REPORT FHWA/NY/RR-94/161

Lightly Reinforced Concrete Bridge-Deck Slabs on Steel Stringers: A Summary of Field Experience

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RESEARCH REPORT 161

ENGINEERING RESEARCH AND DEVELOPMENT BUREAU
NEW YORK STATE DEPARTMENT OF TRANSPORTATION
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A. D. Emerich, Editor
Donna L. Noonan, Graphics and Production
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LIGHTLY REINFORCED CONCRETE BRIDGE-DECK SLABS ON STEEL STRINGERS: A SUMMARY OF FIELD EXPERIENCE

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Final Report on Research Project 142-2
Conducted in Cooperation With
The U. S. Department of Transportation
Federal Highway Administration

Research Report 161
June 1994

ENGINEERING RESEARCH AND DEVELOPMENT BUREAU
New York State Department of Transportation
State Campus, Albany, New York 12232

TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No. FHWA/NY/RR-94/161	2. PB99-100109	3. Recipient's Catalog No.
4. Title and Subtitle LIGHTLY REINFORCED CONCRETE BRIDGE DECK SLABS ON STEEL STRINGERS: A SUMMARY OF FIELD EXPERIENCE		5. Report Date June 1994
		6. Performing Organization Code
7. Author(s) Gongkang Fu, Sreenivas Alampalli, Frank P. Pezze III		8. Performing Organization Report No. Research Report 161
9. Performing Organization Name and Address Engineering Research and Development Bureau New York State Department of Transportation State Campus, Albany, New York 12232		10. Work Unit No.
		11. Contract or Grant No.
12. Sponsoring Agency Name and Address Offices of Research, Development and Technology HRD-10 Federal Highway Administration U.S. Department of Transportation Washington, DC 20590		13. Type of Report and Period Covered Final Report Research Project 142-2
		14. Sponsoring Agency Code

15. Supplementary Notes

Prepared in cooperation with the U.S. Department of Transportation, Federal Highway Administration. Study Title: Load Capacity of Concrete Bridge Decks.

16. Abstract

Experimental deck slabs containing light reinforcement (most with a reinforcement ratio of 0.24 percent) on 28 New York State bridges were evaluated in this study for long-term serviceability. Four were instrumented with strain gages and have been load-tested annually since construction, the longest life being 10 years. Thirteen relatively older bridges have also been inspected annually for the past 6 years, generally performing satisfactorily. Maximum stresses of bottom transverse rebars under 16-kip wheel loads over the years have always been below allowable levels, based on conservative analyses. Their behavior and performance are also compared here with regular deck slabs designed according to current AASHTO specifications. Maximum transverse-rebar stresses in both lightly reinforced and AASHTO deck slabs increased noticeably for their first year or two, but remained relatively constant thereafter. Transverse cracking on the top surface was similar for both types of deck slab. Although longitudinal cracking was minor, it appeared more severe on the lightly reinforced slabs than on the corresponding AASHTO slabs, and this was attributed to possible overloading. Thus, a slightly higher reinforcement ratio (closer to 0.3 percent) is recommended to provide a wider safety margin, especially for cracking under possible overloading.

17. Key Words Bridge decks, light reinforcement, load testing, cracking, reinforcement ratio	18. Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161.

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	v + 41	

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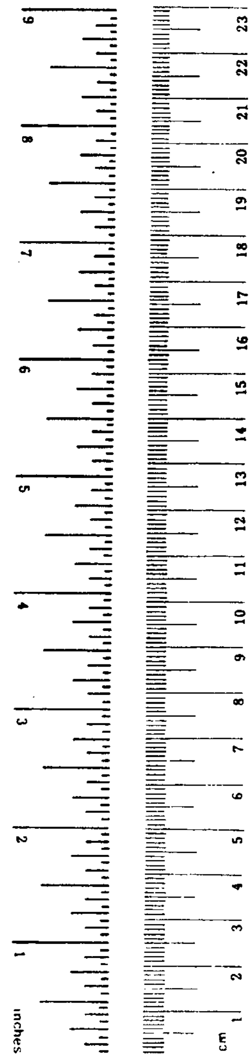
METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures

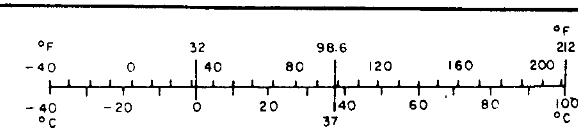
Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.



Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



CONTENTS

I. INTRODUCTION

A. Background

In the United States, reinforced-concrete (RC) bridge deck slabs are designed based on flexural failure mode, according to the current AASHTO design code (*1*). However, extensive research has shown punching shear to be the dominant failure mode for deck slabs, attributed to flexural strength enhancement by membrane compressive force in the slab, induced by its transverse boundary constraints. This is referred to as "arching action" or the "dome effect." Based on research findings (*2*,*3*,*4*,*5*), the Ontario Highway Bridge Design Code (*6*) has incorporated an empirical design with isotropically reinforced slabs having a minimum reinforcement ratio of 0.3 percent in each face. This requires significantly less flexural steel than the AASHTO code. Attracted by the possibility of reduced construction cost and a lower probability of rebar corrosion and concrete spalling, researchers and state agencies in the United States have devoted substantial effort to this subject over the past decade. Several states have either constructed bridges with isotropic deck slabs or plan to build them on an experimental basis. The study reported here is part of continuing New York State efforts to evaluate lightly reinforced concrete slabs for highway bridges subjected to current wheel loads.

