time. Visual inspection is the most basic method. Blaine (1953) applied a conducting paint to the ring and recorded the crack time by measuring the voltage change applied to the paint. Krauss and Rogalla (1996) installed strain gages on the inside of the steel tube and captured cracking time as sudden change in strain readings. Nowadays, different technologies are available for capturing cracking time.

Figure 26 shows the recommended criteria based on Blaine’s work for crack resistant cement (Burrows, 1998).

Ring tests provide valuable information on cracking tendency. However, there are two drawbacks. Ring tests do not fully restrain the cement mortar or concrete due to friction at the interface of cement and steel. Furthermore, there is no direct method of measuring stresses in concrete.

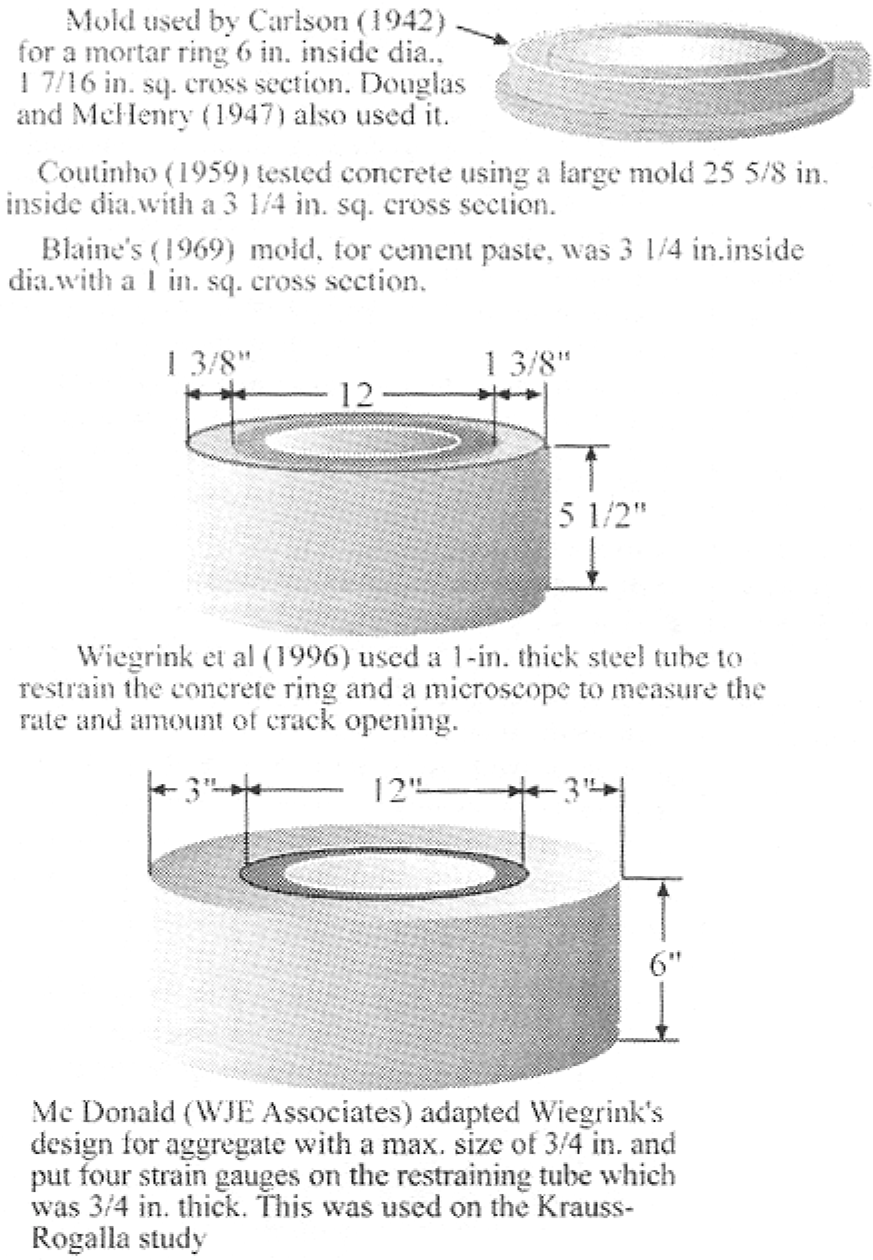


Figure 24. Ring test apparatus used by different researchers (Burrows, 1998).

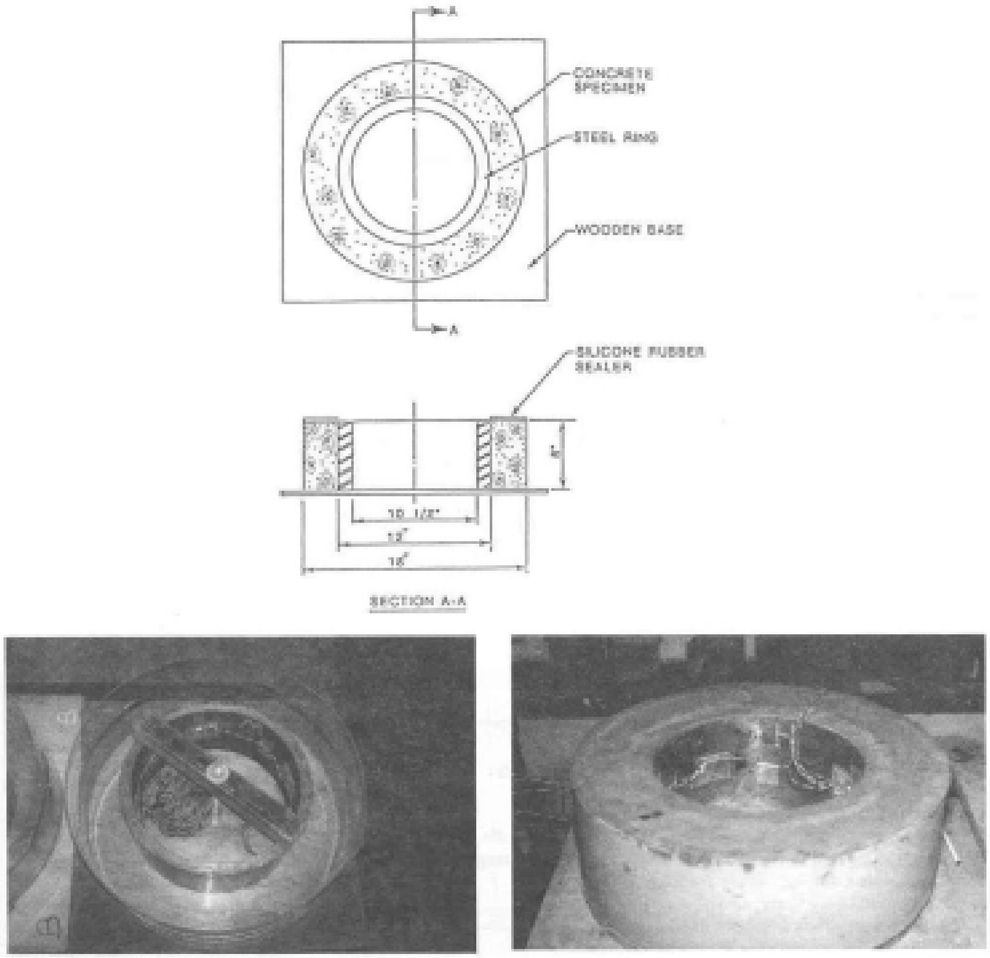


Figure 25. Ring test apparatus used by Krauss and Rogalla (1996).

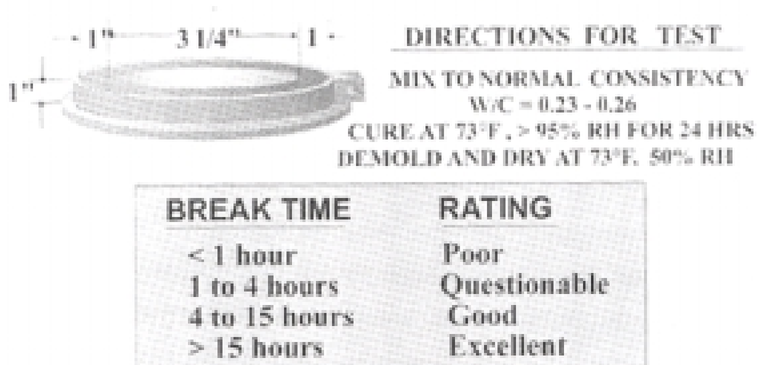


Figure 26. Cement rating criteria based on Blaine's work (Burrows, 1998).

Bar Tests

In this test concrete or cement mortar is cast in the shape of a bar and is hold by two grips at both ends to create restrained condition. The cracking tendency is expressed in terms of level of tensile stress in the specimens as a result of shrinkage. Figure 27 shows the apparatus used in these tests. In a new version of this test, which was developed in Germany two invar bars are used to connect the grips together. In this frame like apparatus, stress is measured indirectly by measuring stress in the invar bars (Figure 28). Using this frame, a test method has been developed in the University of Munich (Sprinegnschmid et al., 1994) to investigate thermal contraction. In this method, without the application of any external heat or cooling, concrete is cooled down to ambient temperature during 4 days. If the concrete has not cracked during 4 days, it is then cooled down artificially at a rate of 1.8 F per hour. The specific temperature at which cracking occurs is defined as cracking temperature. Figure 29 shows typical results of this test.

Bar Tests can be used to measure thermal contraction, autogenous shrinkage and drying shrinkage in a more realistic manner than ring tests. Through the use of controlled environment and specimen size, it is also possible to separate different types of shrinkage more easily and measure them independently.

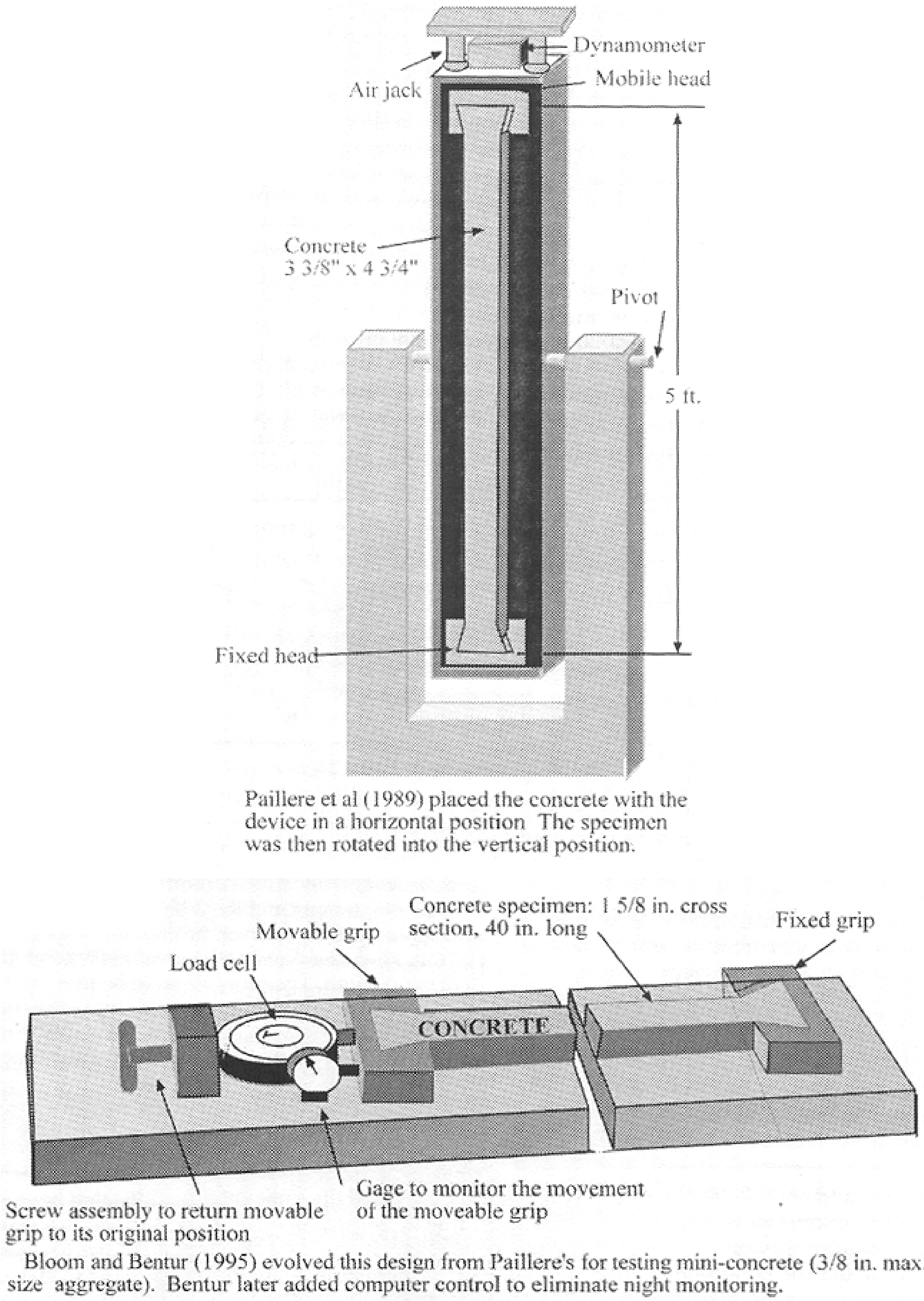


Figure 27. Bar test for studying cracking tendency of concrete (from Burrows, 1998).

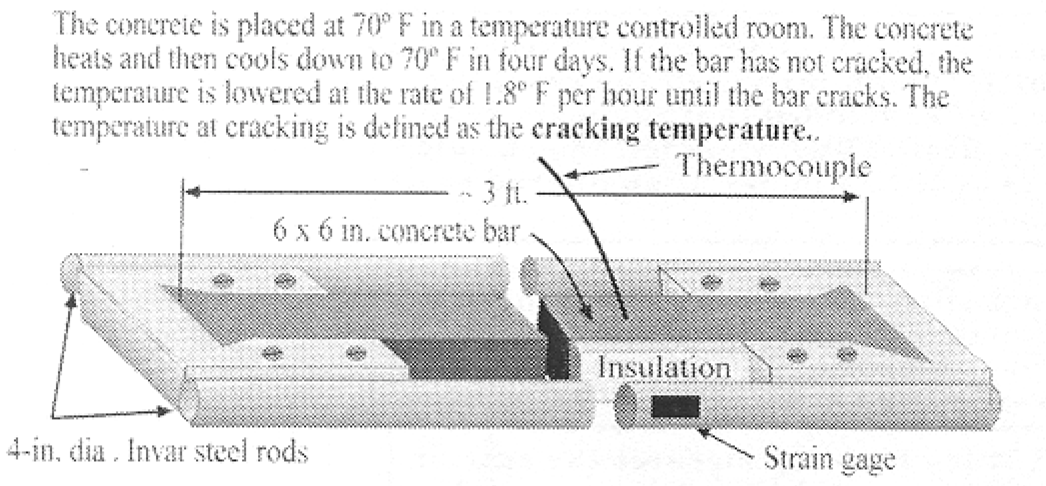


Figure 28. Cracking frame developed in Germany (from Burrows, 1998).

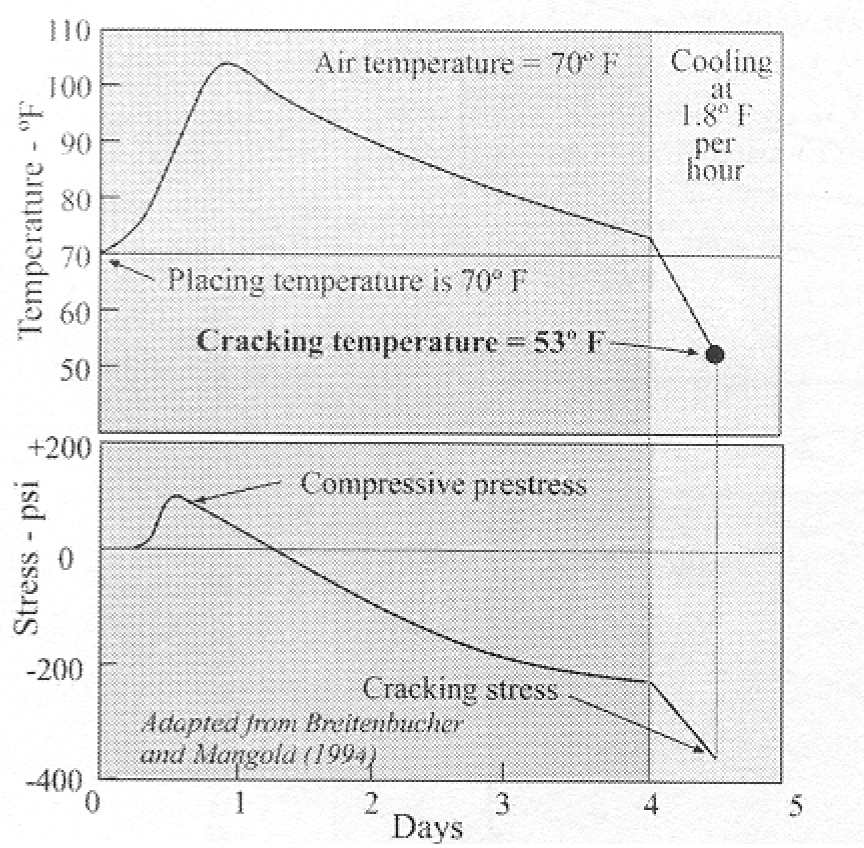


Figure 29. Typical results of cracking frame test (from Burrows, 1998).

FINITE ELEMENT ANALYSIS

This chapter describes development of several finite element (FE) models to study different aspects of transverse deck cracking and presents results of the analyses. Objectives and details of implementation for each model are presented. Effect of different design factors on stresses causing transverse cracking is studied using a 2D FE model. A nonlinear 3D FE model is used to model cracking and study crack patterns on concrete bridge deck with different boundary conditions.

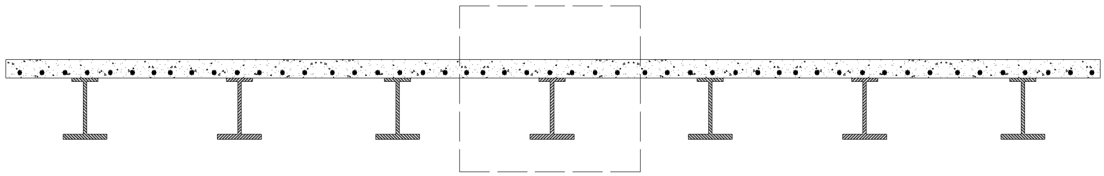
Two Dimensional Model of Deck and Girder

Two-dimensional finite element model is used to study the effect of several design factors on transverse bridge deck cracking caused mainly by drying shrinkage. A linear plane stress model of a single girder and the tributary deck with its reinforcements is developed using ANSYS (V5.5, 1998). Geometric and design information used in development of the finite element model (Table 4) corresponds to one of the bridges surveyed (i.e., Hackensack Ave over NJ Route 4). This bridge is built in 1998 in Bergen County, New Jersey. It is a two-span simply supported bridge on steel girders. The two spans are similar in design. This bridge has shown cracking all over both spans.

Table 4. Geometric and design information for the bridge modeled (Hackensack Ave over NJ Route 4).

Span Length	1082”
Span Width	564”
Girder Spacing	7 Girders@7’-1”
Deck thickness	8.5”
Longitudinal Bars	#5@6”(B), #5@15”(T)
Transverse Bars	#6@7”(T&B)
Girder Bottom Flange Thickness	2”
Girder top Flange Thickness	1.25”
Girder Bottom Flange Width	24”
Girder Bottom Top Width	20”
Girder Web Thickness	1”
Girder Web Height	35.25”
Shear Stud Spacing	12” (Midspan@71’) 9” (Support@18’)

The choice of plane stress model is made after detailed 3D FE analysis of the same bridge structure. Since it is computationally expensive to model and analyze a complete bridge structure in detail, usually a single girder and the corresponding portion of the deck is modeled and analyzed. As a single girder and its tributary deck are considered separate from the rest of the structure, it is required to somehow take into account the effect of the adjacent portion of the structure. Figure 30 shows typical girder and deck bridge cross section and highlights the single girder and its deck, which is modeled. To determine the correct type of finite element model that can be used to model the single girder and its deck a detailed 3D FE model was developed and analyzed. Results of this analysis showed that 2D plane stress and 3D stand alone models of single girder and its tributary deck are good approximations.



Portion to be modeled in detail

Figure 30. Cross-section of typical slab-on-girder bridge superstructure.

The 3D model is shown in Figure 31, which is the right portion of a bridge with a cross section shown in Figure 30. The girders are modeled using elastic shell elements and the deck is modeled with elastic solid elements. Since the overall nature of superstructure response is being investigated the reinforcements are not modeled and full composite action is assumed (i.e, deck and girder nodes are connected). Diaphragms are also modeled using truss elements. Their location and dimensions are chosen based on structural plans. Elastic modulus of steel and concrete are assumed to be equal to: 29E6 psi and 3.83E6 psi, respectively.

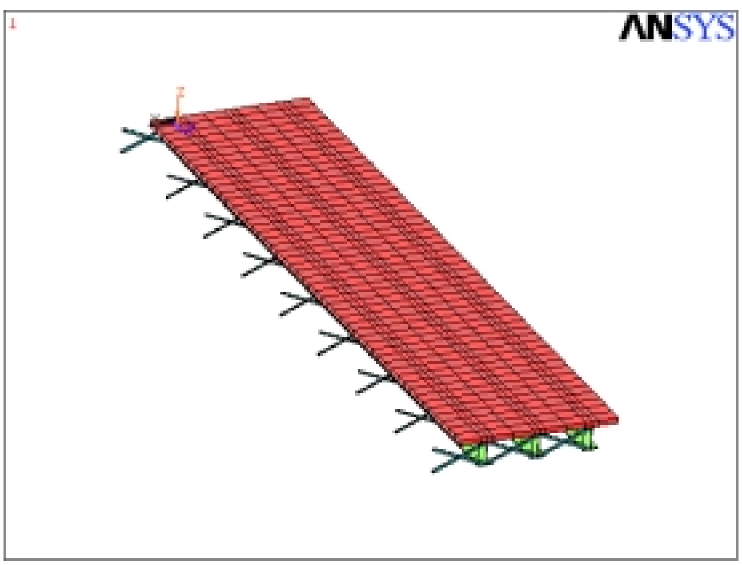


Figure 31. Finite element model of deck, girders and diaphragms.

The span modeled is simply supported. So the nodes on the bottom flange of the girders at one end is restrained in three directions and at the other end is only restrained in transverse and vertical directions. It is assumed that due to symmetry of the system, the middle girder will not have displacement in the X direction (transverse direction). Consequently, one end of the diaphragms, which are connected to middle girder are restrained.

Figure 32 shows the graph of required lateral nodal forces to move the edge of the deck in Figure. 31, 2.5-cm (1-inch) toward the center of the bridge. These values are obtained by specifying uniform unit displacement on the edge nodes of the deck and obtaining the restraining forces at each node. It can be said that these forces would be applied on the single girder and its tributary deck model provided that they have the same 2.5 cm (1 in) displacement. If the single girder and its deck were completely restrained by the adjacent structure (Figure 31) the required forced at each node of Figure 31 should be:

$$E_{Element}A_{Element} = 26E6 \times 0.22 \times 0.56 = 3200 \ \ KN \tag{6}$$

Where $E_{Element}$ (26E6-KN) is element modulus of elasticity, $A_{Element}$ is element area. 0.22-m (8.5-inch) is deck thickness and 0.56-m (22-inch) is element length. Considering the relative magnitude of actual forces in Figure 32 compared to Equation 1 result (3200-KN vs. 400KN), it can be concluded that the restraining effect of girders on each other in the lateral direction is negligible. This means that plane stress elements can be used in 2D models and that stand-alone 3D models are good approximations.

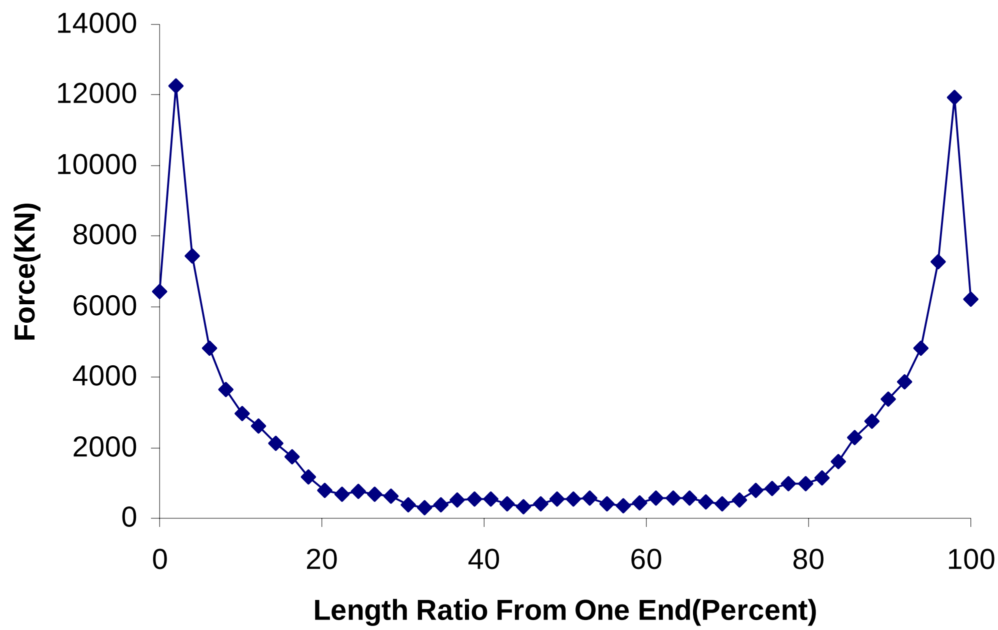


Figure 32. Lateral nodal forces along the deck assuming uniform displacements.

Since it is shown that it is justifiable to use plane stress 2D model to study the behavior of a single girder and its corresponding deck, the details of 2D FE model will be presented.

2D Model Details

Deck and Girder Elements: One girder and tributary deck are modeled using plane stress elements. The span is divided into 50 sections along its length. The deck and girder are modeled with 3 and 5 elements through the depth, respectively. The thickness of plane stress element is varied according to actual thickness of deck and girder flange/web in the transverse direction. Figure 33 shows the FE model a close up view of it.

Reinforcement Elements: The two layers of longitudinal reinforcement are modeled with truss elements. These elements are placed along the boundary of the deck plane stress elements at two different depths within the deck and are connected to the same nodes as plane stress deck elements. Based on the design detail for the bridge being modeled, total area of truss elements is chosen equal to total area of longitudinal reinforcements at the corresponding depth.

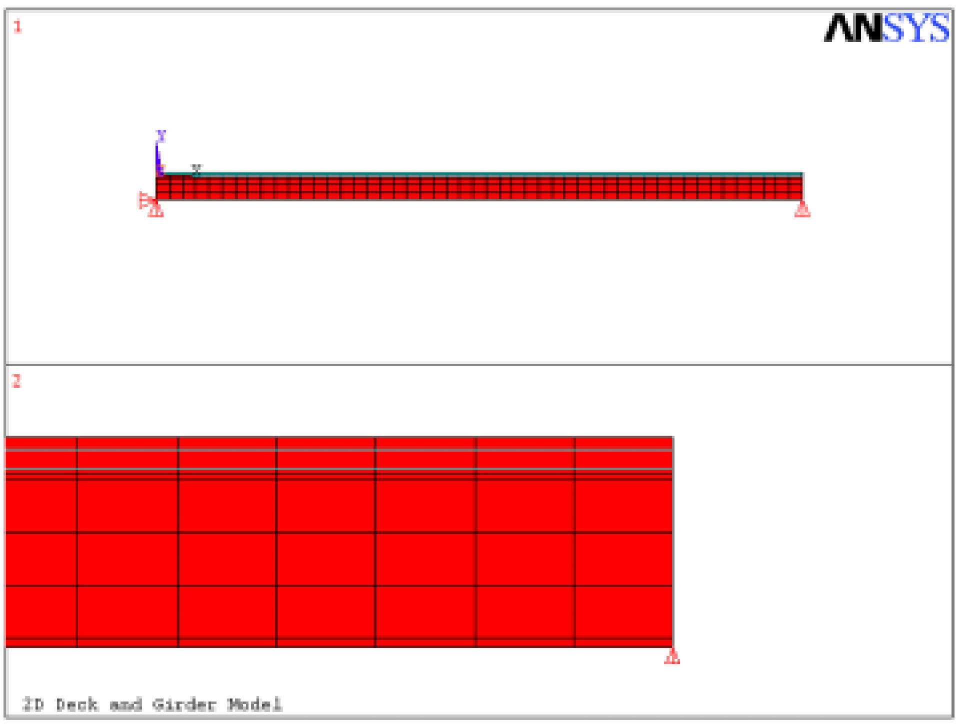


Figure 33. Two-dimensional (2D) finite element model.

Boundary Conditions: Based on the case being considered, different boundary conditions are applied on different nodes or set of nodes. A roller support is modeled by restraining the bottom node of the girder in vertical direction. A pin support is modeled by restraining the bottom node of the girder in the vertical and horizontal directions. To model a fixed end, all nodes at the cross section are restrained from moving in all directions. Figure 33 shows a girder with pin and roller supports.

Modulus of Elasticity: Elastic modulus of steel and concrete are assumed to be equal to: 29E6 psi and 3.83E6 psi, respectively.

Steel and Concrete Coefficient of Thermal Expansion: Winter and Nilson (1986) state that the concrete coefficient of thermal expansion should be in the range of 4E-6 to 7E-6 in/in per degree of Fahrenheit. Khan, Cook, and Mitchell (1998) measure the values of 5.3e-6 to 5.5e-6 in/in per ^{o}F for maturing normal weight concrete. In this study the values of 5.5E-6 and 6.25E-6 in/in per ^{o}F are assumed for concrete and steel, respectively.

Loading: Based on the case under consideration several types of loading are considered for each analysis, which are described in a case-by-case basis.

Results of Two Dimensional Analyses

Effects of several design factors on stresses causing transverse cracking are studied in this section. The design parameters of the base model (shown in Table 4) have been changed to study their effect on the stresses causing transverse cracking (Deck longitudinal stress, i.e. S_{xx} component of stress in Figure 31).

<u>Effect of Boundary Condition</u>

Increasing boundary condition restraint of deck and girder system increases tensile stress produced in deck, which may ultimately cause cracking. Figure 34 and Table 5 show the deck top and bottom stresses caused by 10^{o}F temperature decrease in the deck, while rebar temperature is held constant. This temperature decrease models the effect of uniform shrinkage strain equal to 5.5E-5 or 55 microstrain.

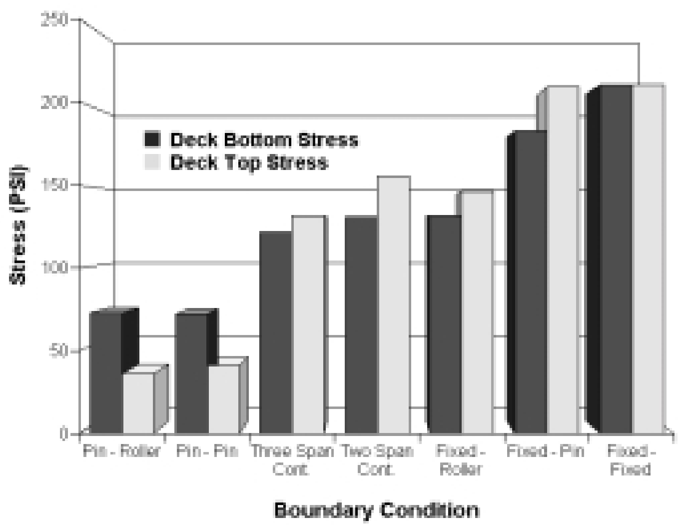


Figure 34. Deck bottom and top stresses caused by 55-microstrain uniform deck shrinkage for different boundary conditions.

Table 5. Deck top and bottom tensile stresses caused by 55-microstrain uniform deck shrinkage for different boundary conditions.

Boundary Condition (Section)	Deck Bottom Stress (psi)	Deck Top Stresses (psi)
Pin – Roller (Mid Section)	72.59	36.17
Pin – Pin (Mid Section)	71.86	40.94
Three Span Continuous (Over Support)	121.21	131.25
Two Span Continuous (Over Support)	130.84	155.35
Fixed – Roller (8%L From End Section)	131.40	145.81
Fixed – Pin (8%L From End Section)	182.74	208.80
Fixed – Fixed (8%L From End Section)	210.30	210.30

Effect of Span Length

Span length does not affect stresses in the deck considerably. This is an interesting result as it shows that the magnitude of stresses developed in the deck do not depend on span length. Effect of 50% change (increase and decrease) in span length on deck stresses caused by 55-microstrain uniform deck shrinkage is shown in Table 6. Neither deck bottom stresses, nor deck top stresses changes considerably (less than 10%) with span length for all boundary conditions considered. Figure 35 and 36 show the graph of top and bottom deck stresses for different span lengths considered.

Table 6. Deck top and bottom stresses caused by 55-microstrain uniform deck shrinkage for different span lengths and boundary conditions.