B. Review of Previous Work

Increase in slab capacity due to arching action was noted as early as 1909 (*7*). Most early research in this area was oriented toward building floor applications, and has been briefly reviewed by several authors (*8*,*9*,*10*,*11*,*12*,*13*). Behavior and strength of RC bridge deck slabs under static load are obviously of particular interest to bridge designers and owners. Modern studies in this area began at Queen's University in Ontario, Canada (*2*,*3*,*5*,*14*), and demonstrated excess reserve strength in 1/8-scale conventional RC deck slabs designed in accord with AASHTO provisions (*1*), as well as a dominant punching failure mechanism in both isotropic and AASHTO orthotropic deck slabs. After examining isotropic slabs having various reinforcement ratios, the researchers recommended an isotropic slab design with a minimum reinforcement ratio of 0.2 percent as possessing an adequate safety factor. Their findings were later confirmed by full-scale bridge testing (*15*,*16*), with 0.3-percent reinforcement adopted for the Ontario code (*6*). In the United States, Beal (*17*,*18*) confirmed these earlier findings by testing 1/6-scale and full-scale bridge deck slabs having isotropic and AASHTO orthotropic reinforcement. He also concluded that rebar stresses under the AASHTO design wheel load (20.8 kips) were lower than predicted by the AASHTO code, and that ultimate strengths were six times larger than the design load in 0.25-percent reinforced isotropic deck slab models. Fang et al. (*8*,*9*) tested full-scale isotropic deck models with 0.4-percent reinforcement in each layer under simulated vehicle wheel loads. They found significant compressive forces present after cracking of the deck slab under load, with the deck slab behaving linearly up to a wheel load three times the AASHTO design load.

Most recently, Perdikaris and Beim (*10,11*) also confirmed adequate safety factors for isotropic deck slabs by testing 1/6.6- and 1/3-scale deck slab models with 0.3-percent reinforcement, as well as their failure by punching shear. Later, Puckett et al. (*19*) tested two full-scale deck slabs under vehicular loads at numerous deck locations -- one reinforced according to the AASHTO code and the other with 0.3-percent isotropically according to the Ontario code (*6*). They found that bottom transverse rebars between girders experienced highest stresses under these loads. Jackson and Cope (*13*) tested two half-scale models of isotropic slabs to examine the global-load effect under wheel loads simulating critical vehicle loading cases. One bridge deck slab had about 20-percent more reinforcement than required by the Ontario code, and the other was lighter. They found that empirical design approaches for isotropic reinforcement appeared to be satisfactory, although global transverse moments could have large effects on deck slab behavior at various load levels.

In addition to static strength, fatigue strength of RC slabs has also been studied by several researchers. Batchelor et al. (*4,20*) fatigue-tested 1/8-scale isotropic and orthotropic bridge-deck-slab models under a sinusoidal concentrated load, and found that fatigue failure was consistently by punching shear. Based on their test results, they recommended an endurance limit of 0.4 (a fatigue load factor of 2.5) for an isotropic slab design with 0.2-percent reinforcement.

Using fixed pulsating loads and stepwise moving loads, Okada et al. (*21*) and Sonoda and Horikawa (*22*) tested 1) full-scale models and panels sawed from distressed bridge deck slabs having orthotropic reinforcement, and 2) 1/3-scale isotropically reinforced deck slab models with 1.32-percent (top and bottom) reinforcement. They found that moving load is substantially more damaging than fixed pulsating load with respect to flexural and shear resistance of the slab.

Fang et al. (*8,9*) tested a full-scale isotropic deck slab model having about 0.4-percent reinforcement under fixed pulsating loads of 26 kips. They concluded that 5 million cycles of this load did not deteriorate the deck significantly. Perdikaris and Beim (*10,11*) used constant rolling wheel loads in their fatigue tests of 1/6-scale isotropic deck slab models with 0.3-percent reinforcement. Their constant rolling wheel load resulted in a gridlike crack pattern on the bottom surface of the models. This has often been observed in bridge decks in service (*10,11,21*). They also found that the constant rolling wheel load simulating real traffic loading is much more deteriorating than a fixed pulsating wheel load. They concluded that their isotropic deck slabs possessed higher ductility and fatigue strength under constant rolling wheel load than the AASHTO orthotropic deck slabs.

Agarwal (*23*) reported a study that included comparative testing of 14 Ontario isotropic and AASHTO bridge decks built in the early 1980s. He found that after several years in service, slab panels displayed no visible signs of distress under wheel loads far exceeding factored loads specified by the codes (*1,6*). He further concluded that the major parameters affecting strength and stiffness of these slabs are slab thickness, girder type, and girder spacing.

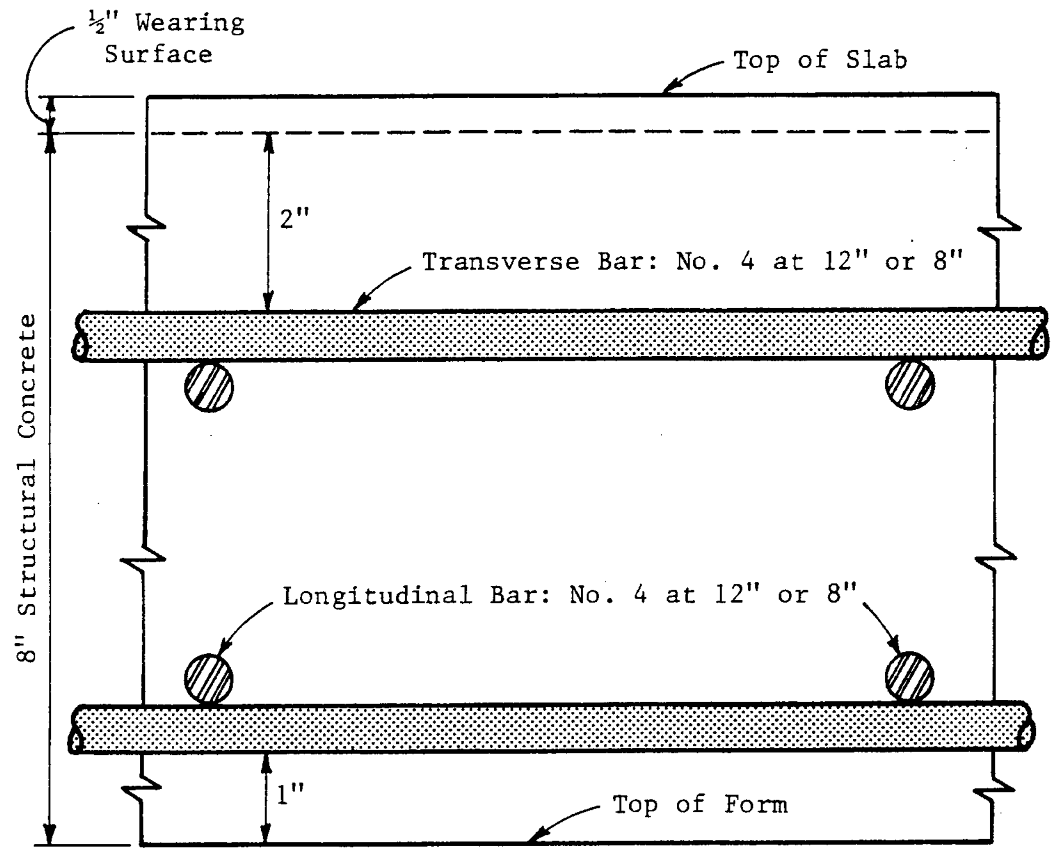
C. Research Objectives

These results have shown that empirically designed isotropic deck slabs containing less steel possess adequate strengths, higher than conventionally predicted. On the other hand, their long-term serviceability needs to be examined before general application. Service fatigue conditions of RC bridge deck slabs are generally much more severe than the laboratory environment where most of these experimental studies were conducted. This is caused by environmental temperature fluctuation, application of deicing chemicals, freeze-thaw cycling, damage cumulation due to interaction of these deteriorating factors, etc. It is extremely difficult, if not impossible, to simulate these influencing factors in a laboratory. The present study was designed to examine long-term serviceability of full-scale, lightly reinforced bridge deck slabs in New York State. It was intended to evaluate them based on their continuous in-service performance and behavior.

D. Organization of this Report

This report has five more chapters. Chapter II describes the scope of study, including design details for the lightly reinforced bridge deck slabs studied, and the definition of serviceability used for their evaluation. Chapter II also provides details of load testing and associated instrumentation for four of the investigated slabs containing both light reinforcement and conventional reinforcement according to the current AASHTO code. Chapter III identifies the critical load paths, describes the behavior of rebar stresses, reports critical rebar stress histories throughout service life, and compares maximum rebar stresses with allowable levels to evaluate serviceability. Histories of maximum rebar stresses are also compared between the lightly reinforced and AASHTO-reinforced bridge deck slabs. Chapter IV reports a study on cracking behavior of both lightly reinforced and AASHTO-reinforced deck slabs, as well as comparing them. Chapter V summarizes the findings and presents recommendations.

Figure 1. Cross-section of lightly reinforced deck slab.



II. TEST STRUCTURES AND INVESTIGATION

A. Design of Lightly Reinforced Bridge Deck Slabs in New York

After verifying previous research results regarding strength of isotropic deck slabs, New York State started to experiment with an empirical light reinforcement design (*18*). The cross-section is shown in Figure 1, with two rebar spacings (8 or 12 in. at centers) and reinforcement ratios (steel to effective cross-section areas) of 0.36 and 0.24 percent, respectively. Because of this isotropic arrangement, the experimental deck slabs are referred to here as "isotropic" decks. Note, however, that the Ontario code (*6*) requires an isotropic reinforcement of at least 0.3 percent.