Boundary Condition (Section)	Deck Bottom Stress (psi)			Deck Top Stresses (psi)		
Percent of Original Length	50%L	100%L	150%L	50%L	100%L	150%L
Pin – Roller (Mid Section)	72.59	72.59	72.59	36.17	36.17	36.17
Pin – Pin (Mid Section)	71.92	71.86	71.84	40.55	40.94	41.06
Three Span Continuous (Over Support)	116.21	121.21	122.05	129.40	131.25	130.84
Two Span Continuous (Over Support)	123.98	130.84	132.64	152.58	155.35	154.84
Fixed – Roller (8%L From End Section)	128.32	131.40	131.71	145.74	145.81	146.28
Fixed – Pin (8%L From End Section)	173.66	182.74	184.18	204.29	208.80	210.71
Fixed – Fixed (8%L From End Section)	210.30	210.30	210.30	210.30	210.30	210.30

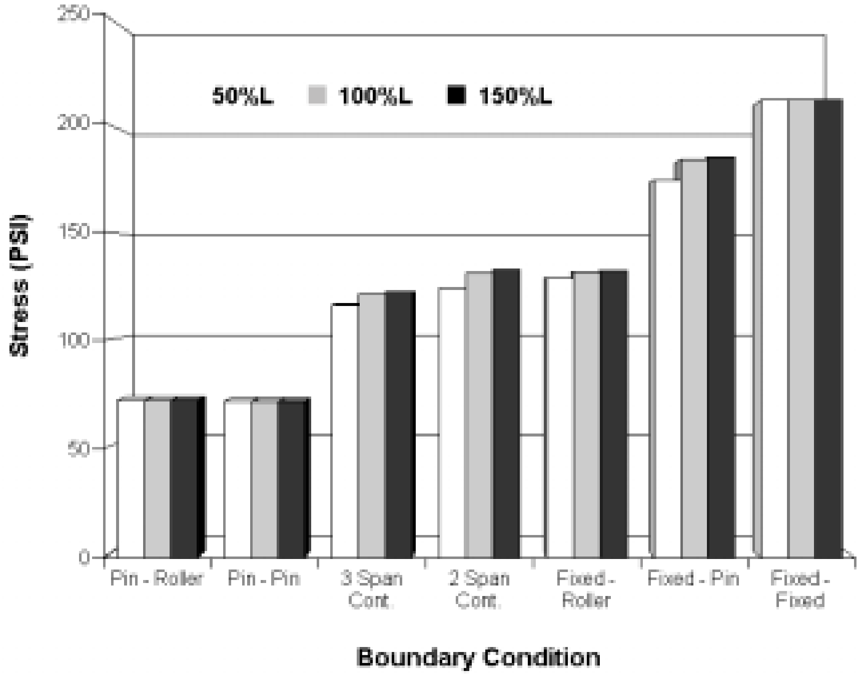


Figure 35. Deck bottom stresses caused by 55-microstrain uniform deck shrinkage for different span lengths and boundary conditions.

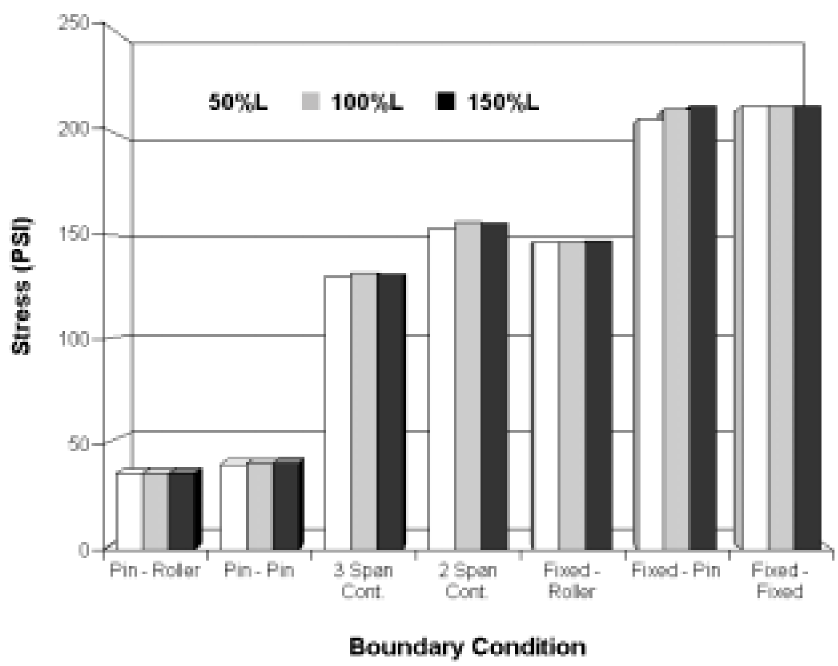


Figure 36. Deck top stresses caused by 55-microstrain uniform deck shrinkage for different span lengths and boundary conditions.

Effect of Deck Thickness

Increasing deck thickness reduces deck stresses for all except fixed-fixed boundary conditions. Thus, thicker decks are preferred over thinner decks with respect to transverse cracking. However, practical limits of deck thickness should be considered.

Table 7 shows effect of thickness on deck stresses due to shrinkage. Concrete cover is held constant while deck thickness is changed. Figures 37 and 38 show graphically the effect of deck thickness on bottom and top stresses for different boundary conditions. These stresses are produced by 55-microstrain uniform concrete deck shrinkage.

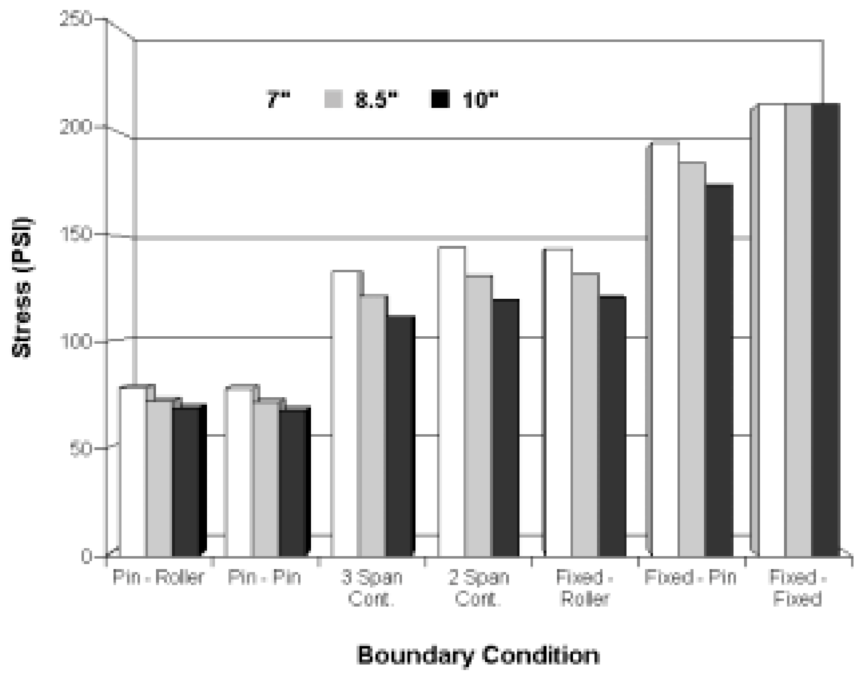


Figure 37. Deck bottom stresses caused by 55-microstrain uniform deck shrinkage for different deck thickness and boundary conditions.

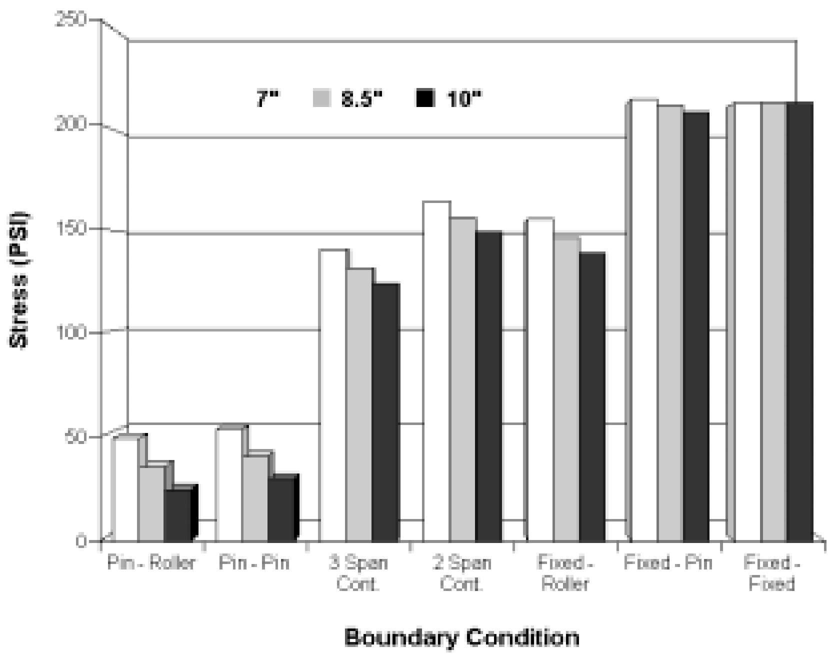


Figure 38. Deck top stresses caused by 55-microstrain uniform deck shrinkage for different deck thickness and boundary conditions.

Table 7. Deck top and bottom stresses caused by 55-microstrain uniform deck shrinkage for different deck thickness and boundary conditions.

Boundary Condition (Section)	Deck Bottom Stress (psi)			Deck Top Stresses (psi)		
Deck Thickness	7”	8.5”	10”	7”	8.5”	10”
Pin – Roller (Mid Section)	78.25	72.59	69.11	49.66	36.17	24.98
Pin – Pin (Mid Section)	78.17	71.86	67.72	53.88	40.94	30.22
Three Span Continuous (Over Support)	132.38	121.21	111.41	139.96	131.25	123.51
Two Span Continuous (Over Support)	143.60	130.84	119.32	163.06	155.35	148.61
Fixed – Roller (8%L From End Section)	143.17	131.40	120.81	154.41	145.81	138.40
Fixed – Pin (8%L From End Section)	192.30	182.74	173.14	212.01	208.80	205.86
Fixed – Fixed (8%L From End Section)	210.30	210.30	210.30	210.30	210.30	210.30

Effect of Girder Spacing

Increasing the girder spacing will reduce deck stresses produced by volume change of concrete in all but fixed-fixed deck girder system.

Table 8 shows results of the finite element analysis for the example bridge with 50, 100, and 150 percent of original spacing and different boundary conditions. These stresses are for 55-microstrain uniform concrete shrinkage. Figures 39 and 40 show deck top and bottom stresses for different spacing and boundary conditions.

Table 8. Deck top and bottom stresses caused by 55-microstrain uniform deck shrinkage for different girder spacing and boundary conditions.

Boundary Condition (Section)	Deck Bottom Stress (psi)			Deck Top Stresses (psi)		
Percent of Original width	50%W	100% W	150%W	50%W	100%W	150%W
Pin – Roller (Mid Section)	100.63	72.59	59.73	71.53	36.17	20.32
Pin – Pin (Mid Section)	100.93	71.86	58.41	76.49	40.94	24.47
Three Span Continuous (Over Support)	156.72	121.21	97.98	163.98	131.25	108.95
Two Span Continuous (Over Support)	168.75	130.84	104.83	187.53	155.35	132.10
Fixed – Roller (8%L From End Section)	167.49	131.40	69.96	179.02	145.81	122.49
Fixed – Pin (8%L From End Section)	206.17	182.74	161.28	224.57	208.80	192.29
Fixed – Fixed (8%L From End Section)	210.30	210.30	210.30	210.30	210.30	210.30

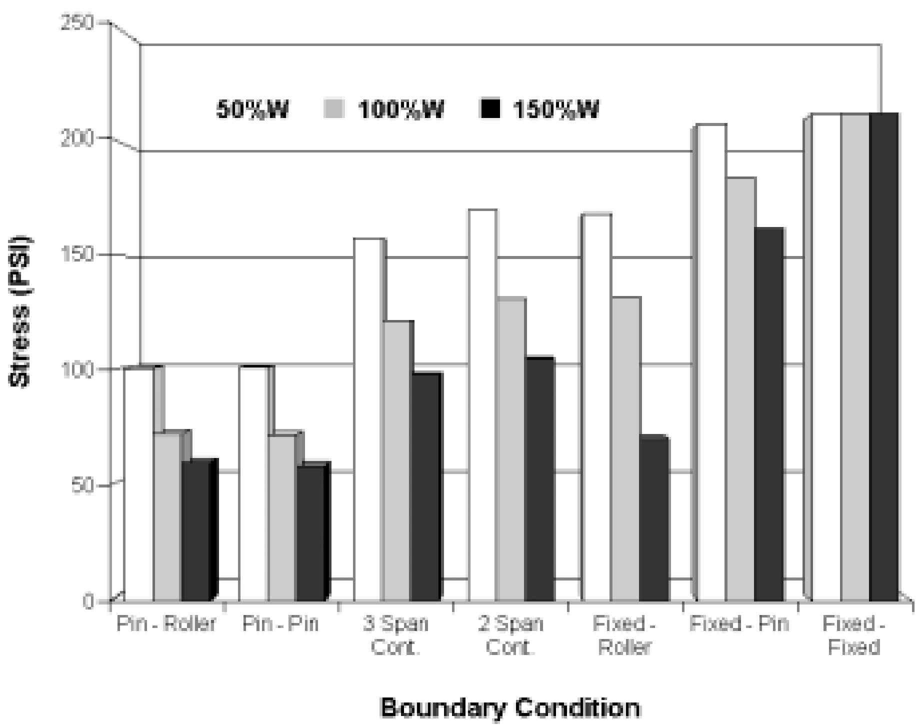


Figure 39. Deck bottom stresses caused by 55-microstrain uniform deck shrinkage for different girder spacing and boundary conditions.

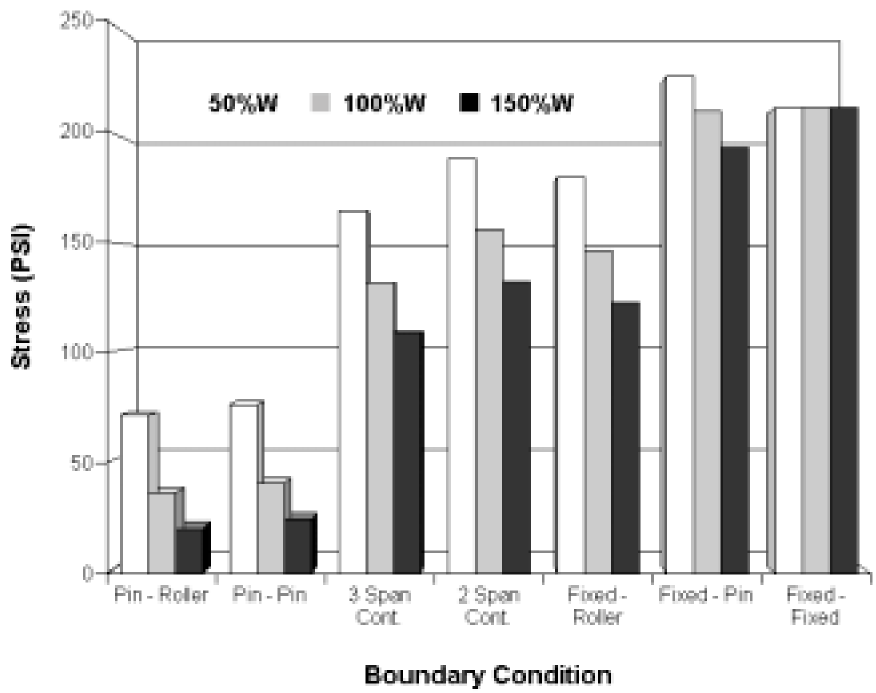


Figure 40. Deck top stresses caused by 55-microstrain uniform deck shrinkage for different girder spacing and boundary conditions.

Effect of Relative Flexibility of Girder to Deck

Increasing the ratio of girder to deck moment of inertia increases deck stresses for all but fixed-fixed boundary condition. Table 9 shows deck stresses for different boundary conditions and three different ratios of girder to deck moment of inertia, while composite section moment of inertia is held constant. In other words all the sections have the same composite section stiffness, however, contribution of deck and girder is different. Deck thickness and girder height are changed to obtain different relative flexibility but equal composite section inertia, all other aspects of design are held constant. Table 10 shows contribution of deck and girder in each case and the corresponding deck thickness and girder height. Concrete cover is held constant while deck thickness is changed. Figures 41 and 42 shows the graph of deck top and bottom stresses for different boundary conditions and various ratios of girder to deck moment of inertia.

These results show that for all but fixed-fixed boundary condition, the potential for deck cracking reduces as the ratio of girder to deck moment of inertia decreases. Since composite section moment of inertia is held constant, change in relative stiffness (flexibility) will not affect bridge serviceability requirements while reducing deck-cracking potential. There is no change in stresses for fixed-fixed boundary condition.

Table 9. Deck top and bottom stresses caused by 55-microstrain uniform deck shrinkage for different ratio of girder/deck moment of inertia and boundary conditions.

Boundary Condition (Section)	Deck Bottom Stress (psi)			Deck Top Stresses (psi)		
Ratio of Girder to Deck Inertia	4.13	6.18	9.66	4.13	6.18	9.66
Pin – Roller (Mid Section)	70.16	72.59	76.19	27.34	36.17	45.97
Pin – Pin (Mid Section)	68.89	71.86	75.99	32.49	40.94	50.33
Three Span Continuous (Over Support)	113.10	121.21	128.25	125.28	131.25	137.15
Two Span Continuous (Over Support)	122.25	130.84	139.91	150.64	155.35	160.52
Fixed – Roller (8%L From End Section)	123.47	131.40	139.80	140.49	145.81	151.70
Fixed – Pin (8%L From End Section)	175.70	182.74	189.61	207.13	208.80	210.67
Fixed – Fixed (8%L From End Section)	210.30	210.30	210.30	210.30	210.30	210.30

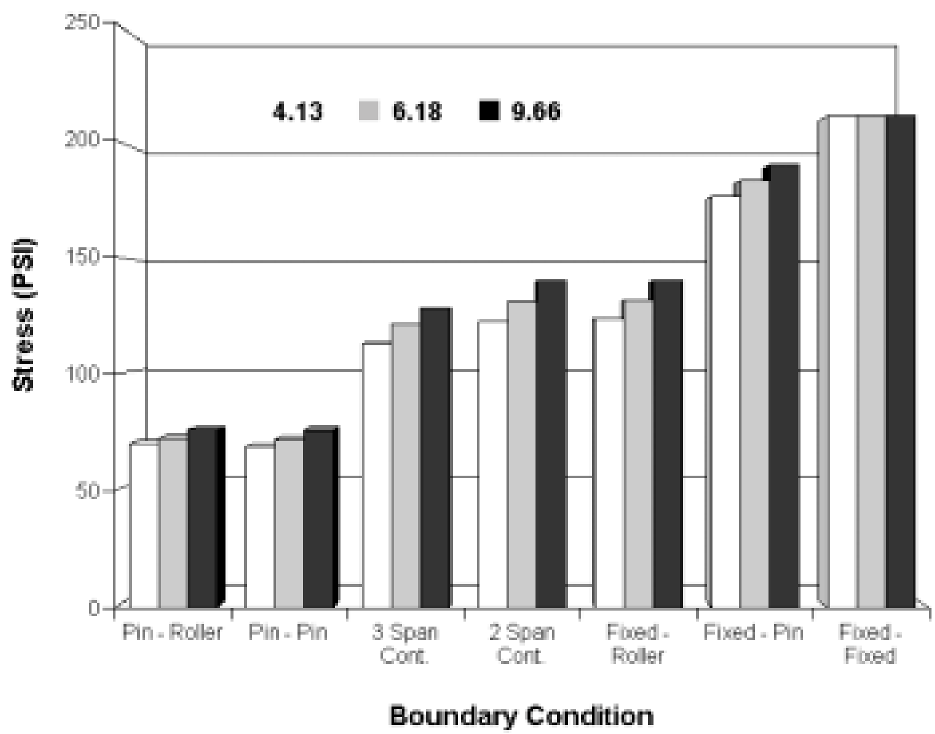


Figure 41. Deck bottom stresses caused by 55-microstrain uniform deck shrinkage for different ratio of girder/deck moment of inertia and boundary conditions.

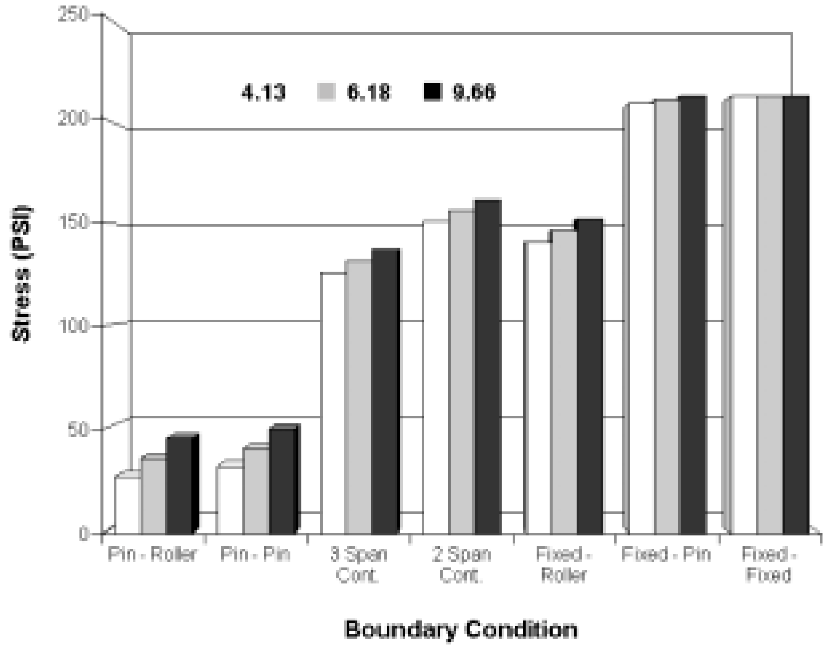


Figure 42. Deck top stresses caused by 55-microstrain uniform deck shrinkage for different ratio of girder/deck moment of inertia and boundary conditions.

Table 10. Different ratio of girder/deck moment of inertia used in calculation and corresponding deck and girder dimensions.

Section Number	Deck Thickness (in)	Web Height (in)	Deck Inertia (in^4)	Girder Inertia (in^4)	Composite Inertial (in^4)	Girder/Deck Ratio
1	7.5	36.474	2988	28855	64853.1	9.66
2	8.5	35.25	4350	26876	64852.5	6.18
3	9.5	34.1	6073	25089	64852.8	4.13

Effect of Composite Section Moment of Inertia

Increasing composite section moment of inertia increases deck stresses slightly for all except fixed-fixed and fixed-pin boundary conditions. Composite section moment of inertia is varied by changes in web height as shown in Table 11. The deck top and bottom stresses are shown in Table 12 and Figures 43 and 44.

Table 11. Different ratio of composite moment of inertia used in calculation and corresponding deck and girder dimensions.

Section Number	Web Height (in)	Composite Inertial (in^4)	%
1	33.25	58415.8	90
2	35.25	64852.5	100
3	37.25	71666.6	110.5

Table 12. Deck top and bottom stresses caused by 55-microstrain uniform deck shrinkage for different composite moment of inertia and boundary conditions.

Boundary Condition (Section)	Deck Bottom Stress (psi)			Deck Top Stresses (psi)		
Composite Moment of Inertia	90%	100%	110.5%	90%	100%	110.5%
Pin – Roller (Mid Section)	72.59	72.59	72.66	34.19	36.17	38.03
Pin – Pin (Mid Section)	71.70	71.86	72.08	39.06	40.94	42.70
Three Span Continuous (Over Support)	119.33	121.21	121.38	130.32	131.25	131.52
Two Span Continuous (Over Support)	129.55	130.84	132.07	155.14	155.35	155.58
Fixed – Roller (8%L From End Section)	130.16	131.40	132.59	145.35	145.81	146.30
Fixed – Pin (8%L From End Section)	181.76	182.74	183.64	209.32	208.80	208.35
Fixed – Fixed (8%L From End Section)	210.30	210.30	210.30	210.30	210.30	210.30

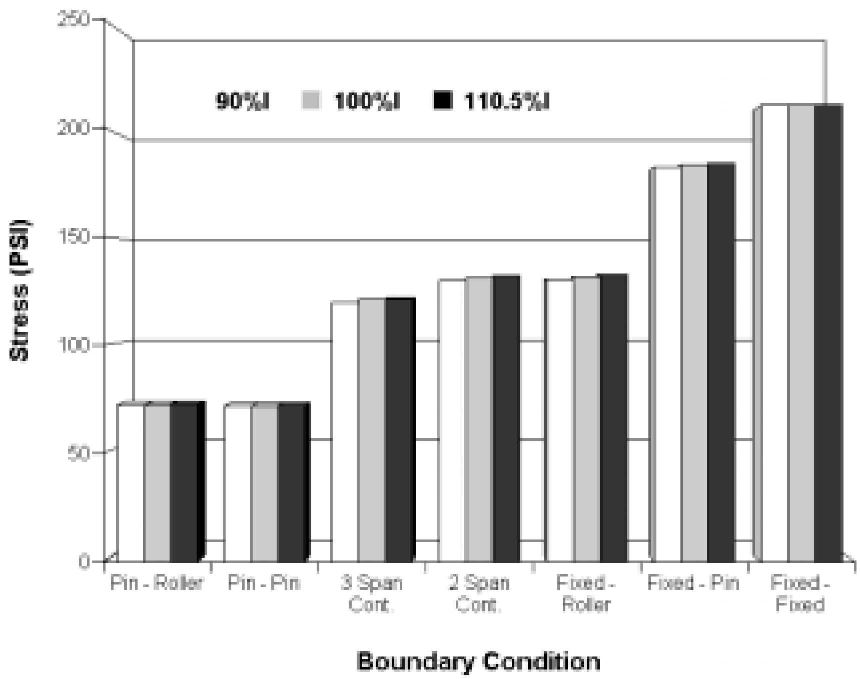


Figure 43. Deck bottom stresses caused by 55-microstrain uniform deck shrinkage for different composite moment of inertia and boundary conditions.

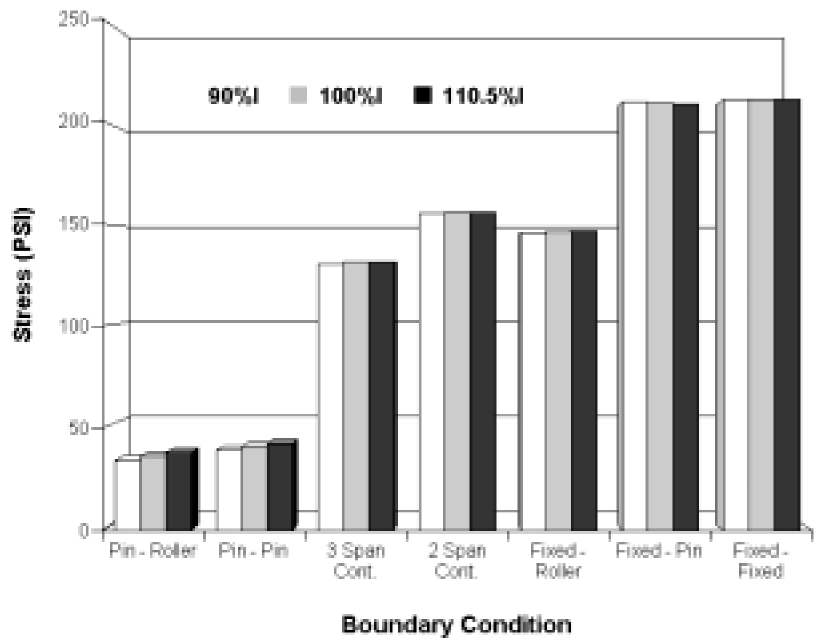


Figure 44. Deck top stresses caused by 55-microstrain uniform deck shrinkage for different composite moment of inertia and boundary conditions.

Effect of Amount of Longitudinal Reinforcement

Increasing the area of longitudinal reinforcement increases the deck stresses for pin-roller and pin –pin spans. There is no significant change in stress for other boundary conditions. These results show that contrary to what is generally believed, increasing the amount of reinforcement not only will not reduce deck stresses prior to cracking, but also in some cases increases deck stresses.

Table 13 shows effect of 50% increase and decrease in rebar area on deck stresses from 55-microstrain uniform shrinkage. 50% change in rebar area corresponds approximately to increasing rebar number by 1 size (say from #4 to #5). For comparison, a theoretical deck with zero reinforcement is also analyzed. These results are shown in Table 14. Figures 45 and 46 show the results in graphical format.

Table 13. Deck top stresses caused by 55-microstrain uniform deck shrinkage for different area of longitudinal reinforcement and boundary conditions.

Boundary Condition (Section)	Deck Bottom Stress (psi)			Deck Top Stresses (psi)		
Percent of Original Reinforcement Area	50%Ar	100%Ar	150%Ar	50%Ar	100%Ar	150%Ar
Pin – Roller (Mid Section)	69.26	72.59	75.52	32.36	36.17	39.82
Pin – Pin (Mid Section)	68.81	71.86	74.79	37.20	40.94	44.51
Three Span Continuous (Over Support)	120.09	121.21	122.30	129.97	131.25	131.85
Two Span Continuous (Over Support)	130.14	130.84	131.53	155.10	155.35	155.61
Fixed – Roller (8%L From End Section)	130.66	131.40	132.14	145.36	145.81	146.26
Fixed – Pin (8%L From End Section)	183.27	182.74	182.25	209.74	208.80	207.91
Fixed – Fixed (8%L From End Section)	210.30	210.30	210.30	210.30	210.30	210.30

Table 14. Deck top stresses caused by 55-microstrain uniform deck shrinkage for zero area of longitudinal reinforcement and boundary conditions.

Boundary Condition (Section)	Deck Bottom Stress (psi)	Deck Top Stresses (psi)
Pin – Roller (Mid Section)	66.32	28.37
Pin – Pin (Mid Section)	65.62	33.30
Three Span Continuous (Over Support)	118.95	129.38
Two Span Continuous (Over Support)	129.45	154.88
Fixed – Roller (8%L From End Section)	129.91	144.92
Fixed – Pin (8%L From End Section)	183.85	210.74
Fixed – Fixed (8%L From End Section)	210.30	210.30

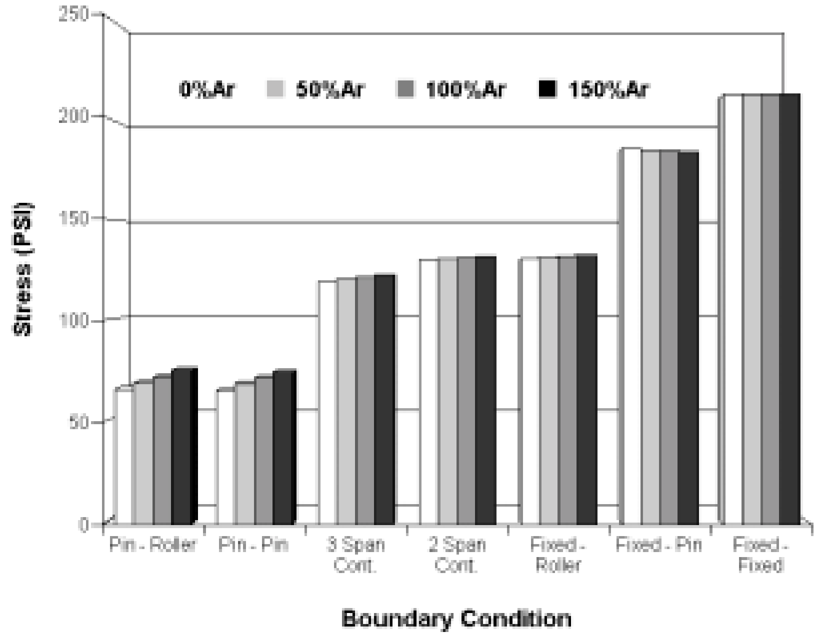


Figure 45. Deck bottom stresses caused by 55-microstrain uniform deck shrinkage for different area of longitudinal reinforcement and boundary conditions.

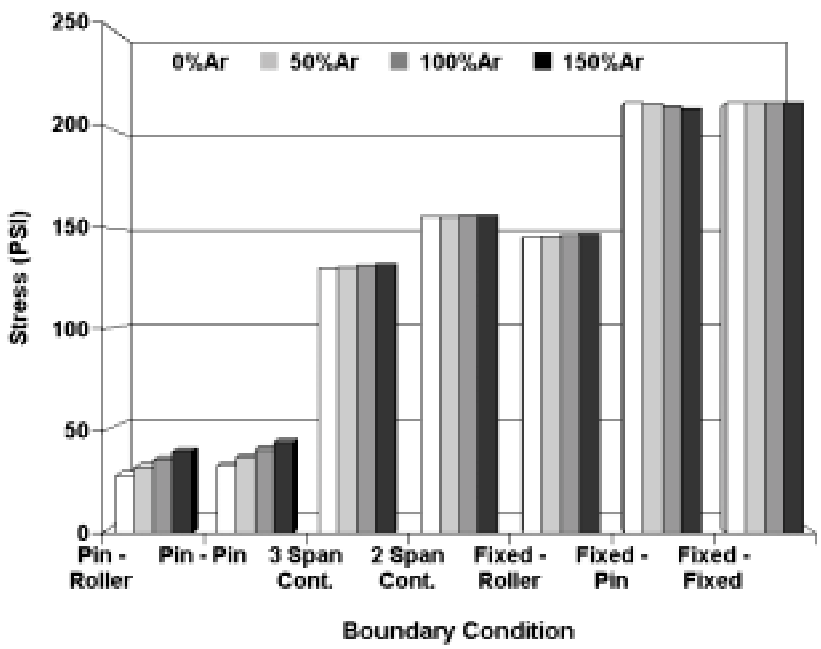


Figure 46. Deck top stresses caused by 55-microstrain uniform deck shrinkage for different area of longitudinal reinforcement and boundary conditions.

Effect of Distribution of Longitudinal Reinforcement

Changing the distribution pattern of total reinforcement area among top and bottom reinforcement do not seem to change the deck stresses. In other words, for the same amount of reinforcement in the section, increasing top or bottom reinforcement do not change the deck stresses.

Table 15 shows the effect of longitudinal reinforcement distribution on deck stresses produced by 55-microstrain uniform shrinkage. Keeping the total amount of reinforcement constant, 25 percent of total reinforcement is moved from bottom to top and vice versa. Figures 47 and 48 illustrate the effect of changing the pattern of longitudinal reinforcement distribution on deck stresses.

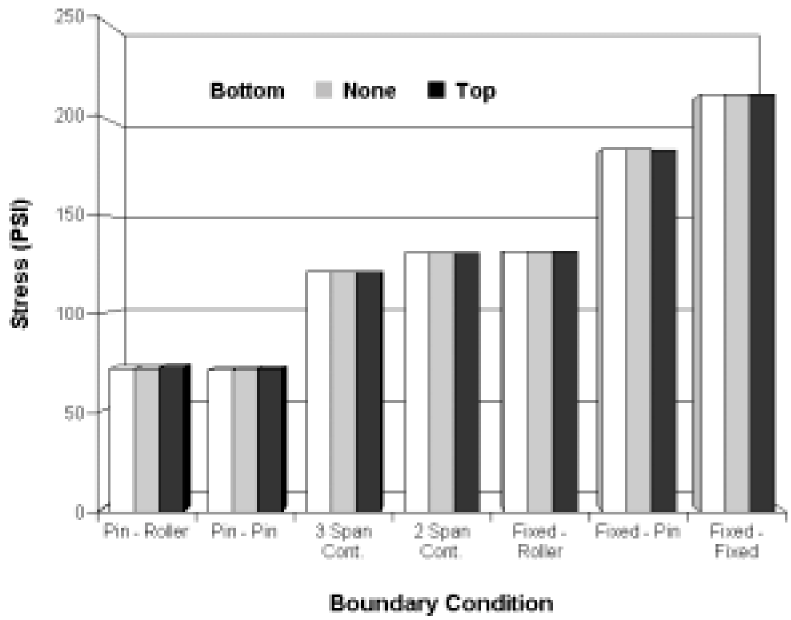


Figure 47. Deck bottom stresses caused by 55-microstrain uniform deck shrinkage for different distribution of longitudinal reinforcement and boundary conditions.