To take advantage of potential mass production of the reinforcement mat, the following provisions (*18*) applied: 1) a maximum stringer spacing of 10 ft (i.e., a maximum ratio of girder spacing to slab thickness of 14 to 1) with no less than four stringers, 2) Grade 60 steel with the top-mat epoxy-coated, 3) Class E structural concrete (water cement ratio 0.44, air content 6.5 percent, slump 3 to 4 in., sand 35.8 percent, and cement 648 lb/cy or its equivalent, for pumping) according to then current New York State construction specifications (*24*), 4) longitudinal bars parallel to stringers and transverse bars parallel to the skew angle up to 30 deg, with bar spacing reduced by the cosine squared of the skew angle, 5) additional reinforcement for fascia overhang and negative moment areas according to the AASHTO specifications (*1*), and 6) permissible metal stay-in-place forms according to the New York State's current practice.

B. Serviceability of Bridge Deck Slabs

"Serviceability" of RC structural components refers to various aspects of their behavior and performance under service conditions, such as deflection and stress level induced by service load, cracking behavior, and surface condition (as affected by concrete cracking, spalling, and delamination as indices of durability). In general, stress levels provide information directly on distress condition and indirectly on available fatigue strength. Surface condition relates to concern over rebar corrosion mainly due to deicing chemicals. In this study, rebar stresses under vehicular service load and surface condition of bridge deck slabs were chosen for serviceability evaluation. Deflection was not used as a criterion here because points of interest are not always accessible without special arrangements.

Table 1. Summary of inspected bridges.

ID	Location	Region & County[c]	BIN	Contract Number	AADT[d]	Daily Truck Volume	% Trucks[e]	Traffic Flow	Slope, ft/100 ft[f]	Girder Spacing	Web Depth, in.	Bearing Type[h]
A. ISOTROPIC DECKS												
I-1	Bay View Rd/I-590[a,b]	43	1051290	D96607	--	--	6.1	E,W	3.2	8'9"	36	4,52
I-2	Rte 7/Elm St[a,b]	11	1072469	D96841	42800	1300	6.1	E	-1.92	10'0"	76	3,52
I-3	Rte 20/Chautauqua Cr[a]	52	1015370	D500350	2800	480	17.2	E,W	1.26	9'10"	61	1,8,60
I-4	HRP SB/Mamaroneck Riv	87	5523371	D250952	50000	0	0.0	S	+2.5	8'3"[g]	86	8,60
I-5	HRP NB/Mamaroneck Riv	87	5523372	D250952	50000	0	0.0	N	-2.55	8'0"	84	8,60
I-6	Rte 3/Oswegatchie Riv	74	1000640	D251507	2700	190	7.0	N,S	1.3	10'0"	92	8,60
I-7	Rte 104 EB/Hard Rd	43	1073452	D500066	37000	1130	6.1	E	+1.6,-2.0	9'0"	65	8,60
I-8	Rte 104 WB/Hard Rd	43	1073451	D500066	37000	1130	6.1	W	+2.0,-1.6	9'0"	65	8,60
I-9	Rte 17/Rte 219	51	6600169	D500043	9900	1700	17.2	E,W	1.35,1.38	8'3"	53	8,60
I-10	US 62/Erie Canal	53	4028510	D251565	21400	860	4.0	N,S	3.5	9'0"	46	8,60
I-11	Rte 446/Cuba Lake Outlet	61	1047750	D500258	1700	130	7.7	E,W	1.6	9'6"	48	1
I-12	Versailles Rd/Cattagaugus Cr	51	6064870	D251718	700	50	7.1	E,W	1.49	7'0"	52	8,60
I-13	Rte 11/Chateaugay Riv[a]	72	1035410	D500519	4500	470	10.5	E,W	0.5	9'0"	48	1,8,60
I-14	Rte 470/EB Mohawk Riv	11	2200470	D500586	12300	750	6.1	E,W	0.5	8'7"	52	1,8,60
I-15	Rte 32/Conrail	11	1022420	D252784	7550	460	6.1	E,W	2.5,2.8	9'4"	38	5,65
I-16	Rte 7/D&H RR	16	1004150	D252623	2950	210	7.0	E,W	1.3,7.0	9'4"	34	1,8,60
I-17	Rte 4/Champlain Canal	18	4001040	D500612	1250	80	6.1	N,S	0.45,4.9	8'3"	48	2,4,65
I-18	Shells Bush Rd/W Canada Cr	23	2204620	D252211	500	20	3.1	E,W	0.75	7'0"	42	1,8,60
I-19	Rte 31/Conrail	26	1022040	D252616	3150	170	5.4	E,W	4.47	9'1"	28	1,16,56
I-20	Horseshoe Island Rd/E Canal	33	4433050	D500514	500	20	3.1	N,S	6.2	6'8"	54	1,8,60
I-21	Rte 5/Little Canada Way	52	1001230	D252647	4500	480	10.7	N,S	0.0	8'9"	30	16,56
I-22	Rte 16/Cazenovia Cr, Holland	53	1011830	D500573	9700	690	7.1	E,W	3.44	9'1"[g]	44	8,60
I-23	Rte 16/Cazenovia Cr, Wales	53	1011870	D500630	9700	690	7.1	N,S	0.0	9'6"	56	65
I-24	Washington Ave/Conrail	62	2215670	D500493	7350	470	6.4	E,W	6.0	8'2"	36	1,16,56
I-25	Madison Ave/Chemung Riv	62	2215810	D252133	1900	450	3.8	N,S	5.0	9'0"	41	8,16,56,60
I-26	County Rd 541/Susquehanna	91	3367870	D253035	980	30	3.1	E,W	2.08,1.9	7'3"	50	65
I-27	S Grand St/Cobleskill Cr	95	3228510	D500690	1440	50	3.8	N,S	3.3	8'4"	48	16,65
I-28	Rte 97/Callicoon Cr	96	1035410	D251900	1250	100	7.7	N,S	4.0	9'3"	56	8,60
B. AASHTO DECKS												
A-1	Bay View Rd/I-590[a,b]	43	1051290	D96607	--	--	6.1	E,W	3.2	8'9"	36	4,52
A-2	Rte 7/Elm St[a,b]	11	1072469	D96841	42800	1300	6.1	E	-1.92	10'0"	76	3,52
A-3	W 3rd St/Chadakoin Riv	52	2258340	D500352	18500	1350	7.3	E,W	2.6,0.4	9'10"	90	1,3,52
A-4	HRP SB/Mamaroneck Ave	87	5523361	D250952	50000	0	0.0	S	+2.5	9'9"	46	4,53
A-5	HRP NB/Mamaroneck Ave	87	5523362	D250952	50000	0	0.0	N	-2.55	--	--	1
A-6	Rte 122/Trout Riv	72	3337920	D500282	1400	130	9.1	E,W	4.4	7'6"	27	65
A-7	Rte 104 EB/Holt Rd	43	1073462	D500066	36500	1110	6.1	E	1.6,-2.0	9'0"	62	8,60
A-8	Rte 104 WB/Holt Rd	43	1073461	D500066	36500	1110	6.1	W	+2.06,-1.6	9'0"	62	8,60
A-9	Rte 17/Allegheny Riv	51	6600179	D500043	10200	1750	17.2	E,W	0.4	9'7"	58	1,8,60
A-10	Forest Rd/Thruway	53	5511950	TAB87618	10000	380	3.8	N,S	4.0	8'6"	54	16,56
A-11	Rte 21/Canacadea Cr	61	1016300	D500105	1600	150	9.2	N,S	0.0	8'0"	82	8,60
A-12	Hasting Rd/Olean Cr	51	3322840	D500081	210	10	3.1	E,W	0.5,3.4	6'8"	32	4,51,52
A-13	Rte 3/Black Riv	73	1000580	D251698	6100	430	7.0	E,W	0.57	9'0"	70	1,8,60