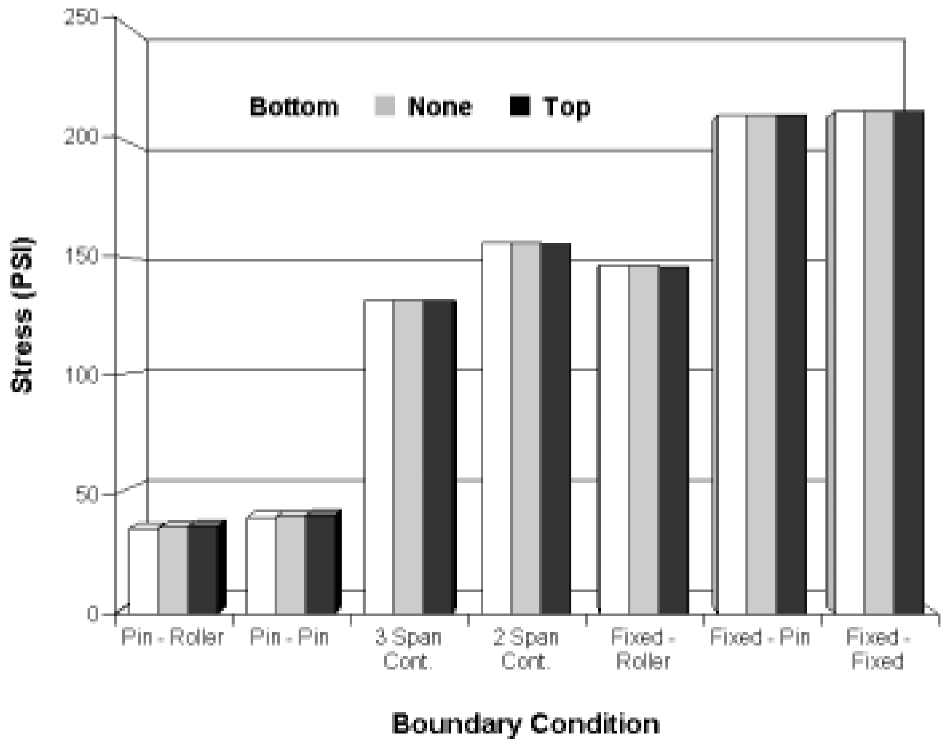


Figure 48. Deck top stresses caused by 55-microstrain uniform deck shrinkage for different distribution of longitudinal reinforcement and boundary conditions.

Table 15. Deck top and bottom stresses caused by 55-microstrain uniform deck shrinkage for different distribution of longitudinal reinforcement and boundary conditions.

Boundary Condition (Section)	Deck Bottom Stress (psi)			Deck Top Stresses (psi)		
Location of Reinforcement Increase	Bottom	None	Top	Bottom	None	Top
Pin – Roller (Mid Section)	72.26	72.59	72.92	35.66	36.17	36.68
Pin – Pin (Mid Section)	71.55	71.86	72.18	40.54	40.94	41.33
Three Span Continuous (Over Support)	121.28	121.21	121.13	131.38	131.25	131.13
Two Span Continuous (Over Support)	131.02	130.84	130.66	155.66	155.35	155.04
Fixed – Roller (8%L From End Section)	131.53	131.40	131.27	146.01	145.81	145.61
Fixed – Pin (8%L From End Section)	182.84	182.74	182.64	208.96	208.80	208.63
Fixed – Fixed (8%L From End Section)	210.30	210.30	210.30	210.30	210.30	210.30

Effect of Girder Shoring During Construction

Temporary shoring during construction reduces the stresses in the concrete deck significantly and even induces some compressive stress for pin-roller and pin-pin boundary conditions. However, temporary shoring for other boundary condition does not have beneficial effect on deck stresses as the dead load produces additional tensile stresses in the deck upon removal of the shoring.

If the girder is shored during deck pouring and curing, the dead load of concrete is carried by composite section after removal of shoring. Table 16 shows the dead load stresses produced in the deck for simply supported girder (tension is positive). Compressive stresses in the deck provide additional safety against deck cracking. In other words, tensile stresses produced by concrete volume change should be much larger for a shored construction compared to un-shored one in order to crack the concrete deck. However, for other types of boundary conditions, the dead load induces tensile stresses in some regions and may worsen the problem of cracking.

Table 16. Deck top and bottom stresses caused deck dead load.

Boundary Condition (Section)	Deck Bottom Stress (psi)	Deck Top Stresses (psi)
Pin – Roller (Mid Section)	-175.43	-331.85
Pin – Pin (Mid Section)	-185.07	-268.72

Effect of Pouring Sequence

Properly selected pouring sequence reduces the possibility of cracking by inducing compressive stress in the deck. As a general rule, positive moment sections should be placed first followed by negative moment sections.

Figure 49 shows the recommended pouring sequence for different boundary conditions. In each case, the numbers on the top of the deck show the pouring sequence of deck concrete and the length of pour is indicated at the bottom of girder. As it is illustrated in Figure 49, the middle half of each span is placed first and then the remaining portion is placed in the sequence shown. Tables 17 through 25 show the residual stresses in the deck as a result of pouring sequence shown in Figure 49. These results show that considerable stress is produced in the deck depending on the sequence of pouring. As a general rule to minimize the tensile stresses and maximize the compressive strength in the deck, in single span bridges, positive moment portions should be poured first, followed by negative moment sections. In multi span bridges, the whole span should be placed in one pour. Since this is not practical in most cases, the pouring sequence as depicted in Figure 49 is suggested.

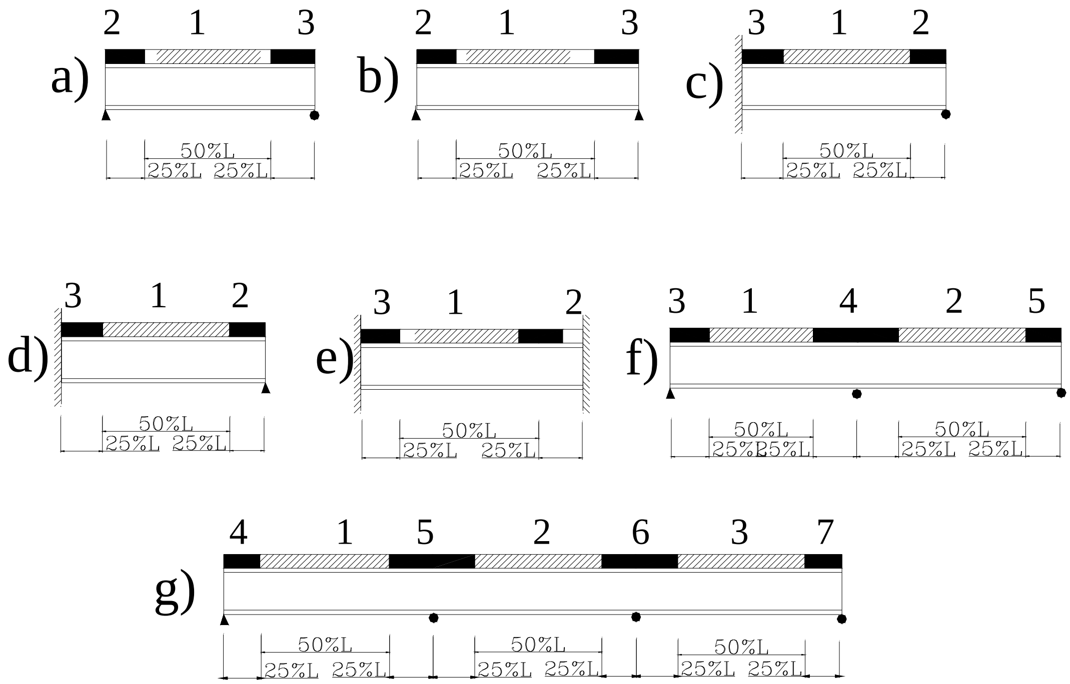


Figure 49. Suggested pouring sequence and lengths for different boundary conditions.

Table 17. Residual top and bottom deck stresses caused by pouring sequence (a) for pin-roller boundary condition.

Pin – Roller Boundary Condition	Deck Bottom Stress (psi)			Deck Top Stresses (psi)		
After Pouring Section	1	2	3	1	2	3
End Section (8%L From Left Support)	0	0	-2.06	0	0	-3.52
Middle Section	0	-17.12	-32.85	0	-37.17	-70.45
End Section (8%L From Right Support)	0	0	0	0	0	0

Table 18. Residual top and bottom deck stresses caused by pouring sequence (b) for pin-pin boundary condition.

Pin – Pin Boundary Condition	Deck Bottom Stress (psi)			Deck Top Stresses (psi)		
After Pouring Section	1	2	3	1	2	3
End Section (8%L From Left Support)	0	0	-3.85	0	0	6.59
Middle Section	0	-22.94	-42.18	0	-31.85	-56.62
End Section (8%L From Right Support)	0	0	0	0	0	0

Table 19. Residual top and bottom deck stresses caused by pouring sequence (c) for fixed-roller boundary condition.

Fixed - Roller Boundary Condition	Deck Bottom Stress (psi)			Deck Top Stresses (psi)		
After Pouring Section	1	2	3	1	2	3
End Section (8%L From Left Support)	0	0	0	0	0	0
Middle Section	0	-11.54	-17.02	0	-22.94	-34.09
End Section (8%L From Right Support)	0	0	-1.57	0	0	-0.80

Table 20. Residual top and bottom deck stresses caused by pouring sequence (d) for fixed-pin boundary condition.

Fixed - Pin Boundary Condition	Deck Bottom Stress (psi)			Deck Top Stresses (psi)		
After Pouring Section	1	2	3	1	2	3
End Section (8%L From Left Support)	0	0	0	0	0	0
Middle Section	0	-19.67	-26.99	0	-25.97	-37.62
End Section (8%L From Right Support)	0	0	-1.43	0	0	0.39

Table 21. Residual top and bottom deck stresses caused by pouring sequence (e) for fixed-fixed boundary condition.

Fixed - Fixed Boundary Condition	Deck Bottom Stress (psi)			Deck Top Stresses (psi)		
After Pouring Section	1	2	3	1	2	3
End Section (8%L From Left Support)	0	0	0	0	0	0
Middle Section	0	11.80	4.49	0	9.60	-2.63
End Section (8%L From Right Support)	0	0	-3.58	0	0	-11.03

Table 22. Residual bottom deck stresses caused by pouring sequence (f) for two span continuous girder.

Two Span Continuous Condition	Deck Bottom Stress (psi)				
After Pouring Section	1	2	3	4	5
Middle of Left Span	0	5.34	-9.70	-9.83	-9.63
Over Interior Support	0	0	0	0	1.80
Middle of Right Span	0	0	4.73	8.34	5.52

Table 23. Residual top deck stresses caused by pouring sequence (f) for two span continuous girder.

Two Span Continuous Condition	Deck Top Stress (psi)				
After Pouring Section	1	2	3	4	5
Middle of Left Span	0	10.62	-18.59	-18.90	-18.62
Over Interior Support	0	0	0	0	4.83
Middle of Right Span	0	0	9.94	17.74	12.38

Table 24. Residual bottom deck stresses caused by pouring sequence (g) for three span continuous girder.

Three Span Continuous Condition	Deck Bottom Stress (psi)						
After Pouring Section	1	2	3	4	5	6	7
Middle of Left Span	0	7.87	6.07	2.66	1.28	1.71	1.40
Over Left Interior Support	0	0	0	0	0	0.65	0.21
Middle of Middle Span	0	0	8.26	8.96	7.97	6.97	8.05
Over right Interior Support	0	0	0	0	0	0	1.97
Middle of Right Span	0	0	0	-0.15	0.12	-1.16	-4.37

Table 25. Residual top deck stresses caused by pouring sequence (g) for three span continuous girder.

Three Span Continuous Condition	Deck Top Stress (psi)						
After Pouring Section	1	2	3	4	5	6	7
Middle of Left Span	0	14.47	11.18	5.27	2.80	3.62	3.04
Over Left Interior Support	0	0	0	0	0	1.69	0.48
Middle of Middle Span	0	0	15.35	16.53	14.73	12.99	14.73
Over right Interior Support	0	0	0	0	0	0	4.81
Middle of Right Span	0	0	0	-0.26	0.21	-8.01	-8.01

Effect of Shrinkage Profile

So far it was assumed that shrinkage is uniform across the deck depth. Shrinkage profile (gradient of shrinkage through the depth of deck) changes stress profile in the deck. Based on actual bridge properties, volume change gradient may increase or decrease stresses in the deck.

Table 26 shows deck stresses produced by +2 and –2^{o}F temperature gradients (11 microstrain) through the deck as shown in Figure 50. As seen in the Figure 51 deck stress profile tends to follow the change in shrinkage profile in the deck. In other words, increasing shrinkage in top causes an increase in the top stress. It seems desirable to design the deck and girder to have uniform stress profile for uniform shrinkage profile. This will reduce the adverse effect of shrinkage gradient through the section.

Table 26. Deck top and bottom stresses caused by different shrinkage profile in deck for different boundary conditions.

Boundary Condition (Section)	Deck Bottom Stress (psi)			Deck Top Stresses (psi)		
Volume Change Distribution Number	1	2	3	1	2	3
Pin – Roller (Mid Section)	96.23	72.59	48.95	18.69	36.17	53.64
Pin – Pin (Mid Section)	95.55	71.86	48.18	23.16	40.94	58.70
Three Span Continuous (Over Support)	143.58	121.21	97.98	111.30	131.25	151.21
Two Span Continuous (Over Support)	152.96	130.84	108.72	134.77	155.35	175.92
Fixed – Roller (8%L From End Section)	153.51	131.40	109.29	125.48	145.81	166.13
Fixed – Pin (8%L From End Section)	204.27	182.74	161.21	187.76	208.80	229.84
Fixed – Fixed (8%L From End Section)	231.33	210.30	189.27	189.27	210.30	231.33

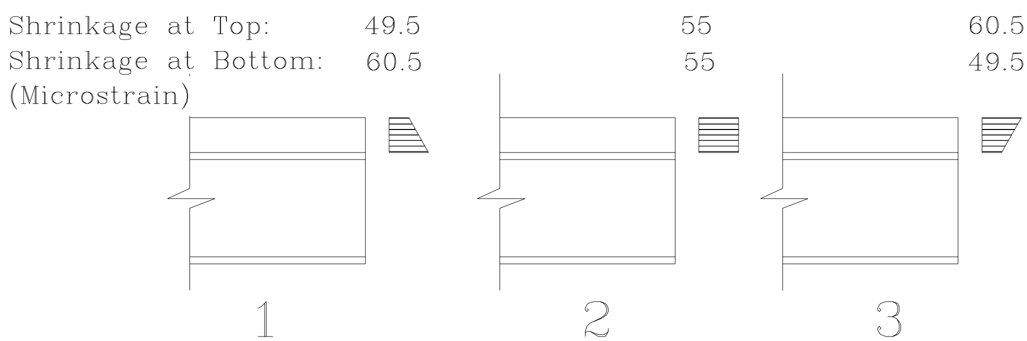


Figure 50. Shrinkage stress profile.

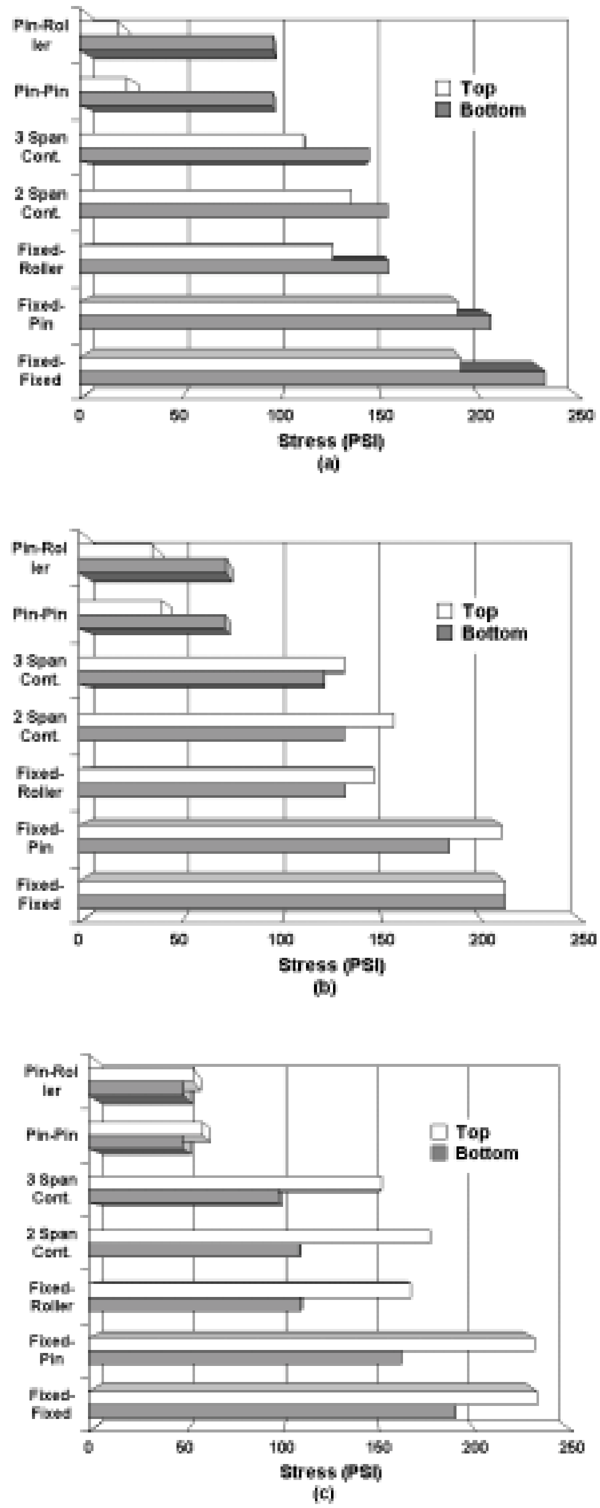


Figure 51. Deck top and bottom stresses for different boundary conditions and (a) shrinkage profile 1, (b) shrinkage profile 2, (c) shrinkage profile 3.

Summary of Results of Two dimensional Analysis

Effect of several design factors on deck stresses is evaluated and the results are presented in tables and figures. In summary:

1. Increasing boundary condition restraint of deck and girder system increases tensile stress in the deck.
2. Span length does not affect the deck stresses considerably.
3. Increasing deck thickness reduces deck stresses for all except fixed-fixed boundary conditions.
4. Increasing girder spacing will reduce deck stresses produced by volume change of concrete in all but fixed-fixed deck girder system.
5. Increasing the ratio of girder to deck moment of inertia increases deck stresses for all but fixed-fixed boundary condition
6. Increasing composite section moment of inertia increases deck stresses slightly for all except fixed-fixed and fixed-pin boundary conditions
7. Increasing the area of longitudinal reinforcement increases the deck stresses for pin-roller and pin –pin spans. There is no significant change in stress for other boundary conditions.
8. Changing the distribution pattern of total reinforcement area among top and bottom reinforcement does not change the deck stresses
9. Deck stresses for fixed-fixed supported deck and girder is only a function of volume change and do not change with other parameters.
10. Temporary shoring during construction reduces tensile stresses in the concrete deck significantly and induces some beneficial compressive stress for pin-roller and pin-pin boundary conditions. However, shoring for other boundary condition does not have beneficial effect on deck stresses as the dead load produces additional tensile stresses in the deck.
11. Properly selected pouring sequence reduces possibility of cracking by inducing compressive stress in the deck. As a general rule, positive moment sections should be placed first followed by negative moment sections.
12. Volume change gradient through the section changes stress profile in the deck. However based on actual bridge properties, this change in profile may increase or decrease the stress in the deck.

Three Dimensional Model of Deck and Girder

The three dimensional model is used to investigate crack patterns in concrete bridge deck. The analysis considers the bridge deck at early ages (less than three months) .To be effective and useful the model must:

1. Capture important design characteristics of the bridge including geometry, boundary conditions, and sectional properties
2. Be capable of predicting cracks
3. Be capable of modeling behavior of bridge after cracking

To achieve these objectives, a *non-linear* finite element model was developed using ANSYS (V5.5, 1998).

Similar to the 2D model, the 3D bridge model uses geometric and design information for one of the bridges surveyed (i.e., Hackensack Ave. over NJ Route 4 SB Bridge - Table 4).

3D Model Details

Basic Assumptions: The following are the basic assumptions made in building the 3D model:

1. Since the girders on the adjacent sides of the girder under consideration are equally spaced, relative symmetry is assumed.
2. The middle girder of the bridge as shown in Figure 31 is modeled. So the girder web is a plane of symmetry under uniform shrinkage, thermal load, and uniform gravity load on the deck. In other words, this means that the girder has no displacement in the direction normal to the plane of its web. Similarly, it is argued that the girder has no rotation about its longitudinal axis passing the girder web plate. Due to this symmetry, only a half of the girder and deck is modeled as shown in Figure 52.
3. As shown earlier in the chapter, the effect of adjacent portion of deck and girders can be ignored.
4. Abutments and supports are considered rigid.

Deck and Girder Modeling: One girder of the bridge with the corresponding portion of the deck is modeled in three dimensions. Due to symmetry of model, only half of deck and girder are modeled. The bridge deck is modeled using solid elements capable of cracking, while the girder is modeled using shell elements. The length of the span is divided into 49 strips. In each strip the deck is modeled with 21 elements located in three layers. The girder is modeled using 4 elements in each strip, 2 elements for the top flange, one for the web and one for the bottom flange. Figure 52 shows the model. The deck elements are slightly smaller over the girder.

It should be mentioned that there is a lack of compatibility between the quadratic shell elements representing the girder with linear solid elements modeling the deck girder boundary. However, the effect of this issue is not important at all because the fine mesh used is well capable of representing the deformation shape(s) expected under the loadings considered.

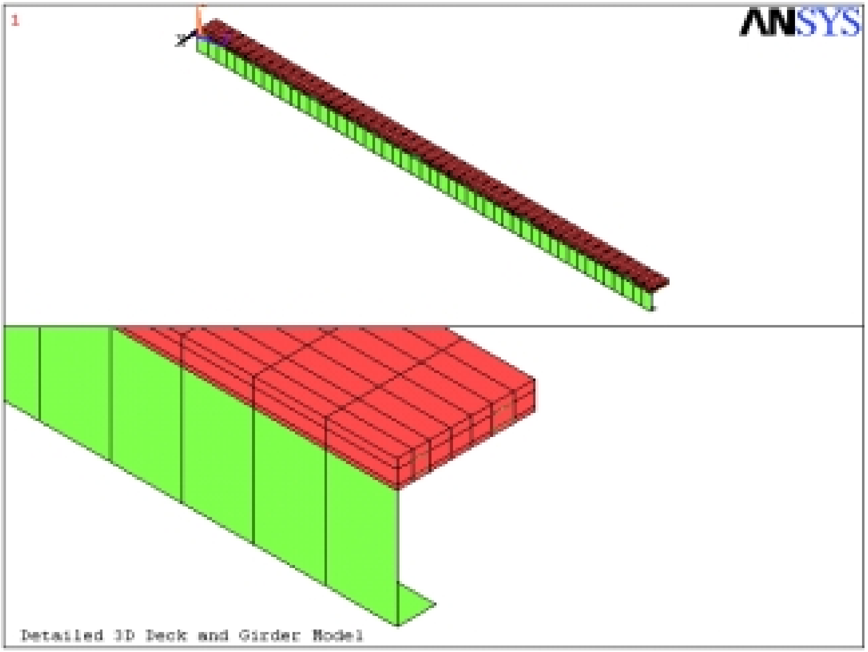


Figure 52. 3D finite element model.

Reinforcement Elements: Reinforcements are modeled with beam elements. Since the moment of inertia of these elements is very small they basically act as truss elements (truss elements produce analysis instability after cracking due to rotation at ends). These elements are placed in two directions and four layers along the boundary of the solid elements on a separate mesh of nodes. These four layers represents top and bottom mesh of reinforcement in longitudinal and transverse directions. To simplify modeling, spacing of truss elements is chosen to be equal to the spacing of the solid elements. Since the spacing of the rebar elements is not exactly equal to the actual reinforcement spacing per plans, the equivalent area of rebars are used for each layer of rebar element to have the same amount of reinforcement in the section. Furthermore to prevent bending of deck in transverse direction, moment of inertia of the transverse rebars are increased to prevent this bending.

Rebar elements are connected to solid element representing the deck with nonlinear springs modeling bond and slip of rebars in order to capture the behavior of the concrete and reinforcement realistically. Modeling bond-slip is essential to capturing post-cracking behavior of the concrete. Bond slip relationship proposed by Houde (1973) is used in this study, which is defined as follow:

$$u = 1.96E6 \times w - 2.35E9 \times w^2 + 1.39E12 \times w^3 - 0.33E15 \times w^4 \quad (7)$$

Where ***u*** is bond stress in psi and ***w*** is slip in inches. Figure 53 shows a plot of this equation.

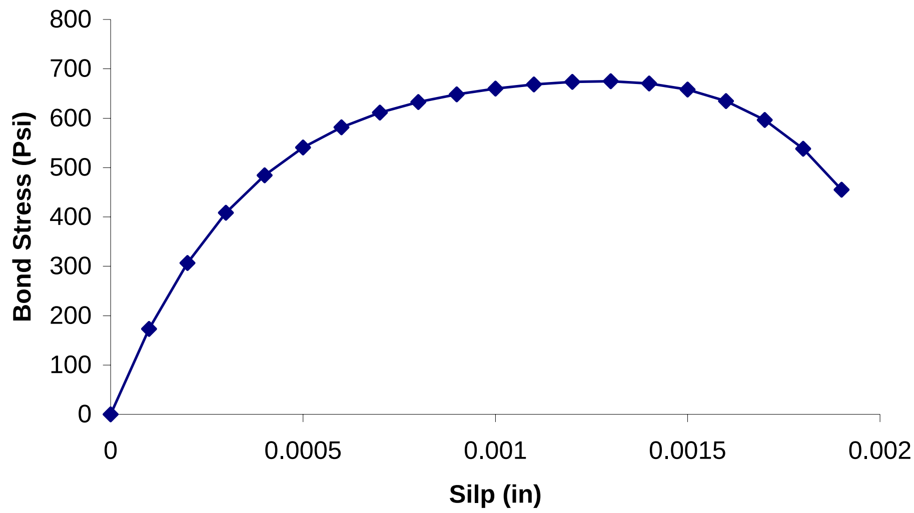


Figure 53. Bond-slip relationship (Houde, 1973).

Since finite element model includes this effect in a discrete manner, equivalent spring constant should be defined as follow:

$$k = u \times \pi d_{rebar} l_{element} \tag{8}$$

Where d_{rebar} is the bar diameter and $l_{element}$ is the element length.

Shear Stud Elements: Figure 54 shows shear stud details extracted from structural plans for the bridge under consideration. Shear studs are modeled using nonlinear spring elements. These elements connect nodes on the top of the girder to nodes on the bottom of the deck (which are geometrically coincident).

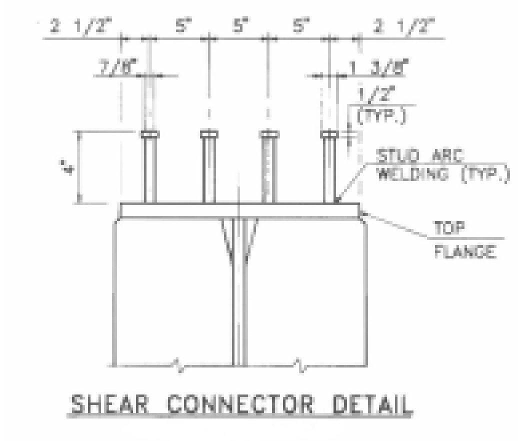


Figure 54. Shear Connector Detail.

Behavior of shear studs modeled using these springs with the load slip model (Yam and Chapman, 1968) shown in Figure 55. The referenced load slip behavior is obtained by back calculation of the composite section behavior by Yam and Chapman (1968). In the absence of substantial experimental data this relationship will be utilized.

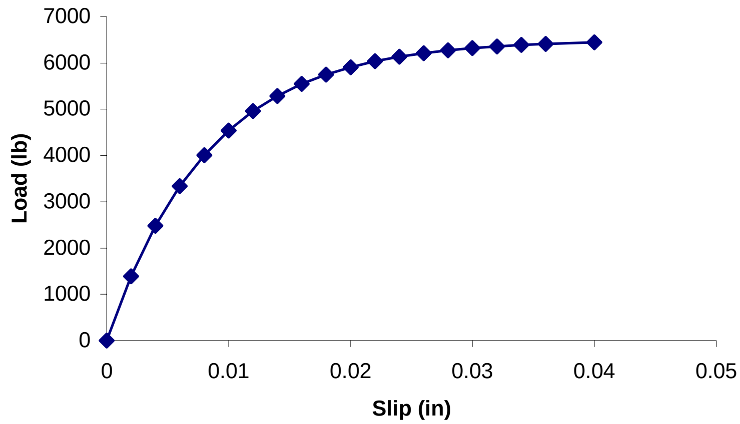


Figure 55. Shear Connector load slip behavior (Yam an Chapman, 1968).

Boundary Condition: The span modeled is simply supported. So the nodes on the bottom flange of the girder at one end is restrained in three directions and those at the other end is only restrained in the transverse and vertical directions. Also symmetric boundary condition is applied on the plane of symmetry (i.e., girder web plane).

Additionally longitudinal rebar nodes are constrained to move with deck in Z and X directions (See Figure 55 for directions) and transverse rebar nodes is constrained to move with the deck in Z and Y directions. Deck bottom nodes in contact with the girder top nodes are also constrained to move in the Z direction with these nodes.

*Concrete Compressive Strength***:** According to NJDOT specifications, a fixed compressive strength of 4500 psi is chosen. This is a good approximation, since the deck is cured at lease 7 days and after 7 days the change in compressive strength is not very large.

Concrete Modulus of Elasticity and Poisson's Ratio: ACI specification proposes the following equation for the value of modulus of elasticity (all values in psi)

$$E_c = 57000\sqrt{f_c'} \qquad (9)$$

Poisson's ratio for concrete at stresses lower than 0.7 f_c' falls within limits of 0.15 to 0.2 (Winter and Nilson, 1968). A value of 0.2 is used for the model and it is assumed constant for all ages.

Concrete Tensile Strength: According to recommendation of ACI the modulus of rupture is assumed equal to:

$$f_r = 7.5\sqrt{f_c'} \qquad (10)$$

Steel Modulus of Elasticity and Poisson's Ratio: The values of 29E6 psi and 0.3 will be used for steel modulus of elasticity and Poisson's ratio, repsectively.

Steel and Concrete Coefficient of Thermal Expansion: Winter and Nilson (1986) state that the concrete coefficient of thermal expansion should be in the range of 4E-6 to 7E-6 in/in per degree of Fahrenheit. Khan, Cook, and Mitchell (1998) measure the values of 5.3e-6 to 5.5e-6 in/in per ^{o}F for maturing normal weight concrete. In this study the values of 5.5E-6 and 6.25E-6 in/in per ^{o}F are assumed for concrete and steel, respectively.

Concrete Stress-Strain Curve: Since the magnitudes of compressive stresses are well within linear portion of the concrete strain stress curve a linear material model would be considered for compression. A linear stress strain curve is also assumed for concrete in tension before cracking.

Crack Modeling: Figure 56 shows the concrete material model in tension as defined by ANSYS (V5.5, 1998). It is further assumed that the shear stress transfer coefficient is 0.5 for open and close cracks. This value basically accounts for the rebar dowel action and aggregate interlocking at crack interface.

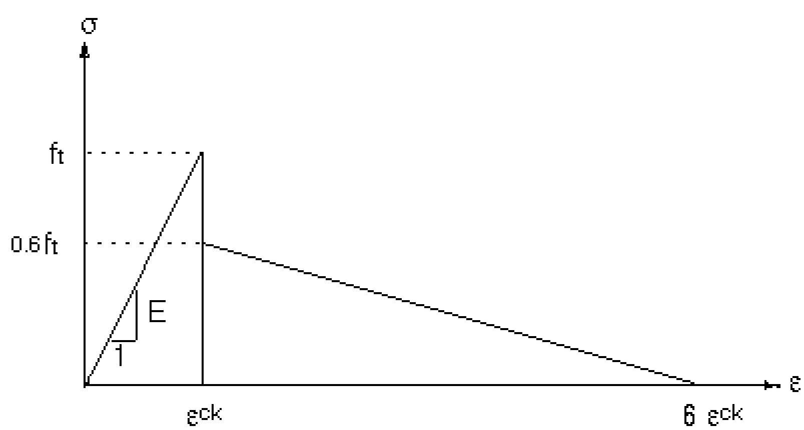


Figure 56. Concrete material model in tension.

Loading: The loading consists of three major components: drying shrinkage, thermal shrinkage, and ambient temperature. Note that creep effects are not modeled. However, considering the fact that the analyses are mostly concerned with behavior of the system in early age (less than 3 month), the results are useful and the error is negligible.

Results of experiments conducted as a part of this study, as shown in Figure 57, shows that after the first few days of pouring there is effectively no temperature difference between ambient temperature and the bridge deck in NJ. Also, field measurements show that hydration temperature rise produces no significant locked in residual stresses in the bridge deck after the first few days of pouring. Based on these two results only shrinkage-induced effects are considered in the 3D models and all models are subjected to a uniform shrinkage in the deck.