NOTE: all structures are composite, with Grade 60 reinforcing steel, epoxy-coated top reinforcing mat, and class E or H structural concrete.
[a]Sites having instrumented rebars.
[b]Sites having AASHTO reinforcement (A-1,A-2), isotropic 12x12 reinforcement (I-1(12),I-2(12)), isotropic 8x8 reinforcement (I-1(8),I-2(8)).
[c]First digit is region, second is county.
[d]1990 Traffic Volume Report and Inventory and Inspection sheets (both directions).
[e]From Paul Polansky, Data Services Bureau.
[f]Unless noted, slope is both positive and negative.
[g]Spacing or width varies at ends of span, values reported are averages.
[h]Bearing Type Codes (from Reference 29):
For expansion bearings, 1 = none, 3 = steel rocker, 4 = steel sliding on phosphor bronze, 5 = steel sliding on steel, 8 = pot bearing.
For fixed bearings, 51 = none, 52 = steel rotating on a rocker, 53 = steel rotating on a pin, 56 = laminated elastomeric, 60 = multi-rotational (pot-bearing), 61 = multi-rotational (disc-bearing), 65 = other fixed.

Span Type	Total Spans	Span Length, ft[i]	Interior Diaphragm, in.	Diaphragm Spacing, ft	SIP Form	Skew, deg	Widths, ft: Outer to Outer	Curb to Curb	Travel Lane	Shoulder[l]	28-Day Concrete Strength, psi	Month/Year Poured	Month/Year Opened
A. ISOTROPIC DECKS													
Simple	4	59,35	15C33.9	19.67	No	14	40	30	12	3/3	5350	7/82	10/82
Simple	1	129	3x3x3/8	25	Yes[j]	50	55.4	52	12	10/18	4880	6/83	12/85
Continuous	7	1026	3x3x3/8	25	Yes	0	47.7	34	12	5/5	4250	9/87	11/87
Simple	1	145	3.5x3.5x3/8	Variable	Yes	Rad[k]	71.5[g]	68.4[g]	12	6.5/6	3940	7/85	1985
Simple	1	144	3.5x3.5x3/8	Variable	Yes	Rad[k]	53	49.5	12	8/6	5350	8/86	1986
Simple	1	176	5x5x3/8	22	Yes	12	47	35	12	8/3	4170	7/86	8/86
Simple	1	150	3x3x3/8	25	No	8	42	39	12	9.5/5.5	5290	9/85	11/85
Simple	1	150	3x3x3/8	25	No	8	41	38	12	9.5/5.5	5100[m]	9/85	11/85
Simple	1	140	3.5x3.5x3/8	23.33	Yes	9	81.1	76.6	12	10/4.25	6190	6/86	10/85
Continuous	3	320	3x2.5x3/8	21.7	Yes	0	70.6	68	12	8/4	4750/3780	9/86,7/87	9/87
Simple	1	103	3x3x5/16	20.7	Yes	20	44	40	12	8/8	4720	6/86	9/86
Continuous	3	400	3x3x3/8	22.8,24	Yes	0	34.8	28	10	4/4	4690	10/86	11/86
Continuous	7	754	3x3x3/8	21.2	Yes	0	42	40	12	8/8	3360	5/88	8/88
Continuous	3	296	3x3x3/8	21.7	Yes	0	40	28	12	2/2	4470	9/88	11/88
Simple & Cont	3	345	3x3x3/8	24	Yes	8	34.6	32	12	4/4	7210	9/90	12/90
Continuous	3	155	MC 18x42.7	16.7	Yes	40	34.6	32	12	4/4	3910	9/89	10/89
Continuous	3	323	3x3x3/8	24.8	Yes	45	40	38	11	8/8	4020	9/89	1/90
Continuous	3	342	WT 18x67.5	20.5	Yes	14	25	23	9	2.5/2.5	6530	8/88	11/88
Continuous	3	153	MC 13x31.8	13.7,23.7	Yes	20	42.6	39.4	12	8/8	5610	4/90	6/90
Continuous	3	332	3x3x3/8	23.1	Yes	0	24	22	9	2/2	4200	6/89	8/89
Simple	1	59	MC 18x42.7	19.7	Yes	15	42	40	12	8/8	4950	9/89	10/89
Simple	1	98	3x2.5x3/8	24.3	Yes	19	47.5	40	12	8/8	4060	9/88	9/88
Simple	1	115	3x3x3/8	23	Yes	22	46	44	12	10/10	4950	5/89	10/89
Continuous	3	181	MC 18x42.7	23.25	Yes	0	39.7	28	12	2/2	4750	5/88	6/88
Simple & Cont	9	548	MC 18x42.7	20.7,24.5	Yes	0	54	42	12	3/3	3470	6/88	9/88
Continuous	4	592	3x3x3/8	22.0,23.5	Yes	5	32	30	11	4/4	5300	9/90	10/90
Simple	1	135	3.5x3.5x3/8	22.5	Yes	10	32	30	11	4/4	4530	8/89	9/89
Simple & Cont	9	900	3x3x3/8	19.2,24.5	Yes	21	34.6	31.4	12	3.7/3.7	5500	9/87	11/87
B. AASHTO DECKS													
Simple	4	59,35	15C33.9	19.67	No	14	40	30	12	3/3	5350	7/82	10/82
Simple	1	129	3x3x3/8	25	Yes[j]	50	55.4	52	12	10/18	4880	6/83	12/85
Continuous	4	909	3.5x3.5x3/8	25	Yes	0,30	57.7	46	11	1/1	5300	9/87	11/87
Simple	1	89	3x3x3/8	22.25	Yes	10	64.7	60.5	12	6.5/6	4310	8/85	1985
Simple	1	98	--	--	No	10	59	49.6	12	6/6	--	8/86	1986
Simple	1	59	MC 18x42.7	19.5	Yes	0	36	34	11	6/6	6030	7/86	10/86
Simple	1	140	3.5x3.5x3/8	23.25	No	0	42	39	12	10/6	5010	8/85	10/85
Simple	1	140	3.5x3.5x3/8	23.25	No	0	42	39	12	10/6	5370	8/85	10/85
Continuous	5	752	3x3x3/8	20.3	Yes	39	89	83.5	12	10/8.5	4580	8/86	10/88
Continuous	2	260	3x3x3/8	22.5,25	Yes	22	40	28	12	2/2	--	1988	1988
Simple	1	152	3x3x3/8	23	Yes	45	36	34	11	6/6	3730	9/85	10/85
Continuous	2	224	MC 18x42.7	22.3	Yes	0	26	24	9	3/3	4870	10/85	11/85
Continuous	3	450	3x2.5x3/8	22.5	Yes	Rad[k]	42	40	12	8/8	4770	5/87	7/87

[i] Span length is from center to center of bearings.
[j] 20 percent of the stay-in-place (SIP) form was removed at Site I-2(12), 17 percent at Site I-2(8), 34 percent at Site A-2.
[k] Skew is zero but bridge is on a slight curve.
[l] If traffic flow is in one direction, the first value is for the driving lane, and the second for the passing lane.
[m] 21-day strength.

C. Overview of the Investigated Bridge Deck Slabs

Since 1982, a total of 29 isotropic decks have been constructed in New York State on bridges having multiple steel stringers with diaphragms or cross-bracings. Of these, 28 were included in the present study, designated as Sites I-1 through I-28 in Figure 2, which indicates their locations (the 29th had not been opened to traffic). All these decks contain rebars with 12-in. spacings except two bridges, I-1 and I-2. They have three reinforcement patterns for comparison: isotropic spacings and orthotropic according to AASHTO (*1*). Note that there are two bridges (sites) each at locations on the Hudson River Parkway (Sites I-4 and I-5) and on Rte 104 (Sites I-7 and I-8). More details of the experimental isotropic decks are given in Table 1A, including year built, average daily truck traffic, structural features, etc. As indicated in Table 1A, 4 of the 28 bridge deck slabs are instrumented with electrical-resistance strain gages on rebars. These four instrumented decks were load-tested annually for rebar stress under service wheel loads, from their construction to the present or until failure of aging strain gages in the slabs. Two of the four instrumented deck slabs also have sections designed according to AASHTO code (*1*), designated as Sites A-1 and A-2 in Table 1B and Figure 2. They provided comparison data for rebar stress behavior in the two types of reinforcement. (This is covered in the next section and in Chapter III.)

From 1986, when the oldest experimental deck had been in service for 4 years, the first 13 experimental decks were inspected annually to examine and record surface condition with respect to serviceability. In 1991, 13 regular bridge deck slabs designed according to the current AASHTO code (*1*) were added to the inspection program for comparison with the first 13 experimental decks. Designated as Sites A-1 to A-13, their locations are shown in Figure 2 and Table 1B gives additional details. In 1991, the rest of the experimental decks (I-14 to I-28) were also included in the inspection program, for more complete coverage. Their locations and details are shown in Figure 2 and Table 1A. Inspection results are included in Chapter IV.

D. Instrumentation and Load Testing Details

1. Types of Load Test

Three types of load were applied to obtain rebar strain/stress under the AASHTO wheel load: 1) a single concentrated load distributed over an 8- by 20-in. plate by jacking a truck's rear axle (referred to here as the simulated wheel load test), 2) a stationary vehicular wheel load applied at various locations longitudinally across the bridge (referred to as the static influence line test), and 3) a moving vehicular wheel load across the bridge at crawl speed (referred to as the dynamic influence line test). Wheel loads were applied along the centerline between two interior girders to produce maximum strain/stress in the instrumented rebars, because this loadpath is the most critical for rebar strain/stress (as will be shown later). Each loading was generally applied three times to eliminate possible instrumental error and accommodate unavoidable variations in vehicular loading. Table 2 lists the load tests performed on the instrumented slabs.

Figure 2. Bridge deck locations.