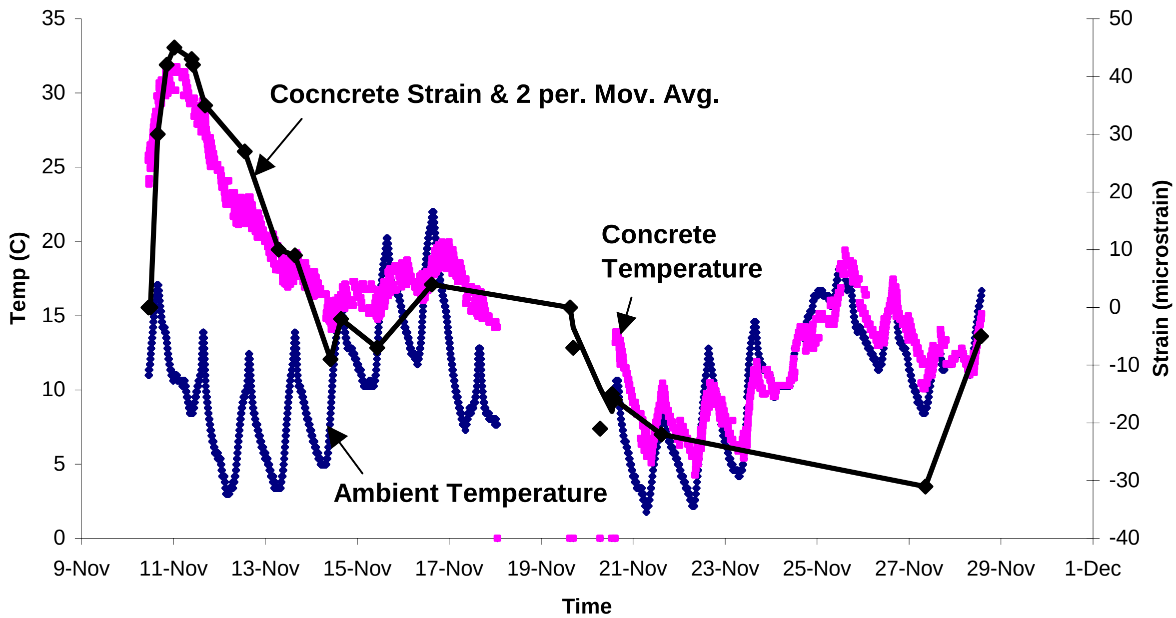


Figure 57. Measured concrete deck strain and temperature of a simply supported bridge.

Results of Three Dimensional Analyses

Crack patterns, stress history, and deflection history has been studied for four different types of boundary conditions. To study bridge response, a uniform shrinkage is applied to the deck and several response parameters are reported.

<u>Pin-Roller Boundary Condition</u>

Figure 58 shows the deflected shape of the structure at the end of the analysis. The cracks at mid span and at the end of the span are shown in Figure 59. Note that the circles in this figure indicate the plane of cracks. It is shown that except for a small portion at the ends, bridge deck develops transverse cracks as a result of applied shrinkage. Deflection at the center of the span vs. the shrinkage strain at mid span is shown in Figure 60. Downward deflection is negative. The sudden jump in deflection (bounce back) at a strain around 0.0004 in/in indicates transverse cracking of the bridge deck and stress relief.

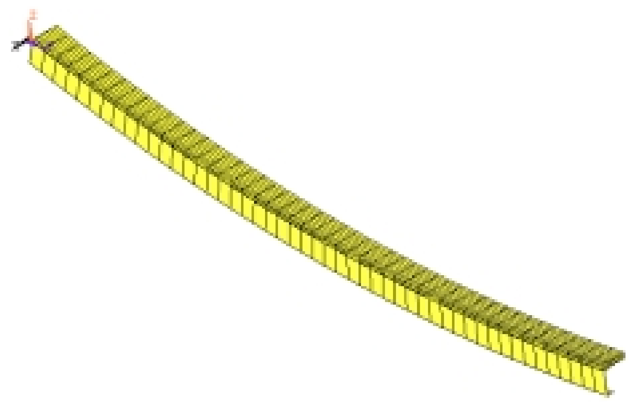


Figure 58. Deformed shape of the girder and cracked deck at the end of analysis (pin-roller boundary condition).

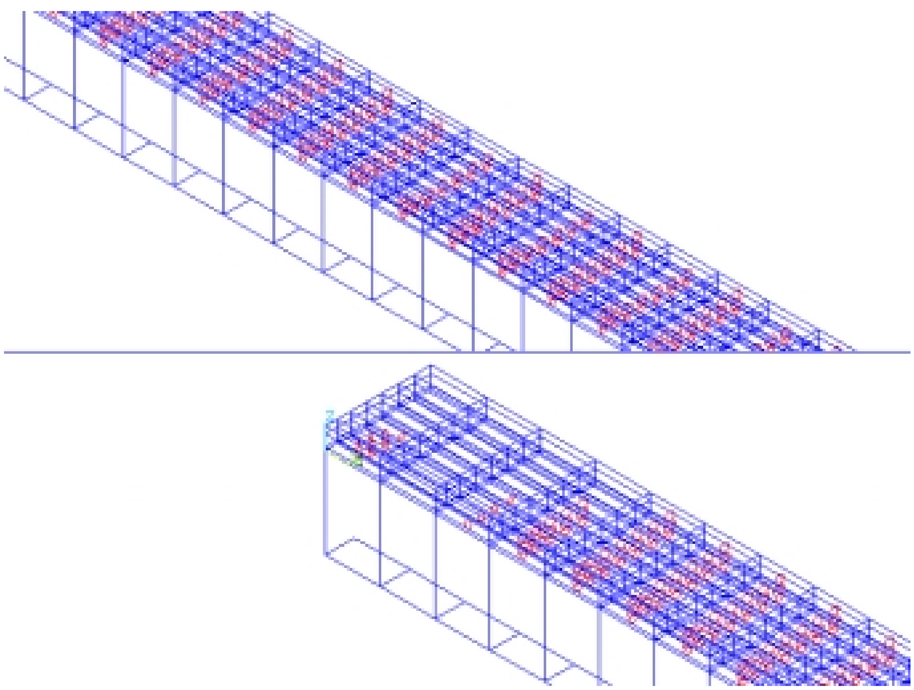


Figure 59. Deck cracks at end of span (bottom) and mid span (top) of the bridge at the end of analysis(pin-roller case).

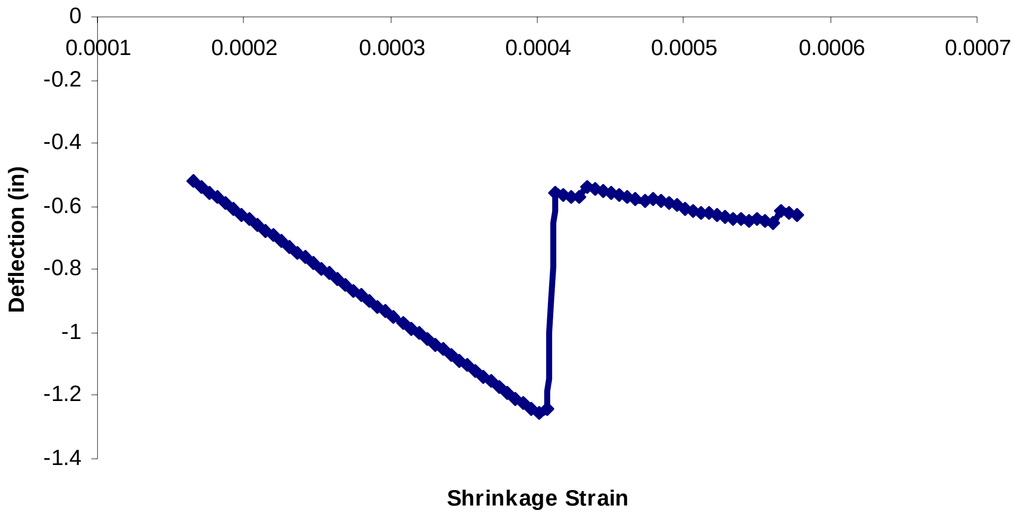


Figure 60. Mid span girder deflection vs. shrinkage strain (pin-roller case).

Figure 61 shows the concrete deck top and bottom stresses at mid span. It is noticeable that the deck top stress is far less than tensile strength of concrete (500 psi). However, it experiences the same jump as the deck bottom stress and the results show that the element at the top has cracked. This indicates that in this particular case cracking has started form bottom of the deck and propagates almost suddenly through the section. In other words, cracks are full depth.

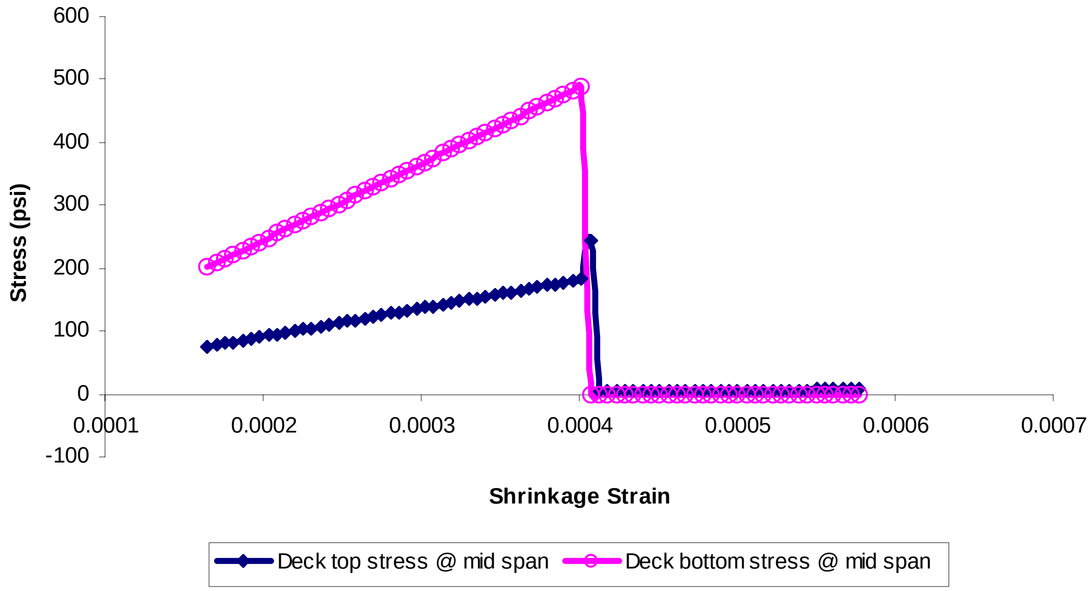


Figure 61. Deck top and bottom stress over girder at mid span for pin-roller boundary condition.

Deck top stresses at mid span and quarter span are compared in Figure 62. The sudden decrease of stress around the strain of 0.0004 in/in again shows development of cracking. Results show that both mid span and quarter span cracks develop at the same time. Results show that cracks at mid span stay open after cracking. However, due to stress redistribution cracks at quarter span close and as the graph shows compressive stresses develop at this region due to downward deflection of the bridge.

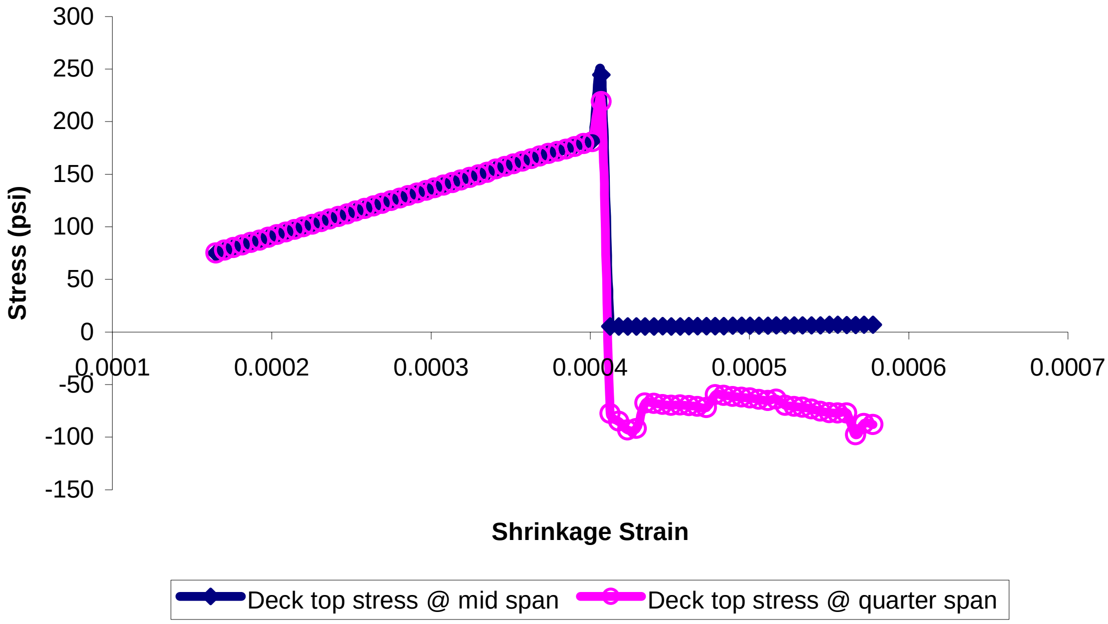


Figure 62. Deck top stress over the girder at mid span and quarter span for pin-roller boundary condition.

It is also observed that cracks are developed almost at the same time. Evaluation of the entire data indicates that, in fact the entire bridge cracks at once (e.g. Figures 61, 62, 63). Figure 63 shows the deck top stress across the deck at mid span at three locations. Location 1 is over girder, location 2 is midway between girder and edge of slab, and location 3 is slab edge. This figure supports the fact that cracks propagate transversely across the whole deck at the same time. This behavior is observed by Krauss and Rogalla (1996) in their experimental study of Portland-Columbia Bridge in Pennsylvania. They did not observe any cracks in their initial survey of the bridge after construction. However, in their next survey after a few days, they noticed that bridge deck has developed transverse cracks all over.

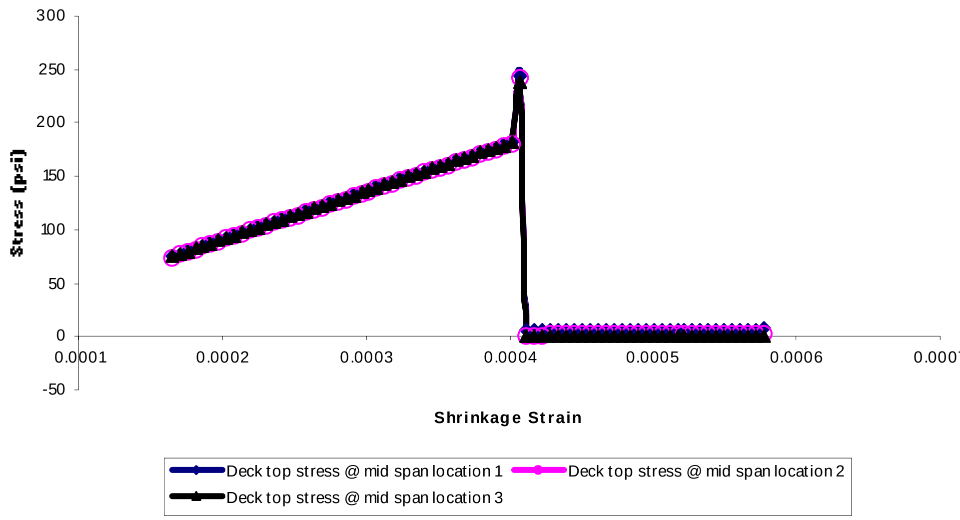


Figure 63. Deck top and bottom stress across the slab at mid span for pin-roller boundary condition.

As it can be seen from Figure 59 not all elements cracked during the analysis. In mid span, every other row of element developed full depth cracks across the slab. Considering that the length of an element is 22 in and each element has 4 integration points, which capture the cracking, crack spacing is 2.75 ft long. This spacing is the spacing between full depth cracks that run across the slab. This spacing as shown in Figure 59 reduces towards the ends of span.

Initiation of cracks can also be observed by sudden change in longitudinal reinforcement stress as shown in Figure 64. The longitudinal stress in reinforcement at mid span experiences a sudden increase followed by increasingly high stress with increasing shrinkage. In fact, in an experimental study Frosch et al. (2002) have observed yielding of longitudinal reinforcement as a result of cracking. Although the reinforcement stress does not reach yielding in FE analysis, the high stress level suggests that yielding might be possible in some cases.

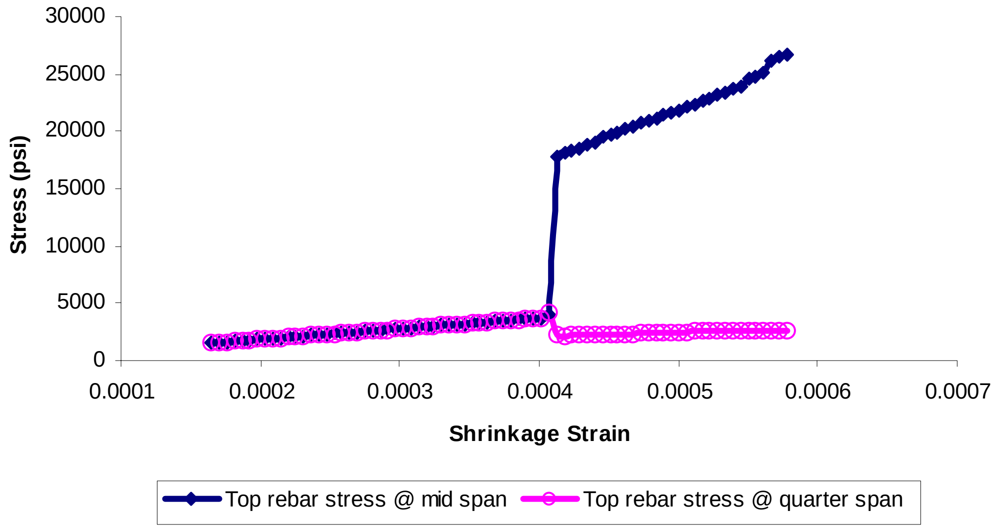


Figure 64. Reinforcement stress over girder at mid span and quarter span for pin-roller boundary condition.

Pin-Pin Boundary Condition

The response of a pin-pin bridge to shrinkage strains is similar to that of a pin-roller case. However, there are some differences that are discussed here. A diagram of cracks at the end of an analysis shown in Figure 65. As it can be seen almost all of the bridge developed full depth transverse cracks. Deflection history is shown in Figure 66. The deflection curve is similar to that for a pin roller case but the maximum deflection is lower indicating the increase in stiffness of the structure due the change in boundary condition. There in not much difference in the level of strain that causes cracking. Cracks develop around 0.0004 in/in shrinkage strain. This result was expected based on the 2D finite element results.

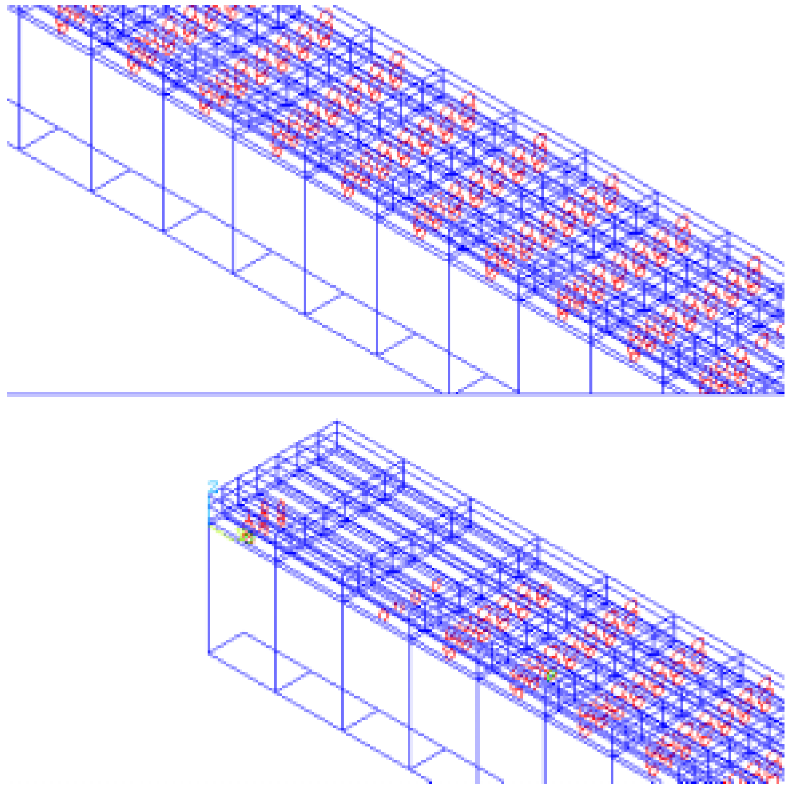


Figure 65. Deck cracks at end of span (bottom) and mid span (top) of pin-pin bridge at the end of analysis. Circles indicate cracks for each element.(pin-pin case).

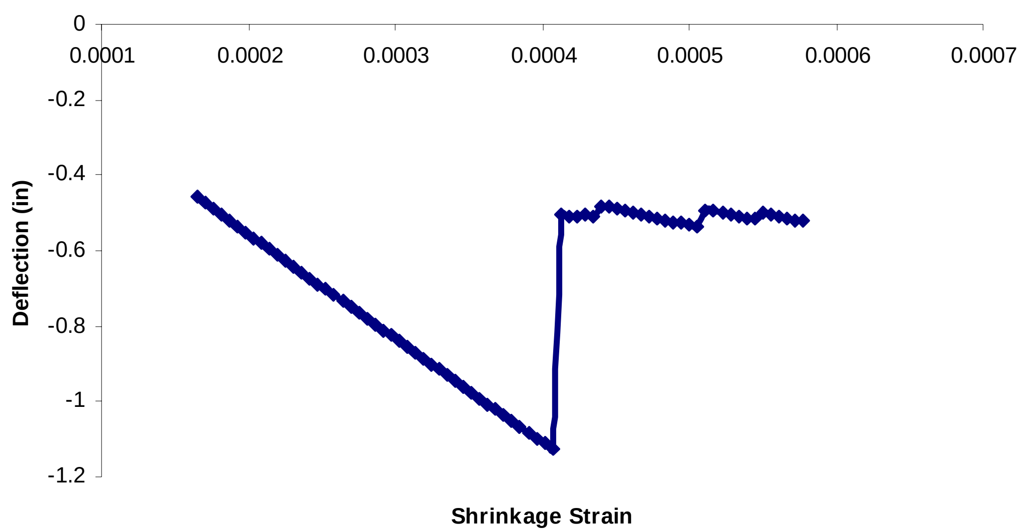


Figure 66. Mid span girder deflection vs. shrinkage strain for pin-pin boundary condition.

The stress histories at two points through the depth of the deck at mid span are shown in Figure 67. Similar to the pin-roller case cracks are full depth and start from the bottom. As it can be seen from Figure 68 stress redistribution results in development of compressive stress in the top of the deck.

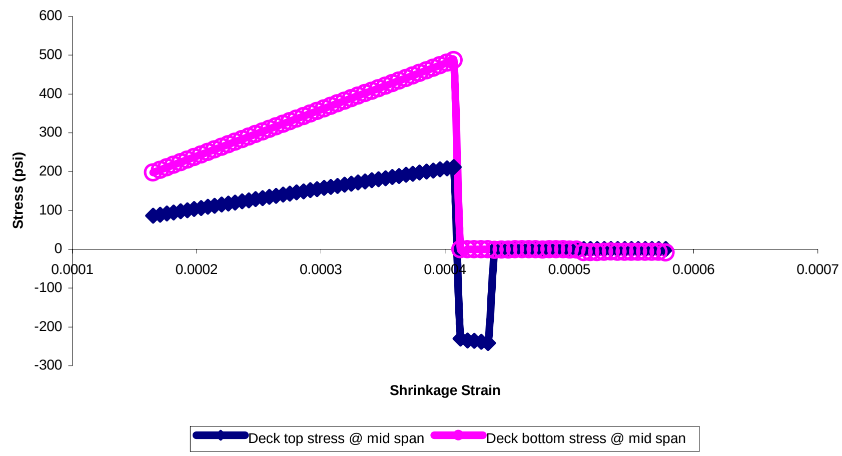


Figure 67. Deck top and bottom stress over girder at mid span for pin-pin boundary condition.

Figure 68 shows a comparison of the deck top stresses at mid span and quarter span, while Figure 69 shows the stress distribution across the deck at mid-span. In this figure location 1 is over girder, location 2 is midway between girder and edge of slab, and location 3 is slab edge.

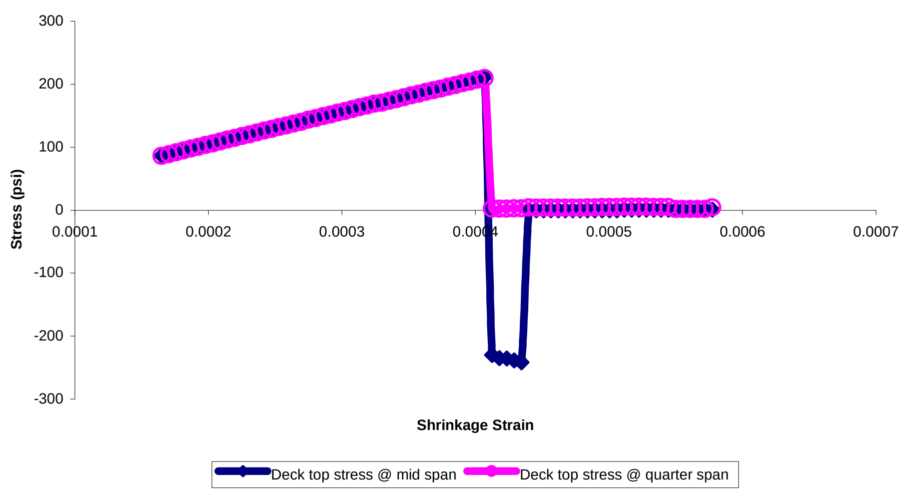


Figure 68. Deck top stress over the girder at mid span and quarter span for pin-pin boundary condition.

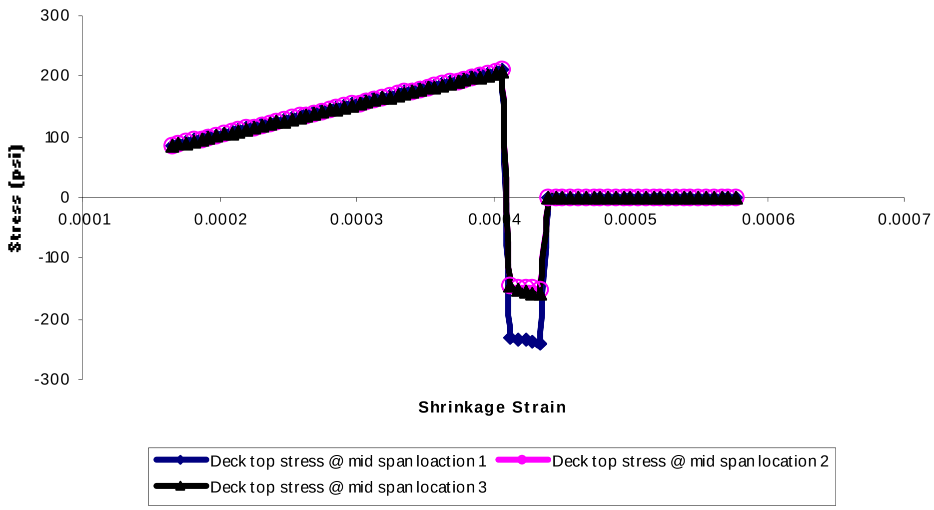


Figure 69. Deck top and bottom stress across the slab at mid span for pin-pin boundary condition.

As it can be seen from Figure 65, more full depth cracks are developed in this case. So, the spacing between full depth cracks that run across the span is lower. This spacing is estimated to be 1 ft based of the mesh size for FE model. However, for a better estimate finer mesh is required. Longitudinal reinforcement stresses are displayed in Figure 70. As shown in this figure, the top and bottom reinforcements develop high stresses after cracking.

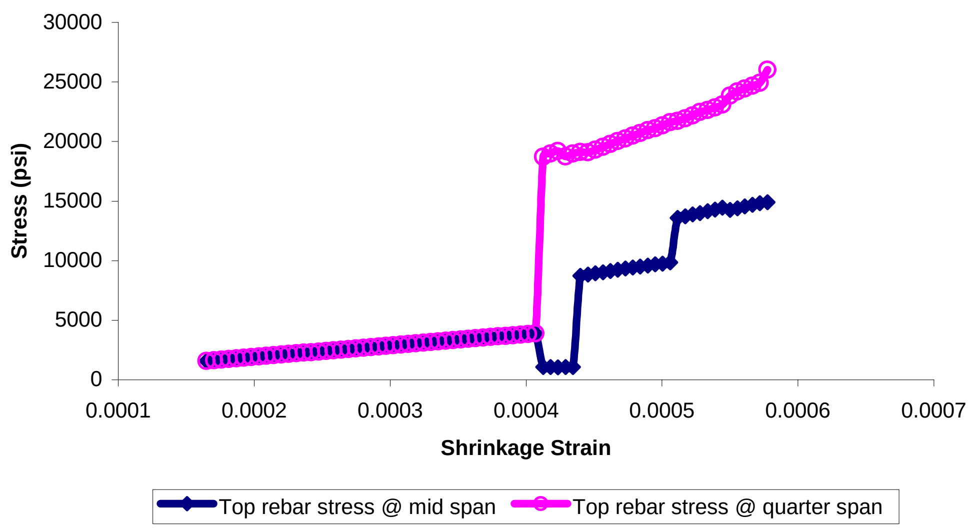


Figure 70. Reinforcement stress over girder at mid span and quarter span for pin-roller boundary condition.

Fixed-Roller Boundary Condition

The deformed shape of fixed-roller bridge is shown in Figure 71. Figure 72 shows the cracks at the end of the analysis. Circles indicate plane of cracking in Figure 73. It is observed that the entire bridge deck is cracked. While cracks are spread right to the fixed end support, they do not appear near the roller end. Deflection history at mid-span is shown in Figure 73. Unlike the previous cases there is no sudden jump in the deflection curve but rather deflection re-bounces in several steps suggesting that cracks do not happen all at once. The re-bounce starts around strain of 0.0002 in/in, which is lower than previous cases but expected due to higher stiffness of a fixed-roller beam compared to pin-roller or pin-pin case. Similarly, deflection values are far less than the previous cases.

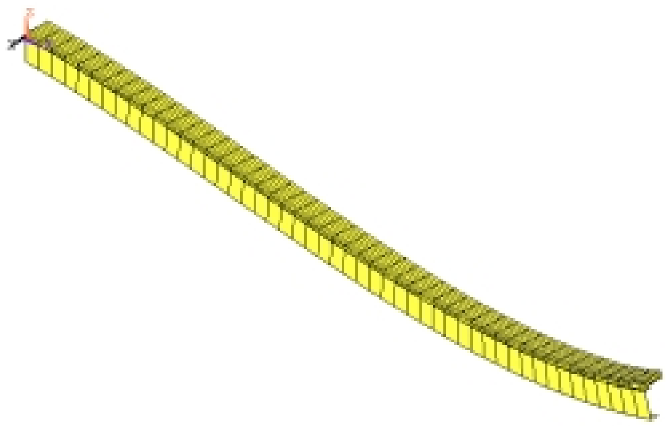


Figure 71. Deformed shape of the girder and cracked deck at the end of analysis (fixed-roller boundary condition).

Figures 74 and 75 show the deck top and bottom stress at different locations along the span. It is apparent from these graphs that transverse cracks start from fixed end and as the shrinkage strain increases they spread towards the roller end. These cracks are again full depth and as the results show extend across the slab. It is also noticeable that cracks happen at a stress lower than tensile strength of concrete. This is due to multi-axial nature of stresses at the time of cracking.

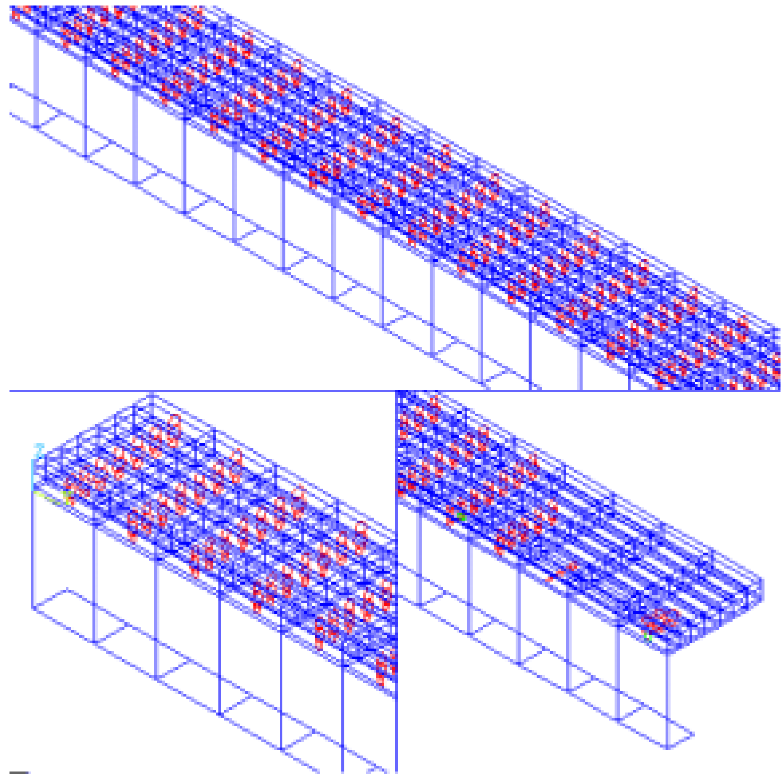


Figure 72. Deck cracks at fixed end of span (bottom left), roller end of span (right end) and mid span (top) of fixed-roller bridge at the end of analysis.

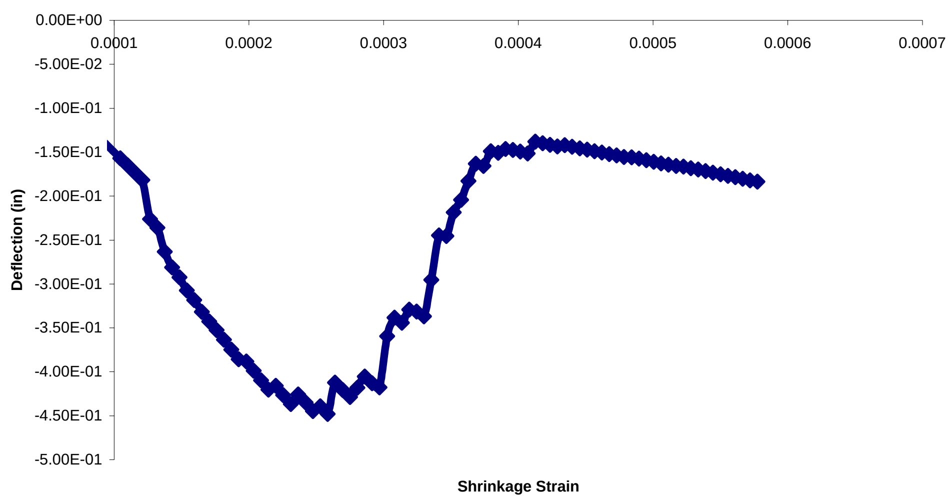


Figure 73. Mid span girder deflection vs. shrinkage strain for pin-pin boundary condition.

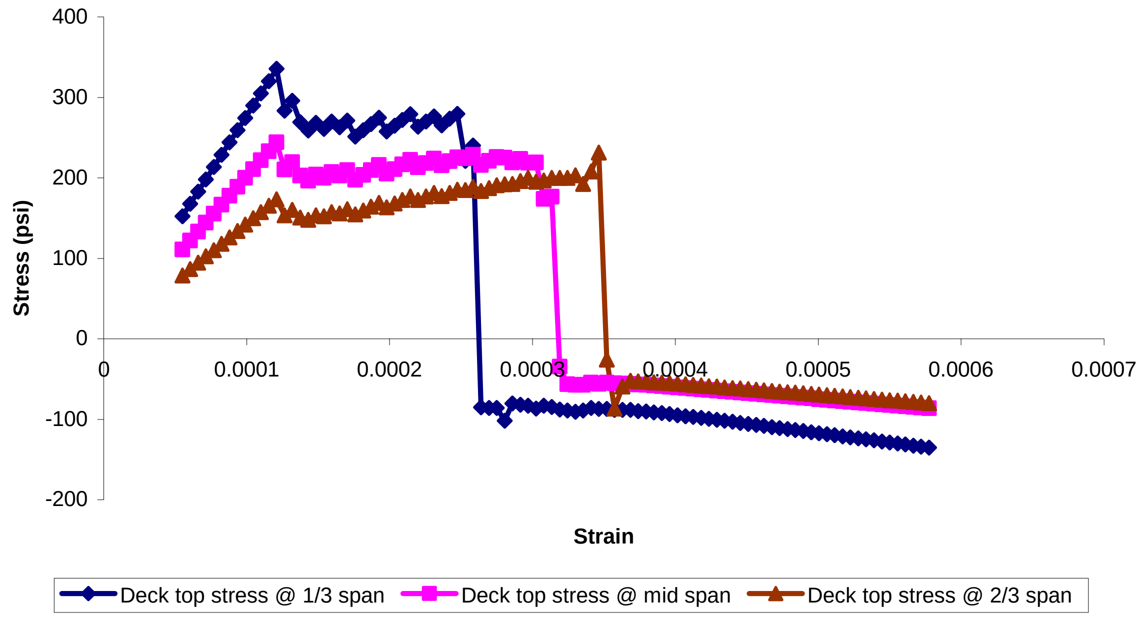


Figure 74. Deck top stress at 1/3, half and 2/3 of span from fixed end for fixed-roller boundary condition.

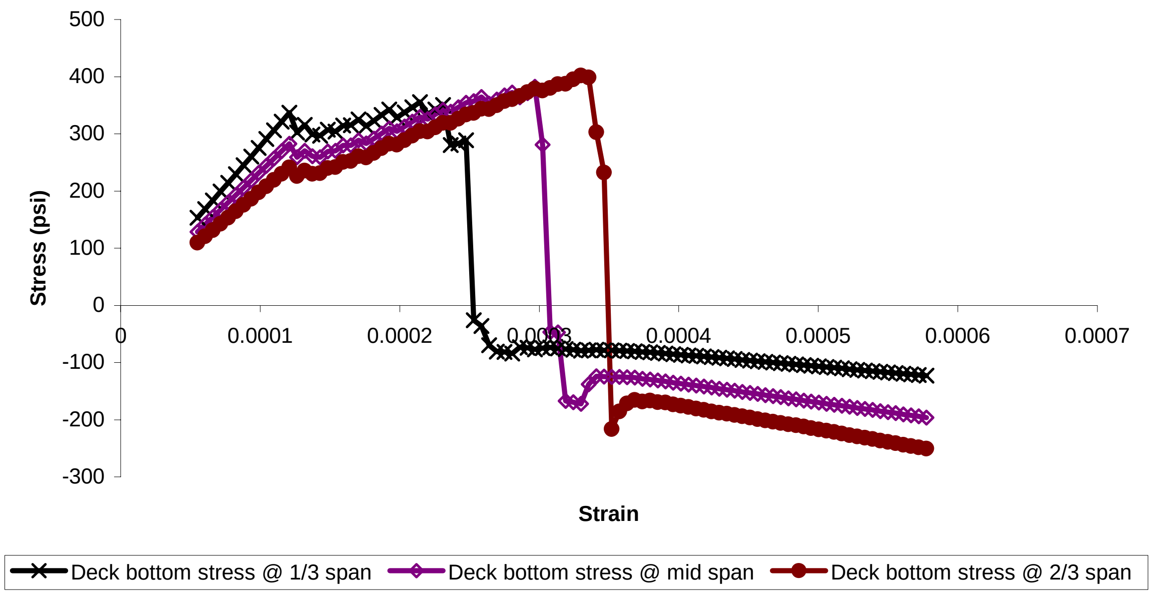


Figure 75. Deck bottom stress at 1/3, half and 2/3 of span from fixed end for fixed-roller boundary condition.

Figure 76 shows the reinforcement stress along the span. There is an increase in reinforcement stress after cracking, which were observed in previous cases too.

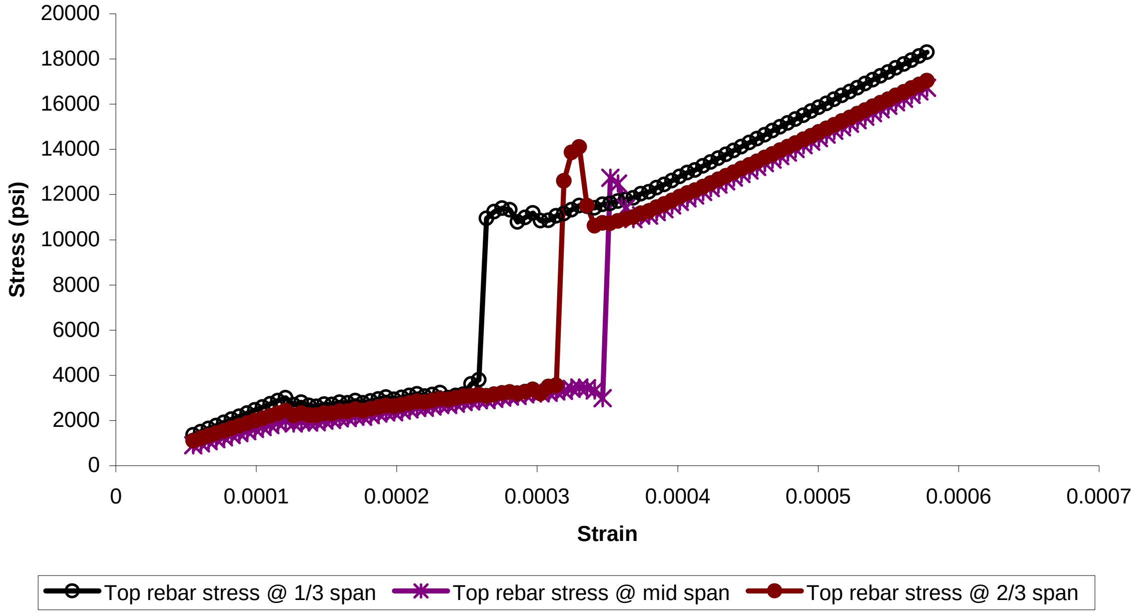


Figure 76. Top reinforcement stress at 1/3, half and 2/3 of span from fixed end for fixed-roller boundary condition.

Fixed-Fixed Boundary Condition

The behavior of the structure before cracking in the case of a fixed-fixed bridge is exactly similar to the classical example of two different materials connected together and undergoing temperature difference. Since the deck and girder are fixed at both ends deflection due to shrinkage is negligible and deck stress is equal to shrinkage strain times the modulus of elasticity for concrete. This stress is the same all over the slab. The moment that the stress level reaches the tensile strength of concrete every element in the deck cracks. Figure 77 shows the stress history of a deck element. The same pattern is observed for all other elements in the deck. It is noticeable that the level of shrinkage strain when cracking occurs is considerably lower for this case. This has also been shown using the 2D models and it is due to higher stiffness of the system.

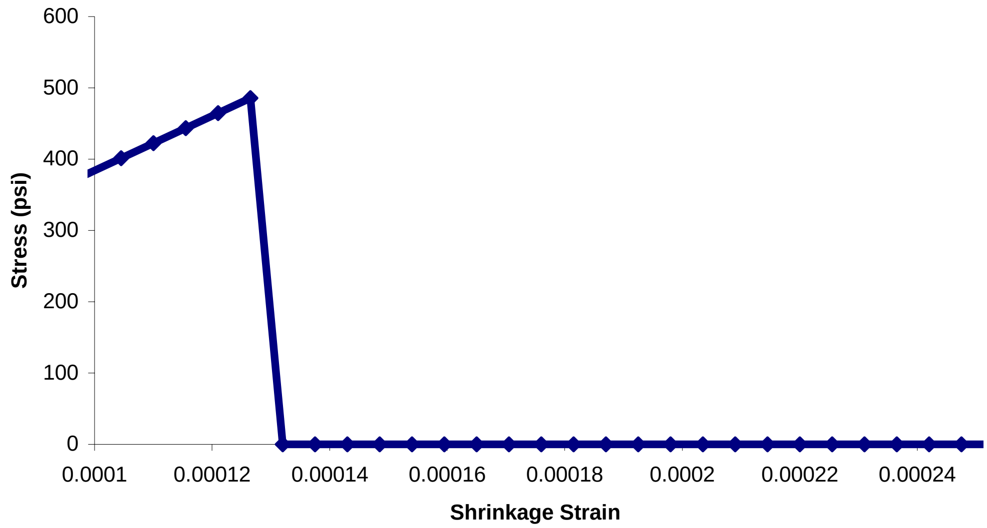


Figure 77. Stress of deck elements for fixed-fixed boundary condition.

Based on FEA results, a bridge with fixed-fixed boundary condition undergoes the most severe cracking while the level of shrinkage is the lowest. This means that fixed-fixed bridges have higher potential for transverse deck cracking. A point that must be explicitly considered in their design. An easy to use program that can facilitate determination of tensile stresses in bridge decks is developed under this study. This is discussed in Chapter six.

Summary of Results of Three Dimensional Analysis

3D analyses of a typical bridge with four different types of boundary conditions were presented and discussed in light of crack pattern, stress and deflection time histories, and distribution of stresses. The results can be summarized as follow:

1. Cracks develop suddenly all over the deck in bridges with pin-roller, pin-pin, and fixed boundary conditions. However, for fixed roller boundary condition cracks spread along the span as the shrinkage strain increases.
2. Cracks are full depth and run across the slab.
3. Crack spacing for simply supported bridges in estimated to be 2.75 ft and it reduces as the rigidity of the boundary condition increases.
4. There is a sudden increase in reinforcement stress after cracking which may cause rebars to yield.
5. Deflection of the bridge reduces as cracks develop and stresses are relieved.

A SIMPLE METHOD FOR ESTIMATING STRESSES CAUSING TRANSVERSE CRACKING IN CONCRETE BRIDGE DECKS

This chapter describes a simple method to estimate stresses causing transverse cracking in concrete bridge decks with full composite action between deck and girder. A system of equations for a pin - roller supported composite girder is developed to estimate the deck stresses due to volume change in concrete caused by shrinkage and/or other effects. These equations are further extended to consider different boundary conditions. The results of this method agree very well with the results of finite element analyses. It is proposed that this practical method be used as a tool during bridge design to examine concrete bridge deck against the possibility of transverse cracking. To facilitate this, a MS Windows application is developed that provides a simple mean for designers to perform AASHTO (3.12) checks on shrinkage and temperature loading. Furthermore, an overview of factors causing the volume change in bridge deck concrete and their magnitude is also presented herein to simplify the use of this method.

Compatibility Equations

The basic method of solid mechanics is used to derive a set of equations for computing internal stresses due to an assumed temperature profile. Figure 78 shows a composite girder with bilinear temperature profile in the deck and girder. To compute internal stresses caused by this temperature profile in a pin-roller supported composite deck-girder system, the composite section is divided into four sections, each with linear temperature changes. Sections are numbered from one to four beginning from the top (Figure 79). Section 1 and 2 divide deck and section 3 and 4 divide girder. Internal stresses transferred between the sections and between sections and reinforcements are replaced by their resultant force and moments (Figure 79).

To calculate internal force and moment resultants, eight compatibility equations should be solved simultaneously. These equations ensure the strain and curvature compatibility among sections and reinforcements. Equations 1, 2, and 3 enforce strain equality at the interface of the separated sections. Equations 4, 5, and 6 ensure curvature equality in all sections and equations 8 and 9 ensure strain equality between reinforcement and surrounding concrete matrix. The current form of the equations, consistent with design practice, assumes that two layers of longitudinal reinforcements exist. First layer of reinforcement is located in section one and the other layer is in section two (Figure 79). However, the equations can be easily modified for more than two layers of reinforcements.

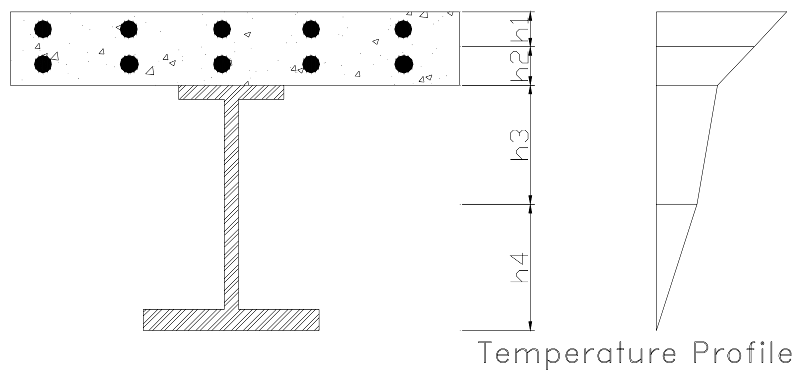


Figure 78. Temperature profile along the section.

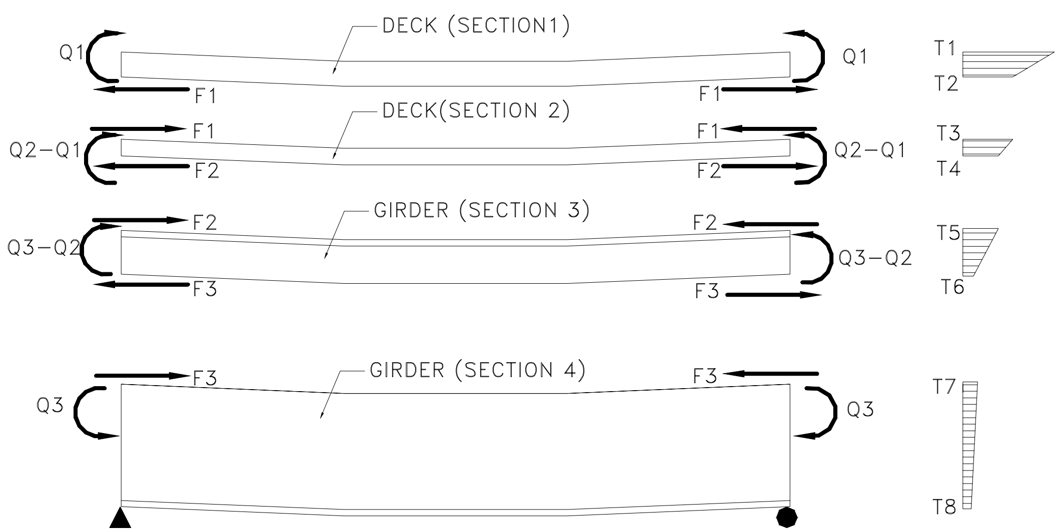


Figure 79. Compatibility forces and moments and temperature distribution.

$$\varepsilon_{Bottom1} = \alpha_1 (T_2 - T_0) + \frac{Q_1}{E_1 S_{b1}} + \frac{F_1}{E_1 A_1} - \frac{Fr_1}{E_1 A_1} + \frac{F_1 d_{b1}}{E_1 S_{b1}} + \frac{Fr_1 (d_{t1} - C_{Top})}{E_1 S_{b1}} =$$
$$\varepsilon_{Top2} = \alpha_2 (T_3 - T_0) + \frac{Q_2 - Q_1}{E_2 S_{t2}} + \frac{F_2 - F_1}{E_2 A_2} - \frac{Fr_2}{E_2 A_2} + \frac{F_2 d_{b2} + F_1 d_{t2}}{E_2 S_{t2}} - \frac{Fr_2 (d_{b2} - C_{Bottom})}{E_2 S_{t2}} \tag{11}$$

$$\varepsilon_{Bottom2} = \alpha_2(T_4 - T_0) + \frac{Q_2 - Q_1}{E_2 S_{b2}} + \frac{F_2 - F_1}{E_2 A_2} - \frac{Fr_2}{E_2 A_2} + \frac{F_2 d_{b2} + F_1 d_{t2}}{E_2 S_{b2}} - \frac{Fr_2(d_{b2} - C_{Bottom})}{E_2 S_{b2}} =$$

$$\varepsilon_{Top3} = \alpha_3(T_5 - T_0) + \frac{Q_3 - Q_2}{E_3 S_{t3}} + \frac{F_3 - F_2}{E_3 A_3} + \frac{F_3 d_{b3} + F_2 d_{t3}}{E_3 S_{t3}} \tag{12}$$

$$\varepsilon_{Bottom3} = \alpha_3(T_6 - T_0) + \frac{Q_3 - Q_2}{E_3 S_{b3}} + \frac{F_3 - F_2}{E_3 A_3} + \frac{F_3 d_{b3} + F_2 d_{t3}}{E_3 S_{b3}} =$$

$$\varepsilon_{Top4} = \alpha_4(T_7 - T_0) - \frac{Q_3}{E_4 S_{t4}} - \frac{F_3}{E_4 A_4} + \frac{F_3 d_{t4}}{E_4 S_{t4}} \tag{13}$$

$$\frac{1}{R_1} = \frac{\alpha_1(T_2 - T_1)}{h_1} + \frac{Q_1}{E_1 I_1} + \frac{F_1 d_{b1}}{E_1 I_1} + \frac{Fr_1(d_{t1} - C_{Top})}{E_1 I_1} =$$

$$\frac{1}{R_2} = \frac{\alpha_2(T_4 - T_3)}{h_2} + \frac{Q_2 - Q_1}{E_2 I_2} + \frac{F_1 d_{t2} + F_2 d_{b2}}{E_2 I_2} - \frac{Fr_2(d_{b2} - C_{Bottom})}{E_2 I_2} \tag{14}$$

$$\frac{1}{R_2} = \frac{\alpha_2(T_4 - T_3)}{h_2} + \frac{Q_2 - Q_1}{E_2 I_2} + \frac{F_1 d_{t2} + F_2 d_{b2}}{E_2 I_2} - \frac{Fr_2(d_{b2} - C_{Bottom})}{E_2 I_2} =$$

$$\frac{1}{R_3} = \frac{\alpha_3(T_6 - T_5)}{h_3} + \frac{Q_3 - Q_2}{E_3 I_3} + \frac{F_2 d_{t3} + F_3 d_{b3}}{E_3 I_3} \tag{15}$$

$$\frac{1}{R_3} = \frac{\alpha_3(T_6 - T_5)}{h_3} + \frac{Q_3 - Q_2}{E_3 I_3} + \frac{F_2 d_{t3} + F_3 d_{b3}}{E_3 I_3} =$$

$$\frac{1}{R_4} = \frac{\alpha_4(T_8 - T_7)}{h_4} - \frac{Q_3}{E_4 I_4} + \frac{F_3 d_{t4}}{E_4 I_4} \tag{16}$$

$$\varepsilon_{\mathrm{Re}bar1} = \alpha_{r1}(T_{r1} - T_0) + \frac{Fr_1}{E_{r1} A_{r1}} = \tag{17}$$

$$\alpha_1(T_{r1}-T_0)+\frac{Q_1}{E_1S_{r1}}+\frac{F_1}{E_1A_1}-\frac{Fr_1}{E_1A_1}+\frac{F_1d_{b1}}{E_1S_{r1}}+\frac{Fr_1(d_{t1}-C_{Top})}{E_1S_{r1}}$$

$$\varepsilon_{\mathrm{Re}bar1}=\alpha_{r2}(T_{r2}-T_0)+\frac{Fr_2}{E_{r2}A_{r2}}=$$

$$\alpha_2(T_{r2}-T_0)+\frac{Q_2-Q_1}{E_2S_{r2}}+\frac{F_2-F_1}{E_2A_2}-\frac{Fr_2}{E_2A_2}+\frac{F_2d_{b2}+F_1d_{t2}}{E_2S_{r2}}-\frac{Fr_2(d_{b2}-C_{Bottom})}{E_2S_{r2}} \tag{18}$$

In these equations $\varepsilon_{Bottomi}$ and ε_{Topi} are strains at bottom and top of section i. R_i is the curvature of section i, F_i and Q_i are the force and moment resultant of stresses at interface of section i and i+1, $E_i, h_i, A_i, I_i, S_{ti}, S_{bi}$, and α_i are respectively modulus of elasticity, height, area, moment of inertia, top section modulus, bottom section modulus and coefficient of thermal expansion of section i. d_{bi} and d_{ti} are respectively the distance from centroid to bottom and top of section. $Fr_i, Sr_i, \alpha_{ri}, C_{Bottom}$, and C_{Top} represent respectively force resultant of stresses in reinforcement layer i, section modulus at level of reinforcement layer i, coefficient of thermal expansion of reinforcement layer i, and bottom reinforcement cover and top reinforcement cover. T_i and Tr_i define the temperature distribution in section as shown in Figure 79.

Upon calculating forces and moments required to satisfy compatibitity equations, the deck stresses can be obtained by using the following equations:

$$\sigma_{DeckBottom}=\frac{F_2-Fr_2-F_1}{A_2}+\frac{F_2d_{b2}+Q_2-Q_1+F_1d_{t2}-Fr_2(d_{b2}-C_{Bottom})}{S_{b2}} \tag{19}$$

$$\sigma_{DeckTop}=\frac{F_1-Fr_1}{A_1}+\frac{F_1d_{b1}+Q_1+Fr_1(d_{t1}-C_{Top})}{S_{t1}} \tag{20}$$

In these equations $\sigma_{DeckBottom}$ and $\sigma_{DeckTop}$ are respectively deck bottom and top stresses. Equations 11 through 20 can be applied to a cantilever system too. Since the same representation of internal forces and the same compatibility equations (equations 11-18) are used, similar system of equations characterizes the behavior of cantilever composite beam. This results in identical stresses from equation 9 and 10 for pin-roller and cantilever systems. However, due to boundary conditions deflection curves would be different for these two cases.

Adequacy of Assumptions

There are two basic assumptions in deriving compatibility equations; namely (i) the assumption that approximating the internal stresses along the boundary of sections and reinforcements with resultant forces and moments is a good approximation, and (ii) the assumption that deck is in plane stress condition.

Deformations and strains computed based on resultant forces and moments do not capture the effect of distributed nature of internal stresses and consequently do not result in exact solution. However, the deviation from exact solution is not significant. In fact the accuracy of equations depends on nature of stress distribution along the boundary of sections and reinforcements, which in turn depends on temperature profile.

To evaluate the accuracy of this assumption and for comparison of results, a 2-D Finite Element (FE) model of a pin-roller supported girder and the tributary portion of the deck with top and bottom reinforcements is developed using ANSYS FE package (1998) (described completely in chapter 5). Deck and girder are modeled using plane stress elements with different thickness and reinforcements are modeled using truss elements attached over the boundary of deck elements (Figure 80). The FE model is developed using an actual bridge dimensions and properties, which developed transverse deck cracks. The bridge data shown in Table 27 are used as an example in this paper. Deck and girder model of Table 27 are subjected to arbitrary 5.6^{o}C (10^{o}F) uniform temperature decrease in the deck. Figure 81 shows the distribution of nodal forces in longitudinal direction between the deck and the girder. Similarly, Figure 82 shows distribution of internal nodal forces in a direction normal to the plane of the deck for the same model. As it can be seen from these figures, internal nodal forces are concentrated at both ends and justify resultant representation between sections 2 and 3 as shown in Figure 79. However based on beam theory, resultant representation of internal stresses between sections 3 and 4 ignores the distributed nature of stresses. This introduces deviation from exact solution. However as results in Table 28 show, this deviation is not significant.

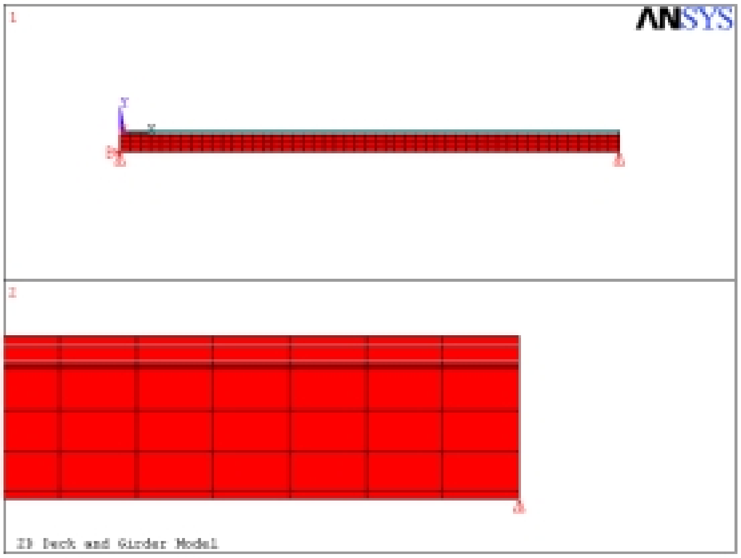


Figure 80. Finite element model of deck and girder (ANSYS V5.5, 1998).

Table 27. Design characteristic of modeled bridge.

Design Characteristic	Value
Span Length	27.48-m (1082-inch)
Span Width	14.33-m (564-inch)
Girder Spacing	7 Space@2.16-m (85-inch)
Deck Width per Girder	2.16-m (85-inch)
Deck thickness	21.6-cm (8.5-inch)
Top Longitudinal Bars	#5@38.1-cm (15-inch)
Bottom Longitudinal Bars	#5@15.2-cm (6-inch)
Top and Bottom Transverse Bars	#6@18-cm (7"-inch)
Girder Bottom Flange Thickness	5-cm (2-inch)
Girder top Flange Thickness	3.17-cm (1.25-inch)
Girder Bottom Flange Width	61-cm (24-inch)
Girder Bottom Top Width	50.8-cm (20-inch)
Girder Web Thickness	2.5-cm (1-inch)
Girder Web Height	89.5-cm (35.25-inch)
Deck Top Cover	6.4-cm (2.5-inch)
Deck Bottom Cover	2.5-cm (1-inch)

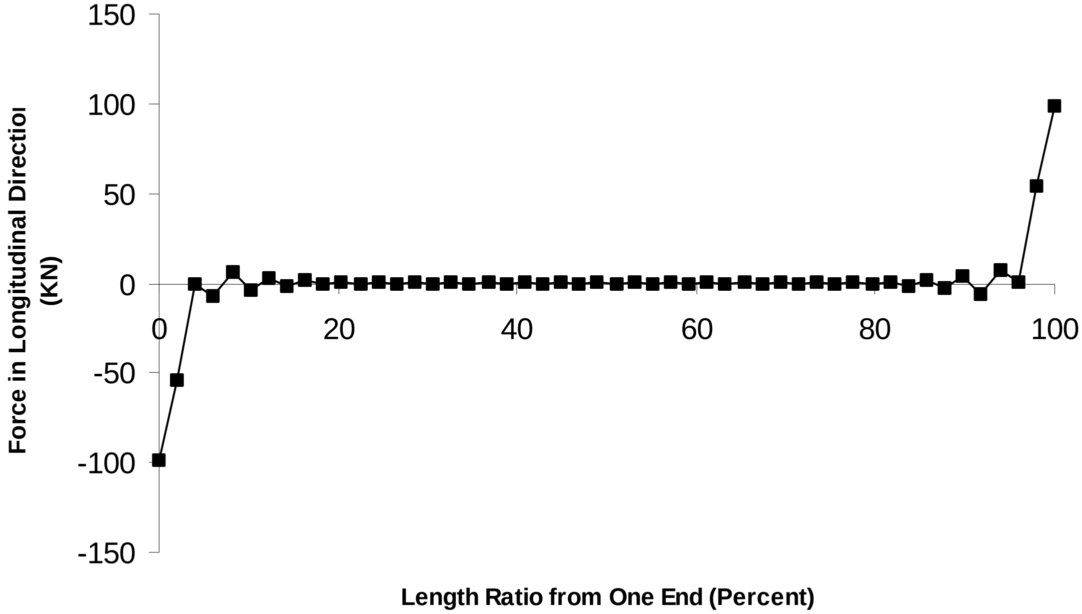


Figure 81. Distribution of nodal forces in longitudinal direction for pin-roller deck girder system between deck and girder.

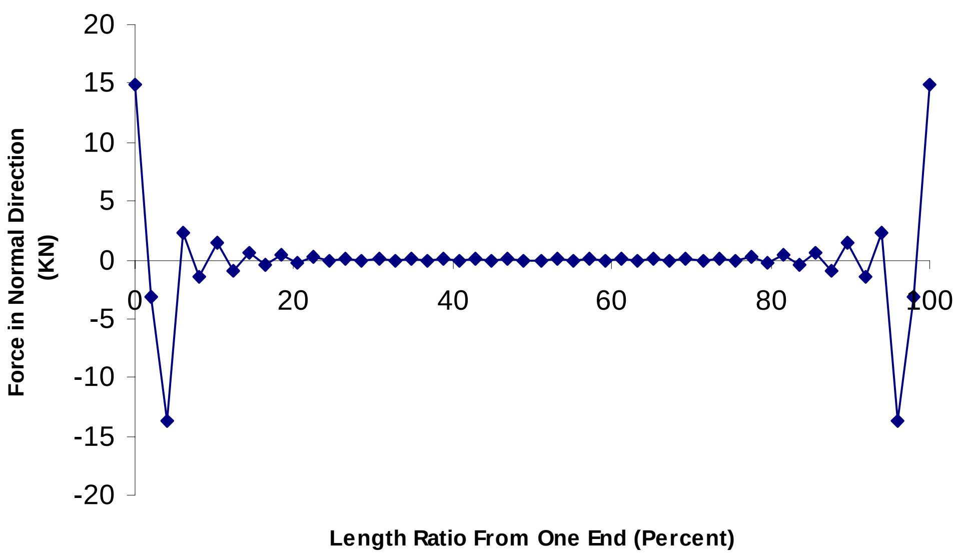


Figure 82. Distribution of nodal forces in direction normal to deck for pin-roller deck girder between deck and girder.

Table 28. Comparison of FEA and compatibility equations results for the pin-roller supported deck-girder system.

	Deck Bottom Stresses KPa (PSI)	Deck Top Stresses KPa (PSI)
Compatibility Equations	434.16 (62.97)	167.06 (24.23)
2D FEA	434.23 (62.98)	167.06 (24.23)

Another assumption in developing the set of equations is that the composite deck is under plane stress condition. Table 29 shows the effect of plane stress vs. plane strain assumption on deck stresses for the example bridge. This table shows that there is a considerable difference between plane stress and plane strain assumptions.

Table 29. Comparison of compatibility equations for plane stress and plane strain assumption.

	Deck Bottom Stresses KPa (PSI)	Deck Top Stresses KPa (PSI)
Plane stress	434.16 (62.97)	160.17 (24.23)
Plane strain	669.00 (97.03)	213.12 (30.91)

A detailed 3-D FE analysis is performed to verify the plane stress assumption. The results of this analysis support the plane stress case (see chapter 5 for details).

Results and Comparison to Finite Element Analysis

To evaluate the results obtained from equations 1-10, a 2-D Finite Element (FE) model of Figure 80, as described earlier, is analyzed when subjected to 5.6^{o}C (10^{o}F) uniform temperature drop in the deck. Table 28 presents deck stresses computed using the compatibility equations and those for 2D finite element analysis. The results are almost identical.

Other Boundary Conditions

Equations 1 through 10 estimate deck stresses in the case of pin-roller supported or cantilever girder. Computation of stresses for other than pin-roller or cantilever boundary condition utilizes the principle of superposition. Either a pin-roller supported or a cantilever system can be chosen as base case. Any extra boundary conditions will be considered as redundant boundary condition. The redundant boundary conditions impose additional forces on the system to satisfy deformation compatibility. These forces can be obtained using compatibility requirements. After calculating redundant forces, deck stresses is calculated by addition of stresses estimated by equations 1 through 10 to the stresses caused by redundant forces. Validity of the final results is guaranteed by the principal of superposition for elastic structures.

Figure 83 shows the application of the principle of superposition to different boundary conditions. Numerical results of application of this method to the example bridge section, (with different boundary conditions) subjected to a 5.6°C (10°F) uniform temperature decrease in the deck are shown in Table 30. The equations used for obtaining these results are presented in the following sections.

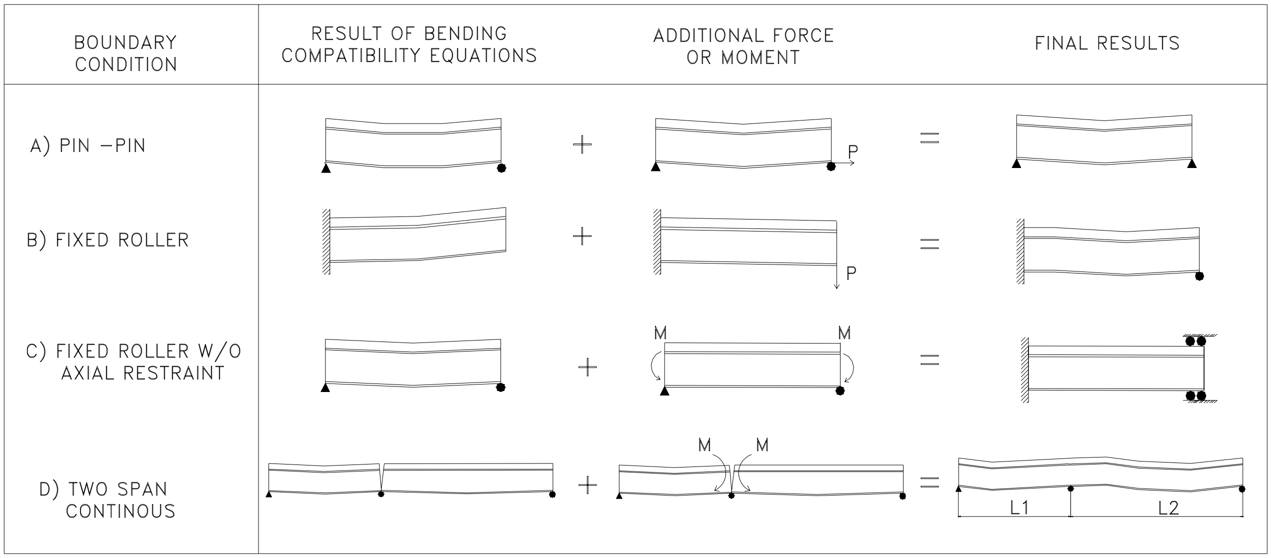


Figure 83. Superposition principle applied for finding deck Stresses in different boundary conditions.

Table 30. Comparison of FEA and compatibility equations results for the different boundary conditions.

Boundary Condition	Deck Bottom Stress			Deck Top Stresses		
	Equations KPa (PSI)	FEA KPa (PSI)	Error (%)	Equations KPa (PSI)	FEA KPa (PSI)	Error (%)
Pin – Pin	429.06 (62.23)	429.00 (62.22)	0	203.60 (29.53)	201.05 (29.16)	1.2
Fixed – Roller	915.07 (132.72)	874.6 (126.85)	4.6	(152.10)	(150.71)	0.9
Fixed – Fixed w/o Axial Restraint	754.70 (109.46)	743.19 (107.79)	1.5	754.56 (109.44)	743.19 (107.79)	1.5
Fixed – Fixed	1449.97 (210.30)	1449.97 (210.30)	0	1449.97 (210.30)	1449.97 (210.30)	0
Two Span Continuous	914.93 (132.7)	867.43 (125.81)	5.4	1048.69 (152.1)	1041.18 (151.01)	0.69
Three Span Continuous	818.96 (118.78)	799.79 (116.00)	2.4	872.32 (126.52)	864.53 (125.39)	0.9

Pin-Pin Boundary Condition

Figure 83-A shows this case and illustrates the application of superposition principle. The value of the redundant force P, required to satisfy compatibility of deformation is:

$$P = -[\frac{(F_2 d_{tg} - Q_3) d_{bg}}{E_{Girder} I_{Girder}} - \frac{F_2}{A_{Girder} E_{Girder}}][\frac{1}{A_{composite} E_{Composite}} + \frac{d_{bc}{}^2}{E_{Composite} I_{Composite}}]^{-1} \qquad (21)$$

In this equation A_{Girder}, I_{Girder}, and E_{Girder} are respectively area, moment of inertia and modulus of elasticity of girder, $A_{Composite}, I_{Composite}$, and $E_{Composite}$ are respectively area, moment of inertia and modulus of elasticity of composite section, d_{bg} and d_{tg} are respectively distance from centroid of the girder to top and bottom fiber of the girder, and d_{bc} and d_{tc} are respectively distance from centroid of composite section to top and bottom of section. This force, P, produces additional stresses at top and bottom of the deck ($\Delta\sigma_{DeckTop}, \Delta\sigma_{DeckBottom}$):

$$\Delta\sigma_{DeckTop} = P[\frac{1}{A_{composite}} - \frac{d_{tc} d_{bc}}{I_{Composite}}] \qquad (22)$$

$$\Delta\sigma_{DeckBottom} = P[\frac{1}{A_{composite}} - \frac{(d_{tc} - t) d_{bc}}{I_{Composite}}] \qquad (23)$$

Where t is the deck thickness. Thus, the deck's final stress is obtained from addition of these stresses to the results of equations 9 and 10.

<u>Fixed-Roller Boundary Condition</u>

Figure 83-B shows this case and illustrates the application of the principal of superposition. Force P required to produce zero displacement in horizontal direction at the support is:

$$P = \frac{(F_2 d_{tg} - Q_3)}{2 E_{Girder} I_{Girder}} \times \frac{3 E_{Composite} I_{Composite}}{L} \qquad (24)$$

Where L is the span length. This force, P, produces additional stresses in the deck, where:

$$\Delta\sigma_{DeckTop} = \frac{P L d_{tc}}{I_{Composite}} \qquad (25)$$

$$\Delta\sigma_{DeckBottom} = \frac{P L (d_{tc} - t)}{I_{Composite}} \qquad (26)$$

Thus, the deck's final stresses are obtained from addition of these stresses to the results of equations 9 and 10.

Fixed-Fixed Boundary Condition With Axial Force Release

Figure 83-C shows this case and illustrates the application of superposition principle. Although cantilever system could be chosen as the base case but due to symmetry, it is computationally easier to chose simply supported case as base case. The value of the redundant moment M required to satisfy compatibility conditions is:

$$M = \frac{(F_2 d_{tg1} - Q_3)E_{Composite} I_{Composite}}{E_{Girder} I_{Girder}} \tag{27}$$

Moment M produces additional stress in the deck, where:

$$\Delta\sigma_{DeckTop} = \frac{Md_{tc}}{I_{Composite}} \tag{28}$$

$$\Delta\sigma_{DeckBottom} = \frac{M(d_{tc} - t)}{I_{Composite}} \tag{29}$$

Deck's final stresses are obtained from addition of these stresses to the results of equations 9 and 10.

Fixed-Fixed Boundary Condition

For the case of fixed-fixed boundary condition the deck stresses are simply computed from the following equation:

$$\sigma_{Dec\,kTop(Bottom)} = \alpha_{Deck} \Delta T_{Top(Bottom)} E_{Deck} \tag{30}$$

Where ΔT is the equivalent temperature change in the deck at top and bottom of deck.

Two Span Continuous System

Figure 83-D shows the application of the superposition method to the continuous beam case. It is assumed here that there is no change in cross section in the two spans. However, the method is general and the reader can apply this concept with necessary modifications when sectional properties are different in each span.

Continuity moment M is calculated by:

$$M = \frac{(F_2 d_{tg} - Q_3)(3E_{Composite} I_{Composite})}{2E_{Girder} I_{Girder}} \tag{31}$$

Moment M produces additional stresses in the deck, which can be calculated using equations 19 and 20.

Three Span Continuous System

It is assumed here again that there is no change in cross section in the three spans. Continuity moments at 2 spans can be computed by solving the following equations, which comes from compatibility:

$$\frac{(F_2 d_{tg} - Q_3)(L_1 + L_2)}{2E_{Girder} I_{Girder}} = \frac{M_1(L_1 + L_2)}{3E_{Composite} I_{Composite}} + \frac{M_2 L_2}{6E_{Composite} I_{Composite}}$$
$$\frac{(F_2 d_{tg} - Q_3)(L_2 + L_3)}{2E_{Girder} I_{Girder}} = \frac{M_2(L_2 + L_3)}{3E_{Composite} I_{Composite}} + \frac{M_1 L_2}{6E_{Composite} I_{Composite}} \quad (32)$$

In these equations L_1, L_2, and L_3 are span lengths and M_1 and M_2 are continuity moment between spans. This gives:

$$M_1 = 3\frac{(F_2 d_{tg} - Q_3)(L_2 + L_3)(2L_1 + L_2)E_{Composite} I_{Composite}}{E_{Girder} I_{Girder}(4L_1L_2 + 4L_1L_3 + 3L_2{}^2 + 4L_2L_3)}$$
$$M_2 = 3\frac{(F_2 d_{tg} - Q_3)(L_2 + L_1)(2L_3 + L_2)E_{Composite} I_{Composite}}{E_{Girder} I_{Girder}(4L_1L_2 + 4L_1L_3 + 3L_2{}^2 + 4L_2L_3)} \quad (33)$$

Based on the bridge data from Table 27 and for $L_1 = L_2 = L_3 = 1374cm\ (541in)$, and an arbitrary 10^{o}F uniform temperature decrease in the deck the results of compatibility equations and FE model are compared (Table 30).

Results show that deck stresses do not depend on length of span. Figure 84 and 85 shows comparison of the deck top and bottom stresses for different boundary conditions.

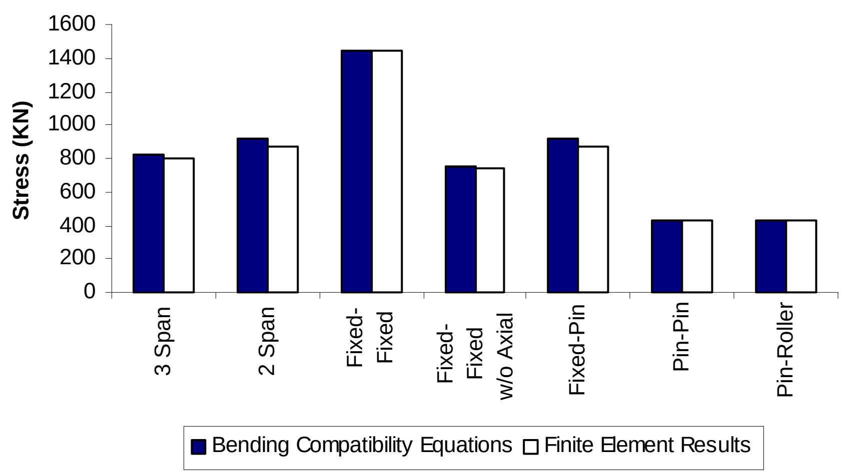


Figure 84. Comparison of deck bottom stresses for different boundary conditions.

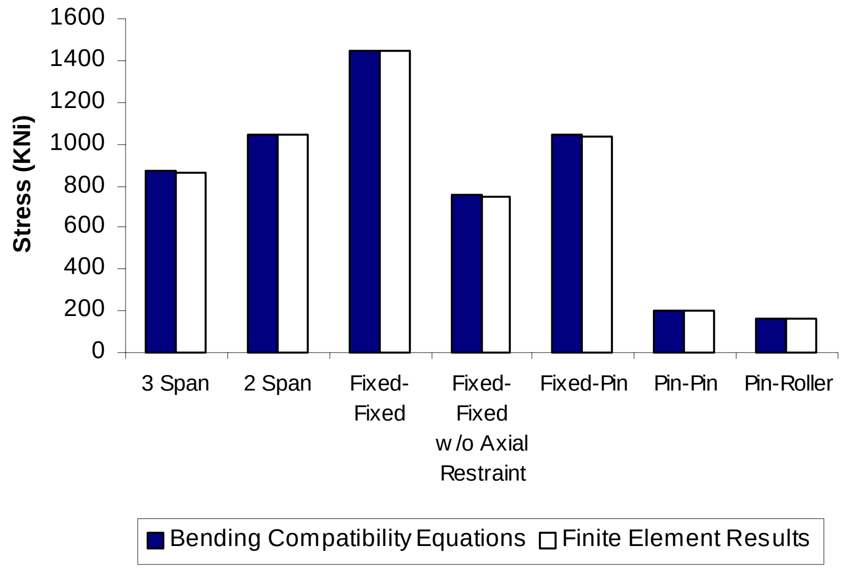


Figure 85. Comparison of deck bottom stresses for different boundary conditions.

Loading definition for simplified procedure

To investigate transverse deck cracking using developed equations appropriate loading values and distributions should be chosen. These temperature distributions represent effect of different thermal factors, which causes deck cracking. There are four major different causes producing stress and strain in the system. These four causes are autogenous shrinkage, drying shrinkage, hydration temperature rise, and temperature gradient. Creep strains are not included among the causes. Excluding stress relief due to creep strains results in higher stresses in the system. Due to early occurrence of these cracks, creep strains seems to not contribute significantly and it is justifiable to ignore creep strains. However, if long-term performance of bridge deck is being investigated, creep effect should also be included.

Temperature profile and values in the section are the key parameters in estimating deck stresses. For modeling purposes and uniformity in the equations, shrinkage strains will be converted to equivalent temperature change (by dividing the strain by the coefficient of thermal expansion) and it will be added to other thermal loadings. Procedures based on design guidelines and literature to obtain reasonable values and profiles for each of the four thermal loadings are presented.

<u>Drying and Autogenous Shrinkage</u>

Drying shrinkage is the result of water loss of hardened concrete. Volume of concrete reduces as water withdraws from concrete. However, only part of (40-70 percent) the shrinkage is recoverable with future wetting cycles. Both AASHTO (LRFD, 1986) and ACI (Committee 207, 1986) recommend different equations for shrinkage computation. Either of these equations can be used in evaluating shrinkage strains, although AASHTO (LRFD, 1986) equation results in lower shrinkage strains. In absence of any other data, engineering judgment should determine the use of one of these two equations.

ACI (Committee 207, 1986) recommends the following equation for predicting drying shrinkage strain of moist cured concrete:

$$\varepsilon_{sh,t} = K_{vs} K_h \frac{\bar{t}}{\bar{t} + 35} 0.78 \times 10^{-3} \tag{34}$$

Whereas AASHTO recommends:

$$\varepsilon_{sh,t} = K_{vs} K_h \frac{\bar{t}}{\bar{t} + 35} 0.51 \times 10^{-3} \tag{35}$$

In this formula $\varepsilon_{sh,t}$ is the shrinkage at time t, K_{vs}, and K_h are volume to surface coefficient and humidity coefficient and t is the time after curing. Both codes provide guidelines for choosing K values.

Due to small thickness of the bridge deck, shrinkage strains across the section are considered uniform. This means that a temperature decrease equal to $\varepsilon_{sh,t} / \alpha_{Deck}$ should be applied to deck concrete to model shrinkage.

Autogenous shrinkage is the concrete shrinkage without loss of water. This kind of shrinkage occurs at low w/c ratios and significantly increases with use of silica fume, HRWRAs (High Range Water Reducing Admixtures) and finer cement. In the past, this type of shrinkage was insignificant. However with the downward trend of w/c ratio in concrete mixes, use of silica fume, use of finer cements, and widespread use of HRWRAs this type of shrinkage has come into attention. If there is any indication that this type of shrinkage may be involved, magnitude of this shrinkage should be considered in similar manner.

Temperature Gradients

Daily temperature variations and solar radiation produces temperature differential throughout the section. Stresses produced by this temperature differential add up to the locked in stresses from different types of shrinkage. AASHTO (LRFD, 1986) provides guidelines for estimating positive and negative temperature gradients through the depth. The temperature gradient profile suggested by AASHTO (LRFD, 1986) for concrete girder is shown in Figure 86. Where C shall be taken as:

- 30-cm (12.0-inch) for concrete superstructures that are 16.0” or more in depth
- For concrete sections shallower than 40-cm (16.0-inch), C shall be taken 10-cm (4.0-inch) less than the actual depth
- 30-cm (12.0-inch) for steel superstructures

Based on the location of the bridge different values are suggested by AASHTO to define the temperature profile (Table 31).

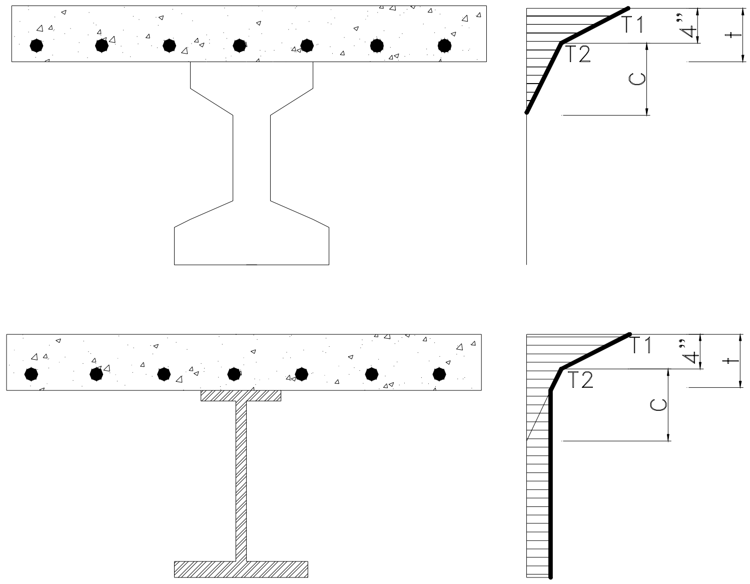


Figure 86. AASHTO temperature gradient profile for steel and concrete girders.

Table 31. Basis for temperature gradients (AASHTO LRFD 1998).

Zone	T1 °C (°F)	T1 °C (°F)
1	30 (54)	7.8 (14)
2	25.6 (46)	6.7 (12)
3	22.8 (41)	6.1 (11)
4	21.1 (38)	5 (9)

Temperature Rise Due to Hydration

Concrete temperature rises after placement due to hydration. If the concrete is restraint, part of temperature rise, subsequent cooling, and increase in modulus of elasticity produces tensile stresses in concrete and may cause cracking few days after placement. It is also possible that these locked in stresses add up to stresses produced by other causes. To evaluate hydration temperature effects, the maximum temperature rise during hydration should be estimated. After computing the maximum temperature rise the structure should be analyzed under a temperature drop equal to peak temperature minus setting temperature (worse case scenario). Setting temperature of bridge is defined as actual air temperature averaged over 24-hour period immediately preceding setting (AASHTO LRFD, 1986).

ACI report 207 (1986),which mainly discusses mass concrete, provides a method for estimating the maximum temperature rise during hydration. ACI mentions, “The report can be applied to normal structural concrete; however its application is not usually warranted”. In the absence of any other data, it is suggested that ACI procedure can be applied to estimate the maximum temperature rise

Windows Based Application

A user friendly MS Windows based application is prepared to perform the analysis based on the input values from user. Figure 87 shows typical input and output windows of the program. The program can be downloaded from http://web.njit.edu/~ala.

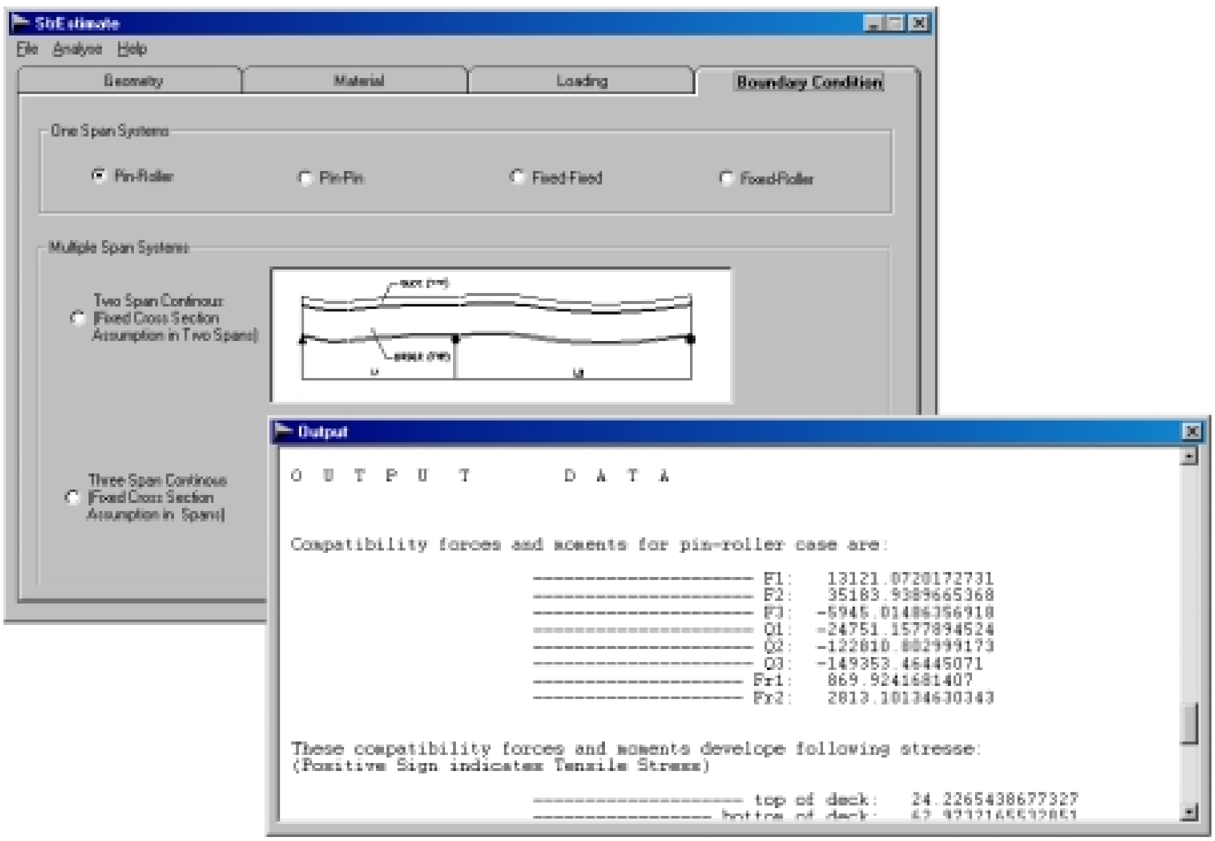


Figure 87. Typical input and output window of application.

CONCLUSIONS AND RECOMMENDATIONS

Many concrete bridge decks develop transverse cracking and most of these cracks develop at early ages, many right after construction. These cracks are typically full-depth and spaced 1-3 m (3-10 ft) apart along the length of the bridge. Transverse cracks can accelerate corrosion of reinforcing steel, deteriorate deck concrete, possibly cause damage to underneath components of the bridge, and damage bridge esthetic. As a result of these adverse effects of transverse cracking, the maintenance costs will increase and ultimately the service life of the bridge system will be shortened. There have been many studies on the cause of transverse deck cracking over the past several decades. However, the causes of transverse deck cracking are not yet fully understood and the problem still exists. Previous studies were mostly focused on concrete mix design and improvement through changes to construction practice to alleviate shrinkage problem. In many instances there are major disagreements on the factors affecting transverse cracking indicating the need for further research. As a part of this research study 24 bridges were surveyed in the state of New Jersey. Material and mix design values for these bridges were consistent with the recommendations reported in the literature. Despite this fact, majority of these bridges surveyed have developed cracks highlighting the importance of design factors, which was the main thrust of this study.

Cracks in concrete occur when a *restraint* mass of concrete tends to *change volume*. Volume change in concrete depends mix design and construction procedures. Restraint, which is basically due to composite action of deck and girder, depends on design characteristics of the bridge such as continuity, relative deck to girder stiffness, span length, girder spacing, amount of deck reinforcement, etc. To study these factors several 2-D and 3-D linear and nonlinear finite element models were employed. Results along with specific conclusions are presented in details in Chapter 5 and will not be repeated here. However, based on these analytical work, survey of bridges, and literature review the following conclusions can be highlighted. These are presented in the form of specific recommendations for possible implementation, and are grouped under the three categories of structural design factors, mix design, and constructions practices.

Structural Design Factors: Recommendations

Specify an Upper Limit on Concrete Strength

Survey results indicate that the actual strength is much higher than the specified design strength. Therefore, it is recommended that an upper limit on concrete strength be required. This should not be viewed upon as discouragement to the use of HSC but rather a requirement on design versus actual concrete strength. Also, when “open-early” is not an issue, design should use low-early strength concrete.

Boundary Restraints Should be Consistent with Design

Construction practice should not introduce undue boundary restraint on the girders. For example, for a span designed as simply supported, which is desirable under shrinkage load, the girders should not be embedded in the end diaphragms or they should be debonded.

Time-Dependent Loadings must be considered in Design of Bridges with Integral Abutments

Integral abutments have many structural and maintenance benefits and are becoming more popular around the nation including in the state of New Jersey. However, bridges with integral abutments have a much higher tendency for transverse deck cracking. Therefore, time and temperature dependent loadings must be explicitly considered in their design. A simple and effective design tool to facilitate this aspect of design has been developed under this study (see chapter six).

Minimize the Ratio of Girder/Deck Stiffness

Try to minimize the ratio of girder to deck stiffness through changes in deck thickness, girder spacing, and girder moment of inertia. As have been shown, larger spacing, flexible girders and thicker decks are preferred. Try to provide the required moment of inertia with more contribution from the deck.

Employ more Flexible Superstructures

Analyses results indicate that more flexible superstructures have lower tendency for deck cracking. Therefore, the design should employ a more flexible superstructure. This objective can be pursued through two different venues. Under current practice, where there is a deflection limit, the design manual should prevent design of an overly stiff superstructure by putting a limit on the margin by which the deflection requirement is satisfied. Currently strength requirements usually control the design, and deflection is only checked and it is quite common for this requirement to be satisfied by a very large margin (e.g., deflection equal to 1/1500 of span length as oppose to required 1/1000 of span length). A second approach would be, consistent with AASHTO LRFD, to drop deflection requirement under service load or at least to increase the limit.

Use Uniform Reinforcement Meshes

Uniform reinforcement meshes on top and bottom are recommended to control cracking. Use of empirical design method is encouraged. Increasing the volume of reinforcement above code requirement does not have any effect on cracking and is discouraged.

Consider AASHTO Article 3.12

Current designs often do not explicitly consider time-dependent loadings as specified under AASHTO article 3.12. Such a consideration is recommended. A simple windows application tool (StrEstimate) has been developed under this study to facilitate comparison of different designs and estimate deck stresses during the design stage.

Mix Design: Recommendations

Adopt a Restraint Shrinkage Test

Concrete mix to be used in bridge deck should be tested for cracking using one of cracking tendency tests described in this report. Cements and mixes with poor results should not be used in bridge deck.

Reduce cement content to 650-660 lb/yd^3, and consider using fly ash.

Use AASHTO specification Type II cement for bridge deck construction.

Limit Water Cement Ratio

Limit water cement (w/c) ratio to 0.4-0.45. Make use of water reducers to reduce water content. Consider w/c<0.4 with the use of water reducers.

Aggregate Size and Shape

- Use largest possible grain size as specified in ACI –318.
- Use crushed stone for coarse aggregate.
- Maximize aggregate content.

Consider using type K Shrinkage Compensating Concrete when available

Construction Practices: Recommendations

Employ the following pouring sequence

- Pour complete deck at one time whenever feasible within the limitation of the maximum placement length based on drying shrinkage consideration.
- If multiple placements must be made and the bridge is composed of simple spans, then place each span in one placement.
- If bridge is simple span, but cannot be placed in a single placement, divide the deck longitudinally and make two placements.

- If the bridge is simple span and single placement cannot be made over full span length, then place the center of span segment first and make this placement as large as possible.
- If multiple placements must be made and the bridge is continuous span, then place concrete in the center of positive moment region first and observe a 72-h delay between placements.
- When deck construction joints are created, require priming existing interfaced surfaces with a Primer/Bonding agent prior to placement of new concrete.

Wind, Weather and Concrete Temperature (Already included in the NJDOT Specs)

Follow procedures in sections 501.12 and 501.17 of NJDOT Specs. Make use of evaporation rate chart proposed by ACI. Cast the deck in mild temperatures. It is also recommended that wind and humidity levels be recorded on the Inspection/Testing datasheets.

Protection and curing (Already included in NJDOT Specs)

Follow procedures in section 501.17. Start curing immediately after finishing and cure at least 7 consecutive calendar days. If “early-open” is not an issue consider 14-day wet curing.

Use temporary shoring for simply supported girders when practical

Future Research Needs

In light of the results of this and prior studies and the recommendations made under this study the following areas for possible future research work are identified.

Restraint Shrinkage Tests

There are two distinct shrinkage tests (bar test and ring test) that can be used in determining shrinkage potential of deck concrete. A study should be conducted to determine the suitability of one of these two tests. More importantly, although several highway agencies are considering the use of these tests, there is still a great need for additional research work to quantify the results of these tests.

Impact of Flexible Superstructure

One recommendation of this study is to employ a more flexible superstructure. It is prudent to conduct additional research study on the implication of this recommendation on serviceability requirements. Results of such a study will also be beneficial in determining the impact of AASHTO-LRFD provision (or lack of) on deflection.

Study on Construction Practices

As it was mentioned before, many of the recommendations on mix design and construction practice already exist in the most recent version of NJDOT Standard Specifications for Road and Bridge Construction (NJDOT Specs.). A study to monitor and compare performance of bridges built based on these specifications will be valuable to our understanding of the causes of deck cracking and will help in identifying more specifically the important construction and mix design factors.

Controlled Composite Action

As discussed before, there is an agreement among researchers that shrinkage (thermal and drying) and a temperature gradient of *restraint deck* are the main causes of cracking. The restraining effect is basically due to the composite design of the superstructure. Composite construction reduces the size of girders, which means less cost. While an economical design requires composite behavior under ultimate loads, under initial portion of service loads (such as dead load) the girders alone are generally capable of resisting the entire load. Thus, shrinkage cracking can be prevented if a mechanism can be developed to prevent composite action during early ages (i.e., as concrete shrinks), while it is activated for higher service load and under ultimate loading condition. To achieve this objective (i.e., no composite action initially and full composite action ultimately) the shear connectors can be wrapped in hyperelastic material of carefully designed thickness, as shown in Figure 88.

Design of these materials, i.e., their thickness, modulus of elasticity, etc. should be chosen very carefully to achieve the required controlled composite action (CCA). A typical stress strain curve for hyperelastic materials is shown in Figure 89. Under low level of stresses the material does not provide any resistance and it deforms easily. Thus, when the shear connectors are wrapped by such a material the concrete deck can shrink without any restraint. There might be a need for bond breaker between the deck and girder to further reduce bonding. Upon development of shrinkage strains the hyperelastic material will start to develop higher level of resistance and will ultimately provide full composite action. Proper design will require development of a realistic relationship between the shrinkage strain and the wrap thickness. A feasibility study to investigate the viability of such a design is highly recommended since it has the potential to entirely eliminate the problem of transverse cracking in concrete bridge decks.

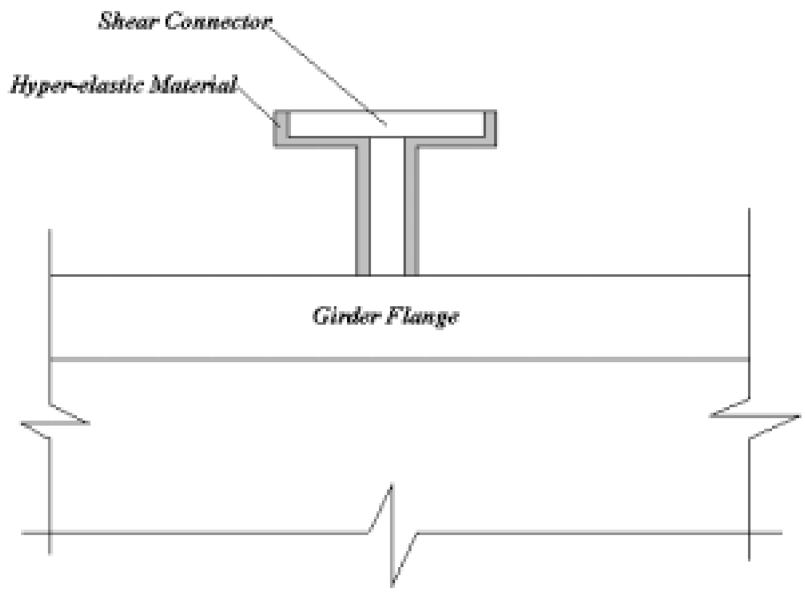


Figure 88. Details of controlled composite action connector.

Figure 89. Stress-strain curve for typical hyperelastic materials

REFRENCES

1. American Association of State Highway and Transportation Officials. *AASHTO LRFD Bridge Design Specification, 2nd Edition.* Washington D.C., 1998
2. ACI committee 209. *Prediction of Creep, Shrinkage, and Temperature Effects.* Detroit, 1978
3. ACI Committee 207. *Shrinkage and Creep effects in Concrete.* Detroit, 1986
4. ACI committee 308. *Standard Practice for Curing Concrete.* Detroit, 1986
5. American Concrete Institute. *ACI Manual of Concrete Practice- Part 1:Materials and general Properties of Concrete*, Detroit, 1986
6. ANSYS. Computer Software, ANSYS Corporation, 1994-1998
7. K. Babaei and R.L. Purvis. “Premature Cracking of Concrete Bridge Decks: Cause and Methos of prevention” *Proceedings, 4th International Bridge Engineering Conference*, 1996
8. R.L. Blaine, H.T. Arni, D.H. Evans, M.R. Defore, J.R. Clifton and R.G. Methey. “Interrelation between Cement and Concrete Properties” *Building Research Division of National Bureau of Standards*, 1953
9. R. Breitenbucher and M. Mangold. “Minimization of Thermal Cracking in Concrete Members at Early Ages” *Thermal Cracking in Concrete at Early Ages*, E&FN Spon, London, pp.205-212, 1994
10. Burrows R, *Visible and Invisible Cracks.* ACI Monograph No. 11, 1999
11. R.W. Carlson. “Cracking of Concrete” *presented at Joint meeting of the Boston Society of Civil Engineers and The Designers’ Section*, BSCE, Jan. 14, 1942
12. T.T.H. Cheng and D.W. Johnston. *Incidence Assessment of Transverse Cracking in Concrete Bridge Decks: Construction and Materials Considerations*. Rep. No. FHWA/NC/85002, 1, Dep. Of Civ. Engrg., North Carolina State University, Raleigh, N.C., 1985
13. J.J. Chui and W.G. Dilger. “Temperature Stress and Cracking Due to Hydration Heat”, *Creep and Shrinkage of Concrete*, E&FN Spon, London, pp.271-276, 1993
14. F.H. Dakhil, P.D. Cady and R.E. Carrier. “Cracking of Fresh Concrete as Related to Reinforcement” *ACI Struct. Journal,* 72(8), pp. 421-428, 1975
15. J. Ducret and J. Lebet. “Measurements on a Composite Bridge: Effect of Concrete Hydration” *Proceedings of an Engineering Foundation Conference*, American Society of Civil Engineers, New York, 1996
16. J. Emanuel and J. L. Hulsey. “ Temperature distributions in composite bridges” *Journal of Structural Division*, ASCE, Jan 1978, pp 65-78
17. C. French, L. Eppers, Q. Le, and J.F. Hajjar. “Transverse cracking in concrete bridge decks” *Transportation Research Record*. n 1688 Nov 1999. p 21-29
18. R.J. Frosch, R.D. Radabaugh and D.T. Blackman. “Investigation of Transverse Deck Cracking“ *ASCE Structures Congress*, Denver, April 2002
19. G.T. Halverson. “Troubleshooting concrete cracking during construction,” *Aberdeen’s Concrete Constr.*, 36(11), 811-816, 1991

20. J. Houde. *Study of force-displacement relationship for the finite element analysis of reinforced concrete.* Report No. 72-3, Dept. of Civil Engineering and applied mechanics, McGill University, Montreal, Dec. 1973.
21. M.A. Issa. "Investigation of cracking in concrete bridge decks at early ages" *Journal of Bridge Engineering,* V 4 n 2 1999, p 116-124
22. A.A. Khan, W.D. Cook, and D. Mitchell. "Thermal properties and transient thermal analysis of structural members during hydration" *ACI Materials Journal,* V95 n 3 May-Jun 1998, p 293-303
23. T. Kochanski, J. Parry, D. Pruess, L. Schuchardt, and J. Ziehr, J. *Premature Cracking of Bridge Decks Study*. Wisconsin Dept. of Transportation, Madison, Wisconsin, 1990
24. P.D. Krauss, and E.A. Rogalla. *Transverse Cracking in Newly Constructed Bridge Decks.* NCHRP Report 380, Transportation Research Board, Washington, DC, 1996
25. Microsoft Access 2000, Computer Software, Microsoft Corporation, 1992-1999
26. Microsoft Excel 2000, Computer Software, Microsoft Corporation, 1985-1999
27. C. Meyers. *Survey of Cracking on Underside of Classes B-1 and B-2 Concrete Bridge Decks in District 4.* Investigation 82-2, Division of Material and Research, Missouri Highway and Transportation Department, Jefferson City, Mo., 1982
28. New Jersey Department of Transportation. *1998 Standard Specification for Road and Bridge Construction and 1998 Supplemental Specification.* Trenton, NJ, 1998
29. Portland Cement Association. *Final Report - Durability of Bridge Decks – A cooperative Study.* Skokie, Illinois, 1970
30. H.B. Prenger. *Bridge deck crackin, Research report.* Rep. No. MD-93-04, Maryland Dept. Of Transp., State Highway Administration, Baltimore, Md.,19., 1992
31. M. Paillere, M. Buil and J.J. Serrano. "Effect of Fiber Addition on the Autogenous Shrinkage of Silica Fume Concrete" *ACI Material Journal*, V.86, No.2, Mar.-Apr., pp. 139-144, 1989
32. G.E. Ramey, A.R. Wolff, R.L. Wright. "Structural design actions to mitigate bridge deck cracking" *Practice Periodical on Structural Design & Construction*, v 2 n 3 p 118-124, 1997
33. T.R. Schmitt, D. Darwin. "Effect of material properties on cracking in bridge decks" *Journal of Bridge Engineering,* v 4 n 1, p 8-13, 1999
34. E. Tazawa, Y. Matsouka, S. Miyazawa, and S. Okamoto. "Effect of Autogenous Shrinkage on Self Stress in Hardening Concrete" *Thermal Cracking in Concrete at Early Ages*, E&FN Spon, London, pp.222-228, 1994
35. E. Tazawa and S. Miyazawa. "Experimental Study on Mechanism of Autogenous Shrinkage of Concrete" *Cement and Concrete Research*, V.25, No.8, pp. 1633-1638, 1995

36. R. Sprinegnschmid and R. Breitenbucher “Cement with low crack Susceptibility”, *Proceedings of Advances in Cementitious Materials Conference*, American Ceramic Society, pp. 701-713, 1990
37. R. Sprinegnschmid, R. Breitenbucher and M. Mangold. “Thermal Cracking in Concrete at Early Ages”, E7FN Spon, London, pp. 137-134, 1994
38. G.J. Verbeck and G.J. Foster. “Long-term study of cement performance in concrete” *ASTM Proceedings*, Vol 50, pp 1235-1257, 1950
39. Winter and Nilson. *Design of concrete structures.* Mc-Graw Hill, 1986
40. L.C.P. Yam, and J.C. Chapman. “The inelastic behaviour of simply supported composite beam of steel and concrete”, *Proc. Istin Civ. Engrs*, 1968, 41, Dec., 651-683.
41. W. Zuk. “Thermal and Shrinkage Stresses in Composite Beams” *Journal of ACI*, Vol 58,p327-339, 1961

APPENDIX A: BRIDGE INFORMATION AND SURVEY RESULTS

This appendix contains important structural design, material properties, concrete mix design and some construction information, supplemented by the results of field surveys of several bridges. Data presented here were collected from different sources and were used in various parts of the study (see chapter 3).

Following this introduction, section A.2 describes major elements of the data for these bridges and how they are grouped. Definitions of some data entry are also provided when necessary. The final section (Section A.3) presents the data for bridges surveyed.

Organization

The data presented for each bridge is grouped into six parts. Part one contains general information about the bridge. This information includes geographical location of the bridge (i.e. county, township) and construction year, which are obtained from the original list provided by NJDOT and subsequently entered into the survey forms.

Part two summarizes important design characteristics of the bridge. These data are mostly derived from the structural plans of the bridge. *Note that spans are numbered in the direction of traffic when it is only in one direction on the bridge, otherwise span number is explicitly identified.* This part contains the following structural design related data for each span:

- Number of spans
- Traffic direction
- Girder type
- Span type (i.e., continuous or simply supported at interior spans)
- Span length and width
- Framing information (i.e., spacing of girders in each span)
- Deck design information (i.e., rebar details, thickness, cover depth, and wearing surface)
- Girder properties (i.e., area, depth, and moment of inertial)
- Shear stud spacing

In some cases where the data item is not constant through the range considered, like span length in curved bridges or the moment of inertia when different girders are used in one span, the range of data for that item is reported.

Part three presents material properties and mix design information based on NJDOT inspection/testing datasheets. This information includes cement type, cement content, water content, air content, w/c ratio, compressive strength, slump and admixtures. As explained in chapter 3, these data represents an average value of the data item considered for the bridge.

In part four construction related data, such as air temperature, concrete temperature at the time of placement, and the month of casting are reported. These data are also obtained from the NJDOT inspection/testing datasheets. Part five summarizes the survey results, and finally part 6 presents observations and photos during the survey, if any.

Bridge Information and Survey Results

Bridges are ordered based on the first four digits of NJDOT structure numbers, from the lowest to the highest.

Structure Number 0206-165

1.General Information

Bridge Location	Hackensack Avenue over NJ RT4		
County	Bergen County	**City**	Hackensack
Year Built	1998		

2.Structural Design Information

Number of spans: 2
Direction of Traffic: Two ways
Girder type: Steel Girder
Span type: Simply Supported
Type Of Bearing: Steel Bearings
Skewness: None

	North Bound		South Bound	
Span Number	**1**	**2**	**1**	**2**
Span Width	47'-0"	47'-0"	47'-0"	47'-0"
Span Length	90'-2"	90'-2"	90'-2"	90'-2"

Framing Information:
-North Bound (All Spans): 5 Girders@7'-6" + 2 Girders@6'-3"
-South Bound (All Spans): 7 Girders@7'-1"

Deck Slab Information:

Deck Thickness	$8^{1/2}$"	**Top Cover**	$2^{1/2}$"
Wearing Surface	Concrete, Sawcut	**Bottom Cover**	1"

Longitudinal Rebar Information:
-North & South Bound (All Spans): #5@6"(B), #5@15"(T)
Transverse Rebar Information:
-North & South Bound (All Spans): #6@7"(T&B)

Girder Information:

	North & South Bound		
Span Number	**I (in^4)**	**A (in^2)**	**Depth (in)**
1&2	26876.39	15.58	38.5

Shear Stud Information:

	North & South Bound	
Span Number	**Spacing (in)**	**Number of Studs in Row**
1&2(Midspan)	12	4
1&2(Support)	9	4

3.Material Properties & Mix Design Information

Cement Type: Blue Circle Atlantic Type II
Cement Content: 658 lb/yd^3
Water Content: 35 Gal/yd^3
W/C Ratio: 0.44
Air Content: 6.6%
Compressive Strength: 5174 Psi
Slump: 3.5"
Admixtures: Water Reducer

4.Construction Information

Air Temperature: 73 F
Concrete Temperature: 83 F
Month Of Casting: August-September

5.Survey results

Crack Information:

	North Bound		South Bound	
Span No.	**1**	**2**	**1**	**2**
Type	Transverse/Other	Transverse/Other	Transverse/Other	Transverse/Other
Location	Center/End	Center/End	Center/End	Center/End
Spacing	0'-2'	0'-2'	0'-2'	0'-2'
Size	Typical	Typical	Typical	Typical

Survey Team Member: Ala

6. Observations & Photos

-None

Structure Number 0713-151

1.General Information

Bridge Location	RT21-I-78, RAMP FROM 21S TO I-78 WEST		
County	Essex	**Township**	Newark
Year Built	2000		

2.Structural Design Information

Number of Spans: 8
Direction of Traffic: One way
Girder type: Steel Girder
Span type: Mixed (3 Continuous + 1 Simply supported+ 4 Continuous)
Type Of Bearing: Steel Bearings
Skewness: Mild

		Ramp	
Span Number	**1**	**2**	**3**
Span Width	29'-6"~32'-4"	29'-6"	29'-6"~ 42'-5"
Span Length	165'-6"	158'-8"	143'-10"
Span Number	**4**	**5**	**6**
Span Width	42'-9"~82'-1"	38'-6"	38'-6"
Span Length	142'-7"	131'-6"	165'-0"
Span Number	**7**	**8**	
Span Width	38'-6"	36'-`0"~38'-6"	
Span Length	165'-0"	131'-2"	

Framing Information:
-Ramp:
-Span 1: 4 Girders @8'-2"
-Span 2: 4 Girders @8'-2"
-Span 3: 4 Girders @8'-2"~12'-6"
-Span 4: 3 Girders @8'-2"~12'-9"+4 Girders@ 6'-4"~12'-10"
-Span 5: 4 Girders @11'-2"
-Span 6: 4 Girders @11'-2"
-Span 7: 4 Girders @11'-2"
-Span 8: 4 Girders @11'-2"

Deck Slab Information:

Deck Thickness	$9^{1/2"}$"	**Top Cover**	$2^{1/2}$"
Wearing Surface	Concrete, Turf Drug	**Bottom Cover**	1"

Longitudinal Rebar Information:
-All Spans Midspan: #5@8"(B), #5@15"(T)
-Continuous Spans Over Pier: #5@8"(B), #5@$7^{1/2}$"(T)
Transverse Rebar Information:
-All Spans EB: #6@6"(T&B)

Girder Information:

		Ramp	
Span Number	**I (in^4)**	**A (in^2)**	**Depth (in)**
1 (Midspan)	76151~105242	81.5~100.75	78~79
1 (support)	66278~165253	73.75~136.25	78~81

2 (Midspan)	61947~64896	70.75~72.75	78
2 (support)	110331~156253	102.25~136.25	79.5~81.5
3 (Midspan)	73450~77368	79.25~81	78
3 (support)	73450~165253	79.25~136.25	78~81.5
4 (Midspan)	72620~111092	81~109	78
4 (support)	62992~94680	74~96	78
5 (Midspan)	67623~99855	74.75~101.75	78~78.75
5 (support)	64896~200418	72.75~159.75	78~81.75
6 (Midspan)	67623~94739	74.75~99.63	78~78.5
6 (support)	136365~200418	119.75~159.75	80.5~81.75
7 (Midspan)	67623~94739	74.75~99.63	78~78.5
7 (support)	136365~183783	119.75~149.5	80.5~81.25
8 (Midspan)	67623~99855	74.75~101.75	78~78.75
8 (support)	67623~183783	74.75~149.5	78~81.25

Shear Stud Information:

	Ramp	
Span Number	**Spacing (in)**	**Number of Studs in Row**
1 (Midspan)	19~22	3
1 (support)	16~18	3
2 (Midspan)	24	3
2 (support)	10	3
3 (Midspan)	24	3
3 (support)	17~19	3
4 (Midspan)	12~22	3
4 (support)	15~18	3
5 (MIDSPAN)	15	3
5 (support)	12~24	3
6 (Midspan)	15	3
6 (support)	24	3
7 (Midspan)	15	3
7 (support)	24	3
8 (Midspan)	15	3
8 (support)	24	3

3.Material Properties & Mix Design Information

Cement Type: Essroc Type II
Cement Content: 658 lb/yd^3
Water Content: 32.6 Gal/yd^3
W/C Ratio: 0.41
Air Content: 5.5 %
Compressive Strength: 5694 Psi
Slump: 3.3”
Admixtures: Water Reducer

4.Construction Information

Air Temperature: 67 F
Concrete Temperature: 77 F
Month Of Casting: July-October

5.Survey results

Crack Information:

	Ramp
Span No.	**All Spans**

Type	Transverse
Location	All over
Spacing	>5’
Size	Hair Line

Survey Team Member: Ala, Allyn, Ross and Raj

6. Observations & Photos

-Not open to traffic at the time of survey but used by construction vehicle
-Hairline Cracks Observed.

Figure 90. Crack on the deck (Bridge No 0713-151).

Figure 91. Construction vehicle traffic on the bridge (Bridge No. 0713-151).

Figure 92. Crack on the deck (Bridge No. 0713-151).

Figure 93. Crack extending into the parapet (Bridge No. 0713-151).

Figure 94. Crack on deck and parapet (Bridge No. 0173-151).

Structure Number 1013-151

1.General Information

Bridge Location	RT31 Over South Branch Raritan River		
County	Hunterdon County	**Township**	Clinton
Year Built	1996		

2.Structural Design Information

Number of spans: 2
Direction of Traffic: Two ways
Girder type: Prestressed Concrete Girder
Span type: Simply Supported
Type Of Bearing: Steel Bearings
Skewness: Mild

	North Bound		South Bound	
Span Number	**1**	**2**	**1**	**2**
Span Width	49'-0"	49'-0"	52'-9"	52'-9"
Span Length	124'-10"~125'-1"	124'-11"~124'-10"	124'-10"~124'-9"	124'-10"~124'-7"

Framing Information:
-North Bound:
-Span1: 6Girders@7'-11" + 1 Girder@Varies (7'-10"~6'-9")
-Span2: 6Girders@7'-4" + 1 Girder@Varies (6"-9"~ 6'-1")
-South Bound:
-Span1: 6Girders@8'-0" + 1 Girder@Varies (6'-6"~7'-2")
-Span2: 6Girders@7'-10" + 1 Girder@Varies (7"-2"~ 8'-3")

Deck Slab Information:

Deck Thickness	$7^{3/4}+1^{1/4}$" Latex Concrete	**Top Cover**	$2^{1/4}$"
Wearing Surface	Latex Concrete, Sawcut	**Bottom Cover**	1"

Longitudinal Rebar Information:
-North Bound (All Spans): #5@9"(B), #5@15"(T)
-South Bound (All Spans): #5@6"(B), #5@15"(T)
Transverse Rebar Information:
-North Bound (All Spans): #6@8"(T&B)
-South Bound (All Spans): #5@6"(T&B)

Girder Information:

		North & South Bound	
Span Number	**I (in^4)**	**A (in^2)**	**Depth (in)**
1&2	761550.4	1125	72

Shear Stud Information:

	North & South Bound	
Span Number	**Spacing (in)**	**Number of Studs in Row**
1&2(Midspan)	12	2
1&2(Support)	6	2

3.Material Properties & Mix Design Information

Cement Type: Essroc Type II
Cement Content: 611 lb/yd^3
Water Content: 32.5 Gal/yd^3

W/C Ratio: 0.44
Air Content: 5.4%
Compressive Strength: 5241 Psi
Slump: 3.5”
Admixtures: Water Reducer

4.Construction Information

Air Temperature: 54 F
Concrete Temperature: 65 F
Month Of Casting: October

5.Survey results

Crack Information:

	North Bound		South Bound	
Span No.	**1**	**2**	**1**	**2**
Type	None	None	None	None
Location	-	-	-	-
Spacing	-	-	-	-
Size	-	-	-	-

Survey Team Member: Ala, Rambod

6. Observations & Photos

-Cracks on side walk
-Crack on North bound Approach slab

Figure 95. A view from underneath the bridge (Bridge No. 1013-151).

Figure 96. Girder end condition at the abutment (Bridge No. 1013-151).

Structure Number 1103-158

1.General Information

Bridge Location	Alexander RD Over US 1		
County	Mercer County	**Township**	West Windsor
Year Built	1996		

2.Structural Design Information

Number of Spans: 2
Direction of Traffic: Two ways
Girder type: Steel Girder
Span type: Simply Supported
Type Of Bearing: Steel Bearings
Skewness: None

	East Bound		West Bound	
Span Number	**1**	**2**	**1**	**2**
Span Width	67'-10"	67'-10"	43'-9"	43'-9"
Span Length	95'-3"	95'-3"	95'-3"	95'-3"

Framing Information:
-East Bound (All Spans): 9 Girders@7'-11" + 1 Girder@5'-3"
-West Bound (All Spans): 6 Girders@7'-11"

Deck Slab Information:

Deck Thickness	$8^{3/4}$"	**Top Cover**	$2^{1/2}$"
Wearing Surface	Concrete, Sawcut	**Bottom Cover**	1"

Longitudinal Rebar Information:
-All Spans EB & WB: #5@6"(B), #5@15"(T)
Transverse Rebar Information:
-All Spans EB & WB: #6@6"(T&B)

Girder Information:

	East & West Bound		
Span Number	**I (in^4)**	**A (in^2)**	**Depth (in)**
1&2 (Midspan)	38457.26	96.375	45.125~45.5
1&2 (Support)	24587.67	68.875	45.125~45.5

Shear Stud Information:

	East & West Bound	
Span Number	**Spacing (in)**	**Number of Studs in Row**
1&2 (Midspan)	14"	4
1&2 (Support)	9"	4

3.Material Properties & Mix Design Information

Cement Type: White Hall Type II
Cement Content: 678 lb/yd^3
Water Content: 32.4 Gal/yd^3
W/C Ratio: 0.40
Air Content: 5.4%
Compressive Strength: 5076 Psi
Slump: 3.1"
Admixtures: Water Reducer & Retarder

4.Construction Information

Air Temperature: 56.9 F
Concrete Temperature: 72.6 F
Month Of Casting: April

5.Survey results

Crack Information:

	East Bound		West Bound	
Span No.	**1**	**2**	**1**	**2**
Type	None	None	None	None
Location	-	-	-	-
Spacing	-	-	-	-
Size	-	-	-	-

Survey Team Member: Ala, Rambod

6. Observations & Photos

-None

Structure Number 1130-152

1.General Information

Bridge Location	NJ Route 29 WB Over Watson's Creek		
County	Mercer	**Township**	Hamilton
Year Built	1995		

2.Structural Design Information

Number of Spans: 12
Direction of Traffic: One way
Girder type: Prestressed Concrete Girder
Span type: Mixed (1 Simply Supported + 1 Simply Supported+ 1 Simply Supported+ 3 Continuous+ 3 Continuous+ 3 Continuous)
Type Of Bearing: Steel Bearings
Skewness: None

	West Bound			
Span Number	**1**	**2**	**3**	**4**
Span Width	59'-1", 75'-11"~51'-11"	111'~95'-3"	95'-3"~90'-1"	90'-1"~88'-9"
Span Length	88'-10"	90'	90'	90'

	West Bound			
Span Number	**5**	**6**	**7**	**8**
Span Width	88'-9"~87'-6"	87'-6"~86'-3"	86'-3"~84'-7"	84'-7"~82'-4"
Span Length	90'	90'	60'	60'

	West Bound			
Span Number	**9**	**10**	**11**	**12**
Span Width	82'-4"~80'-6"	80'-6"	80'-6"	80'-6"
Span Length	60'	90'	90'	88'-10"

Framing Information:
-West Bound:
-Span 1: 8 Girders @7'-9" + 2 Girders @8'-5" + 6 Girders @9'~4'-11"
-Span 2: 10 Girders @8'-5" + 3 Girders @9'-8" ~5'-1"
-Span 3: 10 Girders @8'-5" + 2 Girders @7'-6" ~4'-9"
-Span 4: 10 Girders @8'-5" + 1 Girders @9'-6" ~8'-3"
-Span 5: 10 Girders @8'-5" + 1 Girders @8'-2" ~7'-0"
-Span 6: 10 Girders @8'-5" + 1 Girders @6'-11" ~5'-8"
-Span 7: 8 Girders @8'-5" + 3 Girders @7'-6" ~6'-10"
-Span 8: 8 Girders @8'-5" + 3 Girders @6'-10" ~6'-3"
-Span 9: 8 Girders @8'-5" + 3 Girders @6'-3" ~5'-7"
-Span 10-11-12: 10 Girders @8'-5"

Deck Slab Information:

Deck Thickness	$7^{1/2"}$+ $1^{1/4"}$ Latex Concrete	**Top Cover**	$2^{3/4}$"
Wearing Surface	Latex Concrete, Sawcut	**Bottom Cover**	1"

Longitudinal Rebar Information:
-All Spans WB Midspan: #5@$7^{1/2}$"(B), #5@15"(T)
-All Spans WB Pier: #5@$7^{1/2}$"(B), #5@$7^{1/2}$"(T)
Transverse Rebar Information:
-All Spans WB: #6@6"(T&B)

Girder Information:

	West Bound		
Span Number	**I (in^4)**	**A (in^2)**	**Depth (in)**
1-6 & 10-12	260034	789.0	54
7-8-9	125014	559.5	45

Shear Stud Information:

	West Bound	
Span Number	**Spacing (in)**	**Number of Studs in Row**
1-6 & 10-12 (Midspan)	24”	2
1-6 &10-12(Support)	6”	2
7,8,9 (Midspan)	18”	2
7,8,9 (Support)	12”	2

3.Material Properties & Mix Design Information

Cement Type: Essroc type II
Cement Content: 700 lb/yd^3
Water Content: 31.8 Gal/yd^3
W/C Ratio: 0.38
Air Content: 5.5 %
Compressive Strength: Psi
Slump: 3”
Admixtures: Water Reducer & Retarder

4.Construction Information

Air Temperature: 51.3 F
Concrete Temperature: 72.1 F
Month Of Casting: November-December

5.Survey results

Crack Information:

	West Bound
Span No.	**All Spans**
Type	None
Location	-
Spacing	-
Size	-

Survey Team Member: Ala, Fred and Tom

6. Observations & Photos

-Continuity Joints cracked on this bridge but these joints did not cracked on the eastbound side bridge, which is similar to this bridge. Cracks are observed on eastbound side deck while the westbound side deck was un-cracked.

Figure 97. Cracked continuity joint (Bridge No. 1130-152).

Figure 98. Another cracked continuity joint (Bridge No.1130-152).

Figure 99. End condition in simply supported spans (Bridge No 1130-152).

Structure Number 1130-153

1.General Information

Bridge Location	NJ Route 29 EB Over Watson's Creek		
County	Mercer	**Township**	Hamilton
Year Built	1995		

2.Structural Design Information

Number of Spans: 11
Direction of Traffic: One way
Girder type: Prestressed Concrete Girder
Span type: Mixed (2 Continuous + 3 Continuous+ 3 Continuous+ 3 Continuous)
Type Of Bearing: Steel Bearings
Skewness: None

East Bound

Span Number	**1**	**2**	**3**
Span Width	55'-9", 47'-9"~50'-7"	55'-9", 50'-7"~52'-11"	55'-9", 52'-11"~54'-7"
Span Length	88'-10"	90'	90'

East Bound

Span Number	**4**	**5**	**6-9**
Span Width	55'-9", 54'-7"~55'-6"	55'-9", 55'-6"~55'-9"	55'-9", 55'-9"
Span Length	90'	90'	90'

East Bound

Span Number	**10**	**11**
Span Width	55'-9", 55'-9"~58'-8"	55'-9", 58'-8"~71'-0"
Span Length	90'	88'-10"

Framing Information:
-West Bound:
-Span 1: 7 Girders @8'-6", 4 Girders @8'-6" + 2 Girders @4'-6"~5'-9"
-Span 2: 7 Girders @8'-6", 4 Girders @8'-6" + 2 Girders @5'-9"~7'-1"
-Span 3: 7 Girders @8'-6", 4 Girders @8'-6" + 2 Girders @7'-1"~7'-7"
-Span 4: 7 Girders @8'-6", 4 Girders @8'-6" + 2 Girders @7'-7"~8'-0"
-Span 5: 7 Girders @8'-6", 4 Girders @8'-6" + 2 Girders @8'-0"~8'-6"
-Span 6-8: 7 Girders @8'-6", 6 Girders @8'-6"
-Span 9: 7 Girders @8'-6", 5 Girders @8'-6" + 1 Girders @8'-6"~9'-4"
-Span 10: 7 Girders @8'-6", 5 Girders @8'-6" + 1 Girders @9'-4"~10-2"
-Span 11: 7 Girders @8'-6", 4 Girders @8'-6" + 4 Girders @4'-8"~8'-1"

Deck Slab Information:

Deck Thickness	$7^{3/4}+1^{1/4}$" Latex Concrete	**Top Cover**	$2^{3/4}$"
Wearing Surface	Latex Concrete, Sawcut	**Bottom Cover**	1"

Longitudinal Rebar Information:
- All Spans EB Midspan: #5@$7^{1/2}$"(B), #5@15"(T)
- All Spans EB Pier: #5@$7^{1/2}$"(B), #5@$7^{1/2}$"(T)

Transverse Rebar Information:
- All Spans EB: #6@6"(T&B)

Girder Information:

East Bound

Span Number	I (in^4)	A (in^2)	Depth (in)
All	260034	789.0	54

Shear Stud Information:

	East Bound	
Span Number	**Spacing (in)**	**Number of Studs in Row**
1-6 & 10-12 (Midspan)	24”	2
1-6 &10-12(Support)	6”	2

3.Material Properties & Mix Design Information

Cement Type: Essroc type II
Cement Content: 700 lb/yd^3
Water Content: 31.8 Gal/yd^3
W/C Ratio: 0.38
Air Content: 3.2 %
Compressive Strength: 5933 Psi
Slump: 5.6”
Admixtures: Water Reducer &Retarder

4.Construction Information

Air Temperature: 49.5 F
Concrete Temperature: 70.3 F
Month Of Casting: November-December

5.Survey results

Crack Information:

	East Bound
Span No.	**All Spans**
Type	Transverse
Location	All over
Spacing	3-4’
Size	Typical

Survey Team Member: Ala, Fred and Tom

6. Observations & Photos

-Continuity Joints did not cracked but these joints cracked on the westbound side. Cracks are observed on eastbound side.

Figure 100. Transverse cracks on the deck (Bridge No.1130-153).

Figure 101. Un-cracked continuity joint (Bridge No.1130-153).

Structure Number 1130-154

1.General Information

Bridge Location	RT NJ29 Freeway Over Ramps G, H, I, J and Conrail B Town		
County	Mercer County	**Township**	Hamilton
Year Built	1995		

2.Structural Design Information

Number of spans: 2
Direction of Traffic: Two ways
Girder type: Steel Girder
Span type: Continuous
Type Of Bearing: Steel Bearings
Skewness: Mild

	South Bound		North Bound	
Span Number	**1**	**2**	**1**	**2**
Span Width	66’	66’	70’-5”	70’-5”
Span Length	151’	98’	98’	151’

Framing Information:
-South Bound (All Spans): 8 Girders@8’-9”
-North Bound (All Spans): 8 Girders@9’-3”

Deck Slab Information:

Deck Thickness	$7^{3/4}+1^{1/4}$” Latex Concrete	**Top Cover**	$2^{3/4}$”
Wearing Surface	Latex Concrete, Sawcut	**Bottom Cover**	1”

Longitudinal Rebar Information:
-Mid Span: #5@$7^{1/2}$”(B), #5@15”(T)
-Over Pier: #5@$7^{1/2}$”(T&B)
Transverse Rebar Information:
-All Spans: #6@6”(T&B)

Girder Information:

North Bound

Span Number	I (in^4)	A (in^2)	Depth (in)
1 (Support)	147906~256541	130.6~198.5	80.75~83
1 (Midspan)	147906~196700	130.6~163.5	80.75~81.75
2 (Support)	162400~256541	137.6~198.5	81~83
2 (Midspan)	159928~208896	137.6~170	81~82

South Bound

Span Number	I (in^4)	A (in^2)	Depth (in)
1 (Support)	91025~206425	111~165	68.75~82.00
1 (Midspan)	91025~159928	111~137	68.75~81.00
2 (Support)	84317~206425	105~165	68.5~82.00
2 (Midspan)	84317~137106	105~123	68.5~80.5

Shear Stud Information:

North & South Bound

Span Number	Spacing (in)	Number of Studs in Row
Support	8~12	4
Midspan	8~23	4

3.Material Properties & Mix Design Information

Cement Type: Essroc Type II
Cement Content: 700 lb/yd^3
Water Content: 31.8 Gal/yd^3
W/C Ratio: 0.37
Air Content: 5.4%
Compressive Strength: 6169 Psi
Slump: 3.1”
Admixtures: Water Reducer & Retarder

4.Construction Information

Air Temperature: 73 F
Concrete Temperature: 50 F
Month Of Casting: November-December

5.Survey results

Crack Information:

	North Bound		South Bound	
Span No.	**1**	**2**	**1**	**2**
Type	None	None	None	None
Location	-	-	-	-
Spacing	-	-	-	-
Size	-	-	-	-

Survey Team Member: Ala., Tom and Fred

6. Observations & Photos

-Approach slab cracked.

Structure Number 1130-155

1.General Information

Bridge Location	RT NJ29 Ramp H Over Ramp G and Conrail B Town		
County	Mercer County	**Township**	Hamilton
Year Built	1995		

2.Structural Design Information

Number of spans: 2
Direction of Traffic: One way
Girder type: Steel Girder
Span type: Continuous
Type Of Bearing: Steel Bearings
Skewness: Severe

	Ramp	
Span Number	**1**	**2**
Span Width	44’-3”	44’-3”
Span Length	162’-0”	159’-9”

Framing Information:
-All Spans: 5 Girders@9’-3”

Deck Slab Information:

Deck Thickness	9”	**Top Cover**	$2^{1/2}$”
Wearing Surface	Concrete, Sawcut	**Bottom Cover**	1”

Longitudinal Rebar Information:
-Mid Span: #5@8”(B), #5@15”(T)
-Over Pier: #5@8”(B), #5@5”(T)
Transverse Rebar Information:
-All Spans: #6@6”(T&B)

Girder Information:

		Ramp	
Span Number	**I (in^4)**	**A (in^2)**	**Depth (in)**
1 (Support)	194714~301013	173~220	75~84
1 (Midspan)	116694~301013	117~220	75~84
2 (Support)	194714~301013	173~220	75~84
2 (Midspan)	116694~301013	117~220	75~84

Shear Stud Information:

	Ramp	
Span Number	**Spacing (in)**	**Number of Studs in Row**
All Spans	24	4

3.Material Properties & Mix Design Information

Cement Type: Essroc Type II
Cement Content: 700 lb/yd^3
Water Content: 31.8 Gal/yd^3
W/C Ratio: 0.37
Air Content: 5.6%
Compressive Strength: 6245 Psi
Slump: 3.4”
Admixtures: Water Reducer & Retarder

4.Construction Information

Air Temperature: 78 F
Concrete Temperature: 81 F
Month Of Casting: July

5.Survey results

Crack Information:

	Ramp	
Span No.	**1**	**2**
Type	None	None
Location	-	-
Spacing	-	-
Size	-	-

Survey Team Member: Ala., Tom and Fred

6. Observations & Photos

-Approach slab cracked.

Structure Number 1130-156

1.General Information

Bridge Location	RT NJ29 Ramp F Over Ramp G and Conrail B Town		
County	Mercer County	**Township**	Hamilton
Year Built	1995		

2.Structural Design Information

Number of spans: 4
Direction of Traffic: 1
Girder type: Steel Girder
Span type: 3 Span Continuous (1,2, &3) + 1 Span Simply Supported (4)
Type Of Bearing: Steel Bearings
Skewness: Mild

		Ramp		
Span Number	**1**	**2**	**3**	**4**
Span Width	33’-6”	33’-6”	33’-6”	33’-6”
Span Length	124’-11”	147’-11”	139’-11”	49’-11”

Framing Information:
-All Spans: 5 Girders@7’-3”

Deck Slab Information:

Deck Thickness	$8^{1/2}$”	**Top Cover**	$2^{1/2}$”
Wearing Surface	Concrete, Sawcut	**Bottom Cover**	1”

Longitudinal Rebar Information:
-Span 1-3 (Mid Span): #5@9”(B), #5@15”(T)
-Span 1-3 (Over Pier): #5@9”(B), #5@5”(T)
-Span 4: #5@9”(B), #5@15”(T)
Transverse Rebar Information:
-Span 1-3: #6@9”(T&B)
-Span 4: #6@7”(T&B)

Girder Information:

		Ramp	
Span Number	**I (in^4)**	**A (in^2)**	**Depth (in)**
1 (Support)	73512.66~118564.4	90.5~126.5	70
1 (Midspan)	73512.66	90.5	70
2 (Support)	118564.4	126.5	70
2 (Midspan)	73512.66	90.5	70
3 (Support)	73512.66~118564.4	90.5~126.5	70
3 (Midspan)	73512.66	90.5	70
4	9750~34371	47~52	36~70

Shear Stud Information:

	Ramp	
Span Number	**Spacing (in)**	**Number of Studs in Row**
1(Midspan)	21	3
1(Support)	21~24	3
2(Midspan)	16	3
2(Support)	24	3
3(Midspan)	18	3

3(Support)	18~24	3
4	9~15	3

3.Material Properties & Mix Design Information

Cement Type: Essroc Type II
Cement Content: 700 lb/yd^3
Water Content: 31.8 Gal/yd^3
W/C Ratio: 0.37
Air Content: 5.8%
Compressive Strength: 5245 Psi
Slump: 3.4"
Admixtures: Water Reducer & Retarder

4.Construction Information

Air Temperature: 75 F
Concrete Temperature: 78 F
Month Of Casting: September

5.Survey results

Crack Information:

	Ramp			
Span No.	**1**	**2**	**3**	**4**
Type	None	None	None	None
Location	-	-	-	-
Spacing	-	-	-	-
Size	-	-	-	-

Survey Team Member: Ala., Tom and Fred

6. Observations & Photos

Approach slab cracked.

Structure Number 1136-154

1.General Information

Bridge Location	Route I-295 South Bound Over RT I-195		
County	Mercer	Township	Hamilton
Year Built	1995		

2.Structural Design Information

Number of spans: 2
Direction of Traffic: One way
Girder type: Steel Girder
Span type: Simply Supported
Type Of Bearing: Steel Bearing
Skewness: None

	South Bound	
Span Number	**1**	**2**
Span Width	97'-6"	97'-6"
Span Length	129'	97'

Framing Information:
-All Spans: 6 Girders@10'-3"+ 1 Girder@5'+ 4 Girder @8'

Deck Slab Information:

Deck Thickness	$8^{1/2"}$+ $1^{1/4"}$ Latex Concrete	**Top Cover**	$2^{3/4"}$
Wearing Surface	Latex concrete, Sawcut	**Bottom Cover**	1"

Longitudinal Rebar Information:
-All Spans: #5@15" (T), #6@6" (B)
Transverse Rebar Information:
-All Spans: #6@6"(T&B)

Girder Information:

		South Bound	
Span Number	**I (in^4)**	**A (in^2)**	**Depth (in)**
1 (Support)	55001~62590	65~73.75	79
1 (Midspan)	64196~72198	72~81.75	79
2 (Support)	80990~100124	88.5~100	80
2 (Midspan)	106866~130627	104.5~119.5	81.5

Shear Stud Information:

		South Bound
Span Number	**Spacing (in)**	**Number of Studs in Row**
All Spans (Midspan)	15~18	4
All Spans (Support)	9~10	4

3.Material Properties & Mix Design Information

Cement Type: Essroc Type II
Cement Content: 700 lb/yd^3
Water Content: 31.8 lb/yd^3
W/C Ratio: 0.38
Air Content: 5.6 %
Compressive Strength: 5755 Psi
Slump: 3.1"

Admixtures: Water Reducer & Retarder

4.Construction Information

Air Temperature: 55 F
Concrete Temperature: 72 F
Month Of Casting: November

5.Survey results

Crack Information:

	Ramp	
Span No.	**1**	**2**
Type	None	None
Location	-	-
Spacing	-	-
Size	-	-

Survey Team Member: Fred, Ala

6. Observations & Photos

-Approach slab cracked

Structure Number 1143-166

1.General Information

Bridge Location	Route 133 EB Over One Mile Road
County	Mercer County
Year Built	1998

2.Structural Design Information

Number of Spans: 1
Direction of Traffic: One way
Girder type: Prestressed Concrete
Span type: Simply Supported
Type Of Bearing: Elastomeric Pad
Skewness: None

	East Bound
Span Number	**1**
Span Width	42’
Span Length	91’-11”

Framing Information:
-East Bound (All Spans): 5 Girders@9’-3”

Deck Slab Information:

Deck Thickness	8.75”	**Top Cover**	2.5”
Wearing Surface	Concrete, Saw cut	**Bottom Cover**	1”

Longitudinal Rebar Information:
-Longitudinal Bars (All Spans): #5@15 (T), #5@9 (B)
Transverse Rebar Information:
-Transverse Bars (All Spans EB): #6@7 (T&B)

Girder Information:

		East Bound	
Span Number	**I (in^4)**	**A (in^2)**	**Depth (in)**
1	540516	1053	63

Shear Stud Information:

	East Bound	
Span Number	**Spacing (in)**	**Number of Studs in Row**
1 (Midspan)	24	6
1(Support)	12	6

3.Material Properties & Mix Design Information

Cement Type: Essroc Type II
Cement Content: 700 lb/yd^3
Water Content: 31.8 lb/yd^3
W/C Ratio: 0.38
Air Content: 5.2 %
Compressive Strength: 5800 Psi
Slump: 3.5”
Admixtures: Water Reducer &Retarder

4.Construction Information

Air Temperature: 60F
Concrete Temperature: 78F
Month Of Casting: June-July

5.Survey results

Crack Information:

	East Bound
Span No.	**1**
Type	Transverse
Location	All Over
Spacing	3'-5'
Size	Wide

Survey Team Member: Frank, Fred, Ala, and Tom

6. Observations & Photos

-Cracks begin parallel to the bridge longitudinal axis at the ends and change direction and become normal to the axis as it goes towards the center of span.
-Bridge end diaphragm cast around the girders.
-Approach slabs cracked.

Figure 102. Deck cracks at the bridge end (Bridge No. 1143-466).

Figure 103. Wide view of deck cracks at the bridge end (Bridge No. 1143-466).

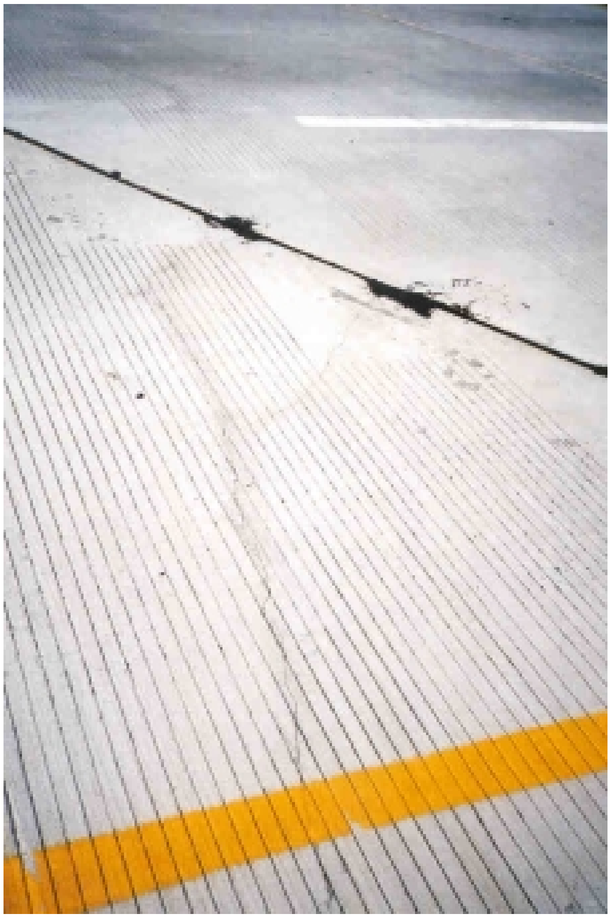


Figure 104. Close view of a crack (Bridge No. 1143-166).

Structure Number 1143-167

1.General Information

Bridge Location	Route 133 WB Over One Mile Road
County	Mercer County
Year Built	1998

2.Structural Design Information

Number of Spans: 1
Direction of Traffic: One way
Girder type: Prestressed concrete
Span type: Simply supported
Type Of Bearing: Elastomeric Pads
Skewness: None

	West Bound
Span Number	**1**
Span Width	42’
Span Length	91’-11”

Framing Information:
-West Bound (All Spans): 5 Girders@9’-3”

Deck Slab Information:

Deck Thickness	8.75”	**Top Cover**	2.5”
Wearing Surface	Concrete, Saw cut	**Bottom Cover**	1”

Longitudinal Rebar Information:
All Spans: #5@15 (T), #5@9 (B)
Transverse Rebar Information:
All Spans EB: #6@7 (T&B)

Girder Information:

		West Bound	
Span Number	**I (in^4)**	**A (in^2)**	**Depth (in)**
1	540516	1053	63

Shear Stud Information:

	West Bound	
Span Number	**Spacing (in)**	**Number of Studs in Row**
1 (Midspan)	24	6
1 (Support)	12	6

3.Material Properties & Mix Design Information

Cement Type: Essroc Type II
Cement Content: 700 lb/yd^3
Water Content: 31.8 lb/yd^3
W/C Ratio: 0.38
Air Content: 5 %
Compressive Strength: 5815 Psi
Slump: 3”
Admixtures: Water Reducer & Retarder

4.Construction Information

Air Temperature: 80F
Concrete Temperature: 70F
Month Of Casting: December- November

5.Survey results

Crack Information:

	West Bound
Span No.	**1**
Type	Transverse
Location	All Over
Spacing	3’-5’
Size	Wide

Survey Team Member: Fred, Ala, Tom, and Frank

6. Observations & Photos

-Cracks begin parallel to the bridge longitudinal axis at the ends and change direction and become normal to the axis as it goes towards the center of span.
-Bridge end diaphragm cast around the girders.
-Approach slabs cracked.

Structure Number 1143-168

1.General Information

Bridge Location	Route 133 EB Over Rocky Brook
County	Mercer County
Year Built	1998

2.Structural Design Information

Number of Spans: 2
Direction of Traffic: Two ways
Girder type: Prestressed concrete
Span type: Continuous
Type Of Bearing: Elastomeric Pads
Skewness: None

	East Bound	
Span Number	**1**	**2**
Span Width	40'-5"	40'-5"
Span Length	110'	110'

Framing Information:
-East Bound (All Spans): 5 Girders@8'-10"

Deck Slab Information:

Deck Thickness	8.5"	**Top Cover**	2.5"
Wearing Surface	Concrete, Saw cut	**Bottom Cover**	1"

Longitudinal Rebar Information:
-All Spans EB Midspan: #5@15 (T) #5@9 (B)
-All Spans EB Pier: #8@5 (T) #7@9 (B)
Transverse Rebar Information:
-All Spans EB: #6@7 (T&B)

Girder Information:

	East Bound		
Span Number	**I (in^4)**	**A (in^2)**	**Depth (in)**
1&2 (Midspan)	540516	1053	63
1&2 (Support)	540516	1053	63

Shear Stud Information:

	East Bound	
Span Number	**Spacing (in)**	**Number of Studs in Row**
1&2 (Midspan)	12	6
1&2 (Support)	6	6

3.Material Properties & Mix Design Information

Cement Type: Essroc Type II
Cement Content: 700 lb/yd^3
Water Content: 31.8 lb/yd^3
W/C Ratio: 0.38
Air Content: 6.7%
Compressive Strength: 6352 Psi
Slump: 3.5"
Admixtures: Water Reducer & Retarder

4.Construction Information

Air Temperature: 47F
Concrete Temperature: 68F
Month Of Casting: October

5.Survey results

Crack Information:

	East Bound	
Span No.	**1**	**2**
Type	Transverse	Transverse
Location	All over	All over
Spacing	0-2’	0-2’
Size	Typical	Typical

Survey Team Member: Fred, Ala, Tom, and Frank

6. Observations & Photos

-Cracks begin parallel to the bridge longitudinal axis at the ends and change direction and become normal to the axis as it goes towards the center of span.
-Bridge end diaphragm cast around the girders.
-Approach slabs cracked.

Figure 105. Girders embedded in end diaphragm (Bridge No.1143-168).

Figure 106. Continuity joint (Bridge No. 1143-168).

Structure Number 1143-169

1.General Information

Bridge Location	Route 133 WB Over Rocky Brook
County	Mercer County
Year Built	1998

2.Structural Design Information

Number of Spans: 2
Direction of Traffic: One way
Girder type: Prestressed concrete
Span type: Continuous
Type Of Bearing: Elastomeric Pads
Skewness: None

	West Bound	
Span Number	**1**	**2**
Span Width	40’-5”	40’-5”
Span Length	110’	110’

Framing Information:
-West Bound (All Spans): 5 Girders@8’-10”

Deck Slab Information:

Deck Thickness	8.5”	**Top Cover**	2.5”
Wearing Surface	Concrete, Saw cut	**Bottom Cover**	1”

Longitudinal Rebar Information:
All Spans WB Midspan: #5@15 (T) #5@9 (B)
All Spans WB Pier: #8@5 (T) #7@9 (B)
Transverse Rebar Information:
All Spans WB: #6@7 (T&B)

Girder Information:

	West Bound		
Span Number	**I (in^4)**	**A (in^2)**	**Depth (in)**
1&2 (Midspan)	540516	1053	63
1&2 (Support)	540516	1053	63

Shear Stud Information:

	West Bound	
Span Number	**Spacing (in)**	**Number of Studs in Row**
1&2 (Midspan)	12	6
1&2 (Support)	6	6

3.Material Properties & Mix Design Information

Cement Type: Essroc Type II
Cement Content: 700 lb/yd^3
Water Content: 31.8 lb/yd^3
W/C Ratio: 0.38
Air Content: 6%
Compressive Strength: 6623 Psi
Slump: 3.5”
Admixtures: Water Reducer & Retarder

4.Construction Information

Air Temperature: 46F
Concrete Temperature: 72F
Month Of Casting: November

5.Survey results

Crack Information:

	West Bound	
Span No.	**1**	**2**
Type	Transverse	Transverse
Location	All over	All over
Spacing	0-2'	0-2'
Size	Typical	Typical

Survey Team Member: Tom, Fred, Ala, and Frank

6. Observations & Photos

-Cracks begin parallel to the bridge longitudinal axis at the ends and change direction and become normal to the axis as it goes towards the center of span.
-Bridge end diaphragm cast around the girders.
-Approach slabs cracked.

Figure 107. Transverse cracks on bridge deck (Bridge No. 1143-169).

Structure Number 1143-170

1.General Information

Bridge Location	Route 133 EB Over Route 130
County	Mercer County
Year Built	1998

2.Structural Design Information

Number of Spans: 2
Direction of Traffic: One way
Girder type: Prestressed concrete
Span type: Continuous
Type Of Bearing: Elastomeric Pads
Skewness: None

	East Bound	
Span Number	**1**	**2**
Span Width	52'-6"	52'-6"
Span Length	80'-6"	87'

Framing Information:
-East Bound (All Spans): 6 Girders@9'-6"

Deck Slab Information:

Deck Thickness	8.75"	**Top Cover**	2.5"
Wearing Surface	Concrete, Saw cut	**Bottom Cover**	1"

Longitudinal Rebar Information:
-All Spans EB Midspan: #7@10"+#5@10"(T), #7@9"(B)
-All Spans EB Pier: #8@10" + #7@10" (T), #7@9" (B)
Transverse Rebar Information:
All Spans EB: #6@7" (T&B)

Girder Information:

	East Bound		
Span Number	**I (in^4)**	**A (in^2)**	**Depth (in)**
1&2 (Midspan)	260034	789	54
1&2 (Support)	260034	789	54

Shear Stud Information:

	East Bound	
Span Number	**Spacing (in)**	**Number of Studs in Row**
1&2 (Midspan)	24	2
1&2 (Support)	12	2

3.Material Properties & Mix Design Information

Cement Type: Essroc Type II
Cement Content: 700 lb/yd^3
Water Content: 31.8 lb/yd^3
W/C Ratio: 0.38
Air Content: 5.3 %
Compressive Strength: 4935 Psi
Slump: 4.0"
Admixtures: Water Reducer & Retarder

4.Construction Information

Air Temperature: 76F
Concrete Temperature: 84F
Month Of Casting: May

5.Survey results

Crack Information:

	East Bound	
Span No.	**1**	**2**
Type	Transverse	Transverse
Location	All over	All over
Spacing	3-5"	3-5"
Size	Typical	Typical

Survey Team Member: Fred, Ala, Tom, and Frank

6. Observations & Photos

-Cracks begin parallel to the bridge longitudinal axis at the ends and change direction and become normal to the axis as it goes towards the center of span.
-Bridge end diaphragm cast around the girders.
-Approach slabs cracked.

Figure 108. Repaired transverse crack (Bridge No. 1143-170).

Figure 109. Close up view of repair patch (Bridge No. 1143-170).

Structure Number 1143-171

1.General Information

Bridge Location	Route 133 WB Over Route 130
County	Mercer County
Year Built	1998

2.Structural Design Information

Number of Spans: 2
Direction of Traffic: One way
Girder type: Prestressed concrete
Span type: Continuous
Type Of Bearing: Elastomeric Pads
Skewness: None

	West Bound	
Span Number	**1**	**2**
Span Width	52’-6”	52’-6”
Span Length	87’	80’-6”

Framing Information:
-West Bound (All Spans): 6 Girders@9’-6”

Deck Slab Information:

Deck Thickness	8.75”	**Top Cover**	2.5”
Wearing Surface	Concrete, Saw cut	**Bottom Cover**	1”

Longitudinal Rebar Information:
-All Spans WB Midspan: #7@10”+#5@10”(T), #7@9”(B)
-All Spans WB Pier: #8@10” + #7@10” (T), #7@9” (B)
Transverse Rebar Information:
-All Spans WB: #6@7” (T&B)

Girder Information:

	West Bound		
Span Number	**I (in^4)**	**A (in^2)**	**Depth (in)**
1&2 (Midspan)	260034	789	54
1&2 (Support)	260034	789	54

Shear Stud Information:

	West Bound	
Span Number	**Spacing (in)**	**Number of Studs in Row**
1&2 (Midspan)	24	2
1&2 (Support)	12	2

3.Material Properties & Mix Design Information

Cement Type: Essroc Type II
Cement Content: 700 lb/yd^3
Water Content: 31.8 lb/yd^3
W/C Ratio: 0.38
Air Content: 5%
Compressive Strength: 6192 Psi
Slump: 3.4”
Admixtures: Water Reducer & Retarder

4.Construction Information

Air Temperature: 72F
Concrete Temperature: 62F
Month Of Casting: June

5.Survey results

Crack Information:

	West Bound	
Span No.	**1**	**2**
Type	Transverse	Transverse
Location	All over	All over
Spacing	3-5”	3-5”
Size	Typical	Typical

Survey Team Member: Fred, Ala, Tom, and Frank

6. Observations & Photos

-Cracks begin parallel to the bridge longitudinal axis at the ends and change direction and become normal to the axis as it goes towards the center of span.
-Bridge end diaphragm cast around the girders.
-Approach slabs cracked.

Figure 110. Cracks on the deck (Bridge No.1143-171).

Figure 111. Another marked crack on the deck (Bridge No.1143-171).

Structure Number 1143-172

1.General Information

Bridge Location	Route 133 EB Over North Main Street
County	Mercer County
Year Built	1998

2.Structural Design Information

Number of Spans: 1
Direction of Traffic: One way
Girder type: Prestressed concrete
Span type: Simply Supported
Type Of Bearing: Elastomeric Pads
Skewness: None

	East Bound
Span Number	**1**
Span Width	42’
Span Length	76’-6”

Framing Information:
-East Bound (All Spans): 5 Girders@ 9’-3”

Deck Slab Information:

Deck Thickness	9.25”	**Top Cover**	2.5””
Wearing Surface	Concrete, Saw cut	**Bottom Cover**	1”

Longitudinal Rebar Information:
-All Spans EB: #5@15”(T), #5@9”(B)
Transverse Rebar Information:
-All Spans EB: #6@7 (T&B)

Girder Information:

		East Bound	
Span Number	**I (in^4)**	**A (in^2)**	**Depth (in)**
1 (Midspan)	260034	789	54
1 (Support)	260034	789	54

Shear Stud Information:

		East Bound
Span Number	**Spacing (in)**	**Number of Studs in Row**
1 (Midspan)	24	2
1 (Support)	12	2

3.Material Properties & Mix Design Information

Cement Type: Essroc Type II
Cement Content: 700 lb/yd^3
Water Content: 31.8 lb/yd^3
W/C Ratio: 0.38
Air Content: 6.1 %
Compressive Strength: 5266 Psi
Slump: 3.4”
Admixtures: Water Reducer & Retarder

4.Construction Information

Air Temperature: 63F
Concrete Temperature: 53F
Month Of Casting: May

5.Survey results

Crack Information:

	East Bound
Span No.	**1**
Type	Transverse
Location	All over
Spacing	3-5'
Size	Typical

Survey Team Member: Ala, Fred, Tom, and Frank

6. Observations & Photos

-Cracks begin parallel to the bridge longitudinal axis at the ends and change direction and become normal to the axis as it goes towards the center of span.
-Bridge end diaphragm cast around the girders.
-Approach slabs cracked.

Figure 112. Close up view of deck cracks at the ends (Bridge No.1143-172).

Structure Number 1143-173

1.General Information

Bridge Location Route 133 WB Over North Main Street
County Mercer County
Year Built 1998

2.Structural Design Information

Number of Spans: 1
Direction of Traffic: One way
Girder type: Prestressed concrete
Span type: Simply supported
Type Of Bearing: Elastomeric Pads
Skewness: None

	West Bound
Span Number	**1**
Span Width	42”
Span Length	76’-6”

Framing Information:
-West Bound (All Spans): 5 Girders@9’-3”

Deck Slab Information:

Deck Thickness	9.25”	**Top Cover**	2.5”
Wearing Surface	Concrete, Saw cut	**Bottom Cover**	1”

Longitudinal Rebar Information:
-All Spans EB: #5@15”(T), #5@9”(B)
Transverse Rebar Information:
-All Spans EB: #6@7 (T&B)

Girder Information:

		West Bound	
Span Number	**I (in^4)**	**A (in^2)**	**Depth (in)**
1 (Midspan)	260034	789	54
1 (Support)	260034	789	54

Shear Stud Information:

	West Bound	
Span Number	**Spacing (in)**	**Number of Studs in Row**
1 (Midspan)	24	2
1 (Support)	12	2

3.Material Properties & Mix Design Information

Cement Type: Essroc Type II
Cement Content: 700 lb/yd^3
Water Content: 31.8 lb/yd^3
W/C Ratio: 0.38
Air Content: 6.7%
Compressive Strength:
Slump: 3.8”
Admixtures: Water Reducer & Retarder

4.Construction Information

Air Temperature: 57F
Concrete Temperature: 65F
Month Of Casting: May

5.Survey results

Crack Information:

	West Bound
Span No.	**1**
Type	Transverse
Location	All over
Spacing	3-5’
Size	Typical

Survey Team Member: Ala, Frank, Tom, and Frank

6. Observations & Photos

-Cracks begin parallel to the bridge longitudinal axis at the ends and change direction and become normal to the axis as it goes towards the center of span.
-Bridge end diaphragm cast around the girders.
-Approach slabs cracked.

Structure Number 1143-174

1.General Information

Bridge Location	Route 133 EB Over Wyckoff 's Mill Road
County	Mercer County
Year Built	1998

2.Structural Design Information

Number of Spans: 1
Direction of Traffic: One way
Girder type: Prestressed concrete
Span type: Simply supported
Type Of Bearing: Elastomeric Pads
Skewness: None

	East Bound
Span Number	**1**
Span Width	42'
Span Length	78'-9"

Framing Information:
-East Bound (All Spans): 5 Girders@9'-3"

Deck Slab Information:

Deck Thickness	: 8.75"	**Top Cover**	2.5"
Wearing Surface	Concrete, Saw cut	**Bottom Cover**	1"

Longitudinal Rebar Information:
- All Spans EB: #5@15"(T), #5@9"(B)
-All Spans EB: #6@7 (T&B)
Transverse Rebar Information:
-All Spans EB): #6@7 (T&B)

Girder Information:

		West Bound	
Span Number	**I (in^4)**	**A (in^2)**	**Depth (in)**
1 (Midspan)	260034	789	54
1 (Support)	260034	789	54

Shear Stud Information:

		West Bound
Span Number	**Spacing (in)**	**Number of Studs in Row**
1 (Midspan)	24	2
1 (Support)	12	2

3.Material Properties & Mix Design Information

Cement Type: Essroc Type II
Cement Content: 700 lb/yd^3
Water Content: 31.8 lb/yd^3
W/C Ratio: 0.38
Air Content: 5.1%
Compressive Strength: 5928 Psi
Slump: 3.9
Admixtures: Water Reducer & Retarder

4.Construction Information

Air Temperature: 74F
Concrete Temperature: 78F
Month Of Casting: May

5.Survey results

Crack Information:

	East Bound
Span No.	**1**
Type	Transverse
Location	All over
Spacing	3-5'
Size	Typical

Survey Team Member: Tom, Ala, Fred, and Frank

6. Observations & Photos

-Cracks begin parallel to the bridge longitudinal axis at the ends and change direction and become normal to the axis as it goes towards the center of span.
-Bridge end diaphragm cast around the girders.
-Approach slabs cracked.

Structure Number 1143-175

1.General Information

Bridge Location	Route 133 WB Over Wyckoff's Mill Road
County	Mercer County
Year Built	1998

2.Structural Design Information

Number of Spans: 1
Direction of Traffic: One way
Girder type: Prestressed concrete
Span type: Simply supported
Type Of Bearing: Elastomeric Pads
Skewness: None

	West Bound
Span Number	**1**
Span Width	42'
Span Length	78'-9"

Framing Information:
-West Bound (All Spans): 5 Girders@9'-3"

Deck Slab Information:

Deck Thickness	8.75"	**Top Cover**	2.5"
Wearing Surface	Concrete, Saw cut	**Bottom Cover**	1"

Longitudinal Rebar Information:
-All Spans WB: #5@15"(T), #5@9"(B)
Transverse Rebar Information:
-All Spans WB: #6@7 (T&B)

Girder Information:

		West Bound	
Span Number	**I (in^4)**	**A (in^2)**	**Depth (in)**
1 (Midspan)	260034	789	54
1 (Support)	260034	789	54

Shear Stud Information:

	West Bound	
Span Number	**Spacing (in)**	**Number of Studs in Row**
1 (Midspan)	24	2
1 (Support)	12	2

3.Material Properties & Mix Design Information

Cement Type: Essroc Type II
Cement Content: 700 lb/yd^3
Water Content: 31.8 lb/yd^3
W/C Ratio: 0.38
Air Content: 5.9%
Compressive Strength: 4500 Psi
Slump: 3.1"
Admixtures: Water Reducer & Retarder

4.Construction Information

Air Temperature: 86.5 F
Concrete Temperature: 82F
Month Of Casting: May

5.Survey results

Crack Information:

	West Bound
Span No.	**1**
Type	Transverse
Location	All over
Spacing	3-5'
Size	Typical

Survey Team Member: Tom, Ala, Fred, and Frank

6. Observations & Photos

-Cracks begin parallel to the bridge longitudinal axis at the ends and change direction and become normal to the axis as it goes towards the center of span.
-Bridge end diaphragm cast around the girders.
-Approach slabs cracked.

Structure Number 1143-176

1.General Information

Bridge Location	Route 133 EB Over One NJ Turnpike
County	Mercer County
Year Built	1998

2.Structural Design Information

Number of Spans: 3
Direction of Traffic: One way
Girder type: Prestressed Concrete
Span type: Mixed (1 Simply supported +2 Continuous)
Type Of Bearing: Elastomeric Pads
Skewness: None

	East Bound		
Span Number	**1**	**2**	**3**
Span Width	41’	41’	41’
Span Length	94’	131’-3”	124’

Framing Information:
-East Bound (All Spans): 5 Girders@9’

Deck Slab Information:

Deck Thickness	: 8.5”	**Top Cover**	2.5”
Wearing Surface	Concrete, Saw cut	**Bottom Cover**	1”

Longitudinal Rebar Information:
-All Spans EB Midspan: #5@15(T), #5@9(B)
-EB Span 2&3 Over Pier: #6@7(T&B)
Transverse Rebar Information:
-All Spans EB: #6@7 (T&B)

Girder Information:

	East Bound		
Span Number	**I (in^4)**	**A (in^2)**	**Depth (in)**
All (Midspan)	540516	1053	63
All (Support)	540516	1053	63

Shear Stud Information:

	East Bound	
Span Number	**Spacing (in)**	**Number of Studs in Row**
All (Midspan)	24	6
1 (Support)	12	6
2&3 (Support)	6	6

3.Material Properties & Mix Design Information

Cement Type: Essroc Type II
Cement Content: 700 lb/yd^3
Water Content: 31.8 lb/yd^3
W/C Ratio: 0.38
Air Content: 5.9%
Compressive Strength: 5670 Psi
Slump: 3.4”

Admixtures: Retarder & Water Reducer

4.Construction Information

Air Temperature: 79.5F
Concrete Temperature: 90F
Month Of Casting: August

5.Survey results

Crack Information:

	East Bound		
Span No.	**1**	**2**	**3**
Type	Transverse	Transverse	Transverse
Location	All over	All over	All over
Spacing	3-5’	3-5’	3-5’
Size	Typical	Typical	Typical

Survey Team Member: Tom, Ala, Fred, and Frank

6. Observations & Photos

-Cracks begin parallel to the bridge longitudinal axis at the ends and change direction and become normal to the axis as it goes towards the center of span.
-Bridge end diaphragm cast around the girders.

Figure 113. End diaphragm (Bridge No.1143-176).

Figure 114. A view from underneath the bridge (Bridge No.1143-176).

Figure 115. Transverse deck cracks on Bridge No.1143-176 (NJ Turnpike in the background).

Structure Number 1143-177

1.General Information

Bridge Location	Route 133 WB Over NJ Turnpike
County	Mercer County
Year Built	1998

2.Structural Design Information

Number of Spans: 3
Direction of Traffic: One way
Girder type: Prestressed Concrete
Span type: Mixed (2 Continuous + 1 Simply supported)
Type Of Bearing: Elastomeric Pads
Skewness: None

		West Bound	
Span Number	**1**	**2**	**3**
Span Width	41’	41’	41’
Span Length	124’	131’-3”	94’

Framing Information:
-West Bound (All Spans): 5 Girders@9’

Deck Slab Information:

Deck Thickness	: 8.5”	**Top Cover**	2.5”
Wearing Surface	Concrete, Saw cut	**Bottom Cover**	1”

Longitudinal Rebar Information:
-All Spans WB Midspan: #5@15(T), #5@9(B)
-WB Span 1&2 Over Pier: #6@7(T&B)
Transverse Rebar Information:
-All Spans WB: #6@7 (T&B)

Girder Information:

		West Bound	
Span Number	**I (in^4)**	**A (in^2)**	**Depth (in)**
All (Midspan)	540516	1053	63
All (Support)	540516	1053	63

Shear Stud Information:

	West Bound	
Span Number	**Spacing (in)**	**Number of Studs in Row**
All (Midspan)	24	6
1&2 (Support)	6	6
3 (Support)	12	6

3.Material Properties & Mix Design Information

Cement Type: Essroc Type II
Cement Content: 700 lb/yd^3
Water Content: 31.8 lb/yd^3
W/C Ratio: 0.38
Air Content: 6.2%
Compressive Strength: 5351 Psi
Slump: 3.5”

Admixtures: Retarder & Water Reducer

4.Construction Information

Air Temperature: 63F
Concrete Temperature: 80F
Month Of Casting: August

5.Survey results

Crack Information:

	West Bound		
Span No.	**1**	**2**	**3**
Type	Transverse	Transverse	Transverse
Location	All over	All over	All over
Spacing	3-5'	3-5'	3-5'
Size	Typical	Typical	Typical

Survey Team Member: Tom, Ala, Fred, and Frank

6. Observations & Photos

-Cracks begin parallel to the bridge longitudinal axis at the ends and change direction and become normal to the axis as it goes towards the center of span.
-Bridge end diaphragm cast around the girders.
-Approach slabs cracked.

Figure 116. Transverse (and longitudinal) cracks on deck (Bridge No.1143-177).

Structure Number 1149-168

1.General Information

Bridge Location	Whitehead Road Over AMTRAK		
County	Mercer County	**Township**	Hamilton
Year Built	1999		

2. Structural Design Information

Number of Spans: 3
Direction of Traffic: Two ways
Girder type: Steel Girder
Span type: Continuous
Type Of Bearing: Steel Bearings
Skewness: None

	East & West Bound		
Span Number	**1**	**2**	**3**
Span Width	41'-0"	41'-0"	41'-0"
Span Length	103'-0"	175'-3"	103'-0"

Framing Information:
-East & West Bound (All Spans): 6 Girders@11'-0"

Deck Slab Information:

Deck Thickness	$8^{1/2"}$	**Top Cover**	$2^{1/2"}$
Wearing Surface	Concrete, Sawcut	**Bottom Cover**	1"

Longitudinal Rebar Information:
- All Spans EB & WB Midspan: #5@10"(B), #5@15"(T)
- All Spans EB & WB Pier: #5@10"(B), #5@7.5"(T)

Transverse Rebar Information:
- All Spans EB & WB: #5@7"(T&B)

Girder Information:

	East & West Bound		
Span Number	**I (in^4)**	**A (in^2)**	**Depth (in)**
1&3 (Midspan)	41206	90	50.5
1&3 (Support)	34576~65792	78~126	50~52
2 (Midspan)	41207	90	50.5
2 (Support)	65792	126	52

Shear Stud Information:

	East & West Bound	
Span Number	**Spacing (in)**	**Number of Studs in Row**
1&3 (Midspan)	18"	4
1&3 (Support)	14"~36"	4
2 (Midspan)	18"	4
2 (Support)	36"	4

3.Material Properties & Mix Design Information

Cement Type: Essroc Type II
Cement Content: 700 lb/yd^3
Water Content: 31.8 Gal/yd^3
W/C Ratio: 0.37

Air Content: 5.8%
Compressive Strength: 5622 Psi
Slump: 3.4”
Admixtures: Water Reducer & Retarder

4.Construction Information

Air Temperature: 69.3 F
Concrete Temperature: 79.0 F
Month Of Casting: June

5.Survey results

Crack Information:

		East & West Bound	
Span No.	**1**	**2**	**3**
Type	None	Transverse/Other	Transverse/Other
Location	-	All Over	All Over
Spacing	-	-	-
Size	-	-	-

Survey Team Member: Ala, Fred and Tom

6. Observations & Photos

-Lots of seals are on the deck.

Figure 117. A view of the bridge (Bridge No.1149-168).

Structure Number 1312-154

1.General Information

Bridge Location	RT35 Over NAVESINK River		
County	Monmouth	**Township**	Red Bank
Year Built	2000		

2.Structural Design Information

Number of Spans: 9
Direction of Traffic: Two ways
Girder type: Prestressed Concrete
Span type: Mixed (3 Continuous + 3 Continuous + 3 Continuous)
Type Of Bearing: Steel Bearings
Skewness: None

	North Bound & South Bound
Span Number	**All**
Span Width	89'-3"
Span Length	113'-5"

Framing Information:
- North Bound & South Bound:-All Spans: 10 Girders @7'-10"

Deck Slab Information:

Deck Thickness	$8^{1/2}$	**Top Cover**	$2^{1/2}$"
Wearing Surface	Concrete, Sawcut	**Bottom Cover**	$1^{1/2}$

Longitudinal Rebar Information:
-All Spans Midspan: #5@6"(B), #5@15"(T)
-Continuous Spans Over Pier: #5@6"(B), #5@15"+#8@15(T)
Transverse Rebar Information:
-All Spans EB: #6@6"(T&B)

Girder Information:

	North Bound & South Bound		
Span Number	**I (in^4)**	**A (in^2)**	**Depth (in)**
All	761550	1125	72

Shear Stud Information:

		Ramp
Span Number	**Spacing (in)**	**Number of Studs in Row**
All (Midspan)	6	6
All (Support)	23	6

3.Material Properties & Mix Design Information

Cement Type: Essroc Type II
Cement Content: 707 lb/yd^3
Water Content: 32.3 Gal/yd^3
W/C Ratio: 0.38
Air Content: 5.3 %
Compressive Strength: 6711 Psi
Slump: 3.1"
Admixtures: Water Reducer & Corrosion inhibitor

4.Construction Information

Air Temperature: 56 F
Concrete Temperature: 73 F
Month Of Casting: March-June

5.Survey results

Crack Information:

	Ramp
Span No.	**All Spans**
Type	Transverse
Location	All over
Spacing	3'-5'
Size	Hair Line

Survey Team Member: Tom, Anthony, and Jennifer

6. Observations & Photos

-Most of the cracks are on the northbound side, which has been built earlier