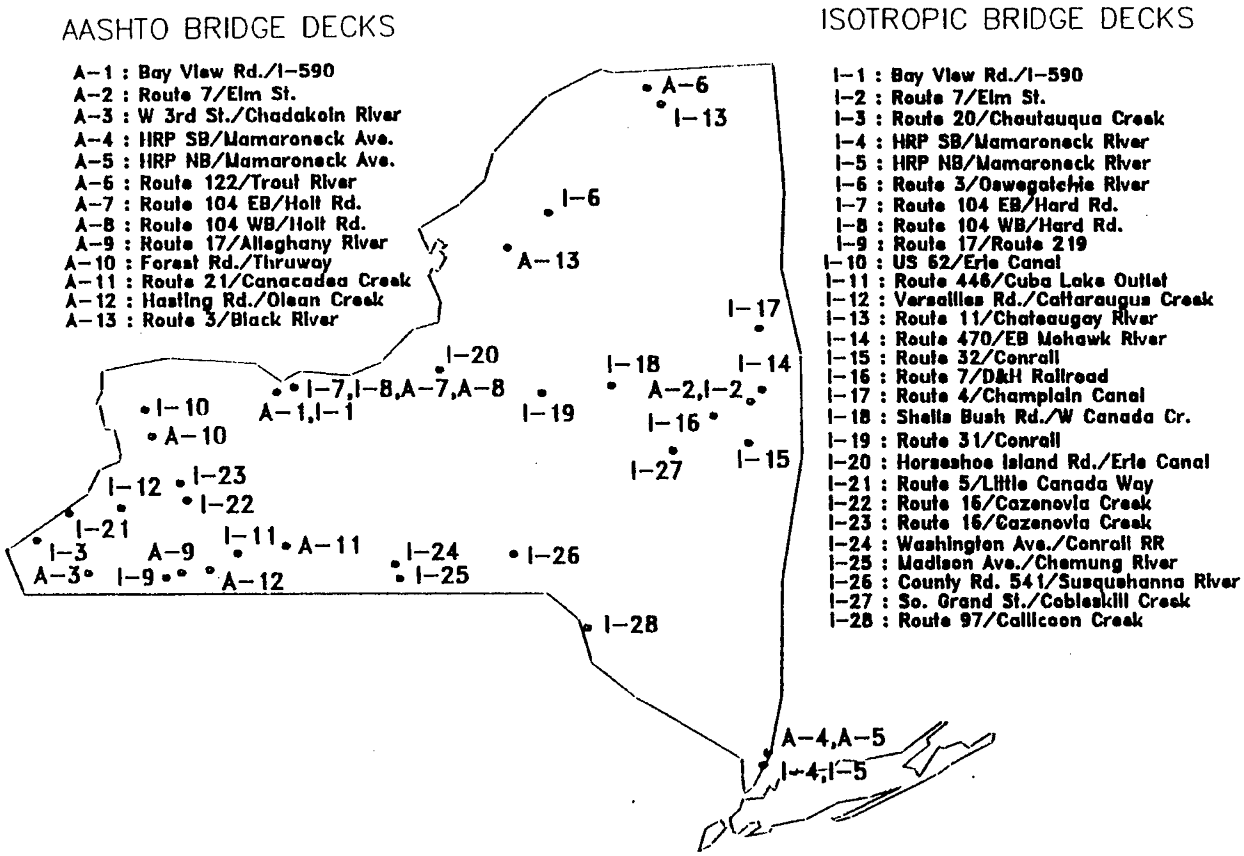


Table 2. Load tests performed 1982-92 on instrumented deck slabs*.

Year Tested	Sites A-1,I-1 (Built 1982)	Sites A-2,I-2 (Built 1983)	Site I-3 (Built 1987)	Site I-13 (Built 1988)
1982	Sim, Stat	--	--	--
1983	Sim, Stat	Sim, Stat	--	--
1984	Sim, Stat	Sim, Stat	--	--
1985	Sim, Stat	Sim, Stat	--	--
1986	Sim, Stat	Sim, Stat	--	--
1987	Failed	Sim, Stat	Dyn	--
1988	Failed	Stat, Dyn	Dyn	Dyn
1989	Failed	Stat, Dyn	Dyn	Dyn
1990	Failed	Stat, Dyn	Dyn	Dyn
1991	Failed	Stat, Dyn	Dyn	Dyn
1992	Failed	Stat, Dyn	Dyn	Dyn

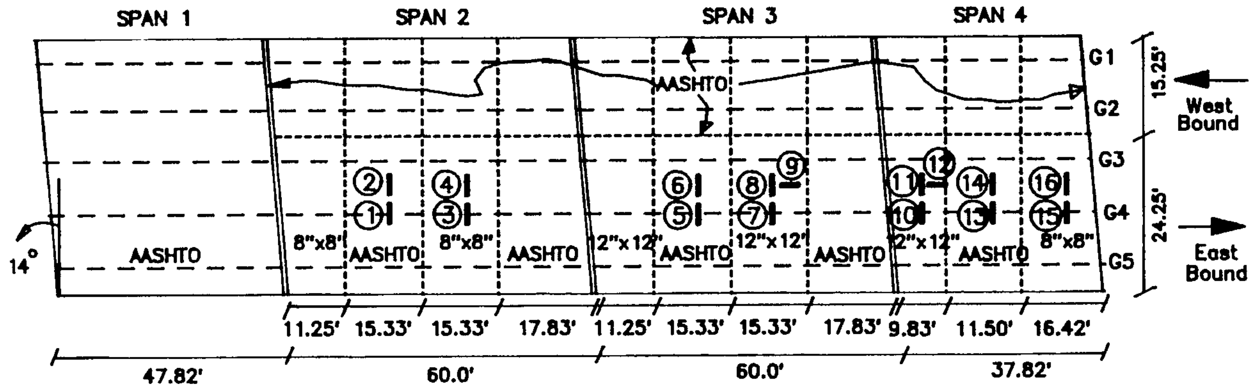
*Sim = simulated wheel load test
Stat = static influence-line test
Dyn = dynamic influence-line test
Failed = not tested due to failure of aging strain gages.

The simulated wheel load test was discontinued in 1987, because its results were regarded as having little value with respect to service load effects, considering the load's unrealistic distribution area and magnitude. Rebar strain readings in the static influence line test were recorded by a static data-acquisition system having 99 channels, while those in the dynamic influence line test used a dynamic data-acquisition system recording on eight channels at 25 samples/sec. The dynamic influence line test was used to reduce test time, compared to the static test, and these two tests produced consistent results.

2. Instrumentation

Figures 3 through 6 show instrumentation details for the four instrumented experimental deck slabs, including strain gage types and locations which were selected to monitor rebar strain/stress in critical areas. For behavior comparison, Sites 1 and 2 have both AASHTO orthotropic (A) and the two isotropic (I) reinforcement arrangements -- 8-in. grid (8) and 12-in. grid (12). They are identified as Sites A-1, I-1(8), and I-1(12), and A-2, I-2(8), and I-2(12) in Table 1. Figure 7 shows details of the instrumented rebars, which were 4 to 6 ft long, including 15 in. at each end for overlapping with regular rebars.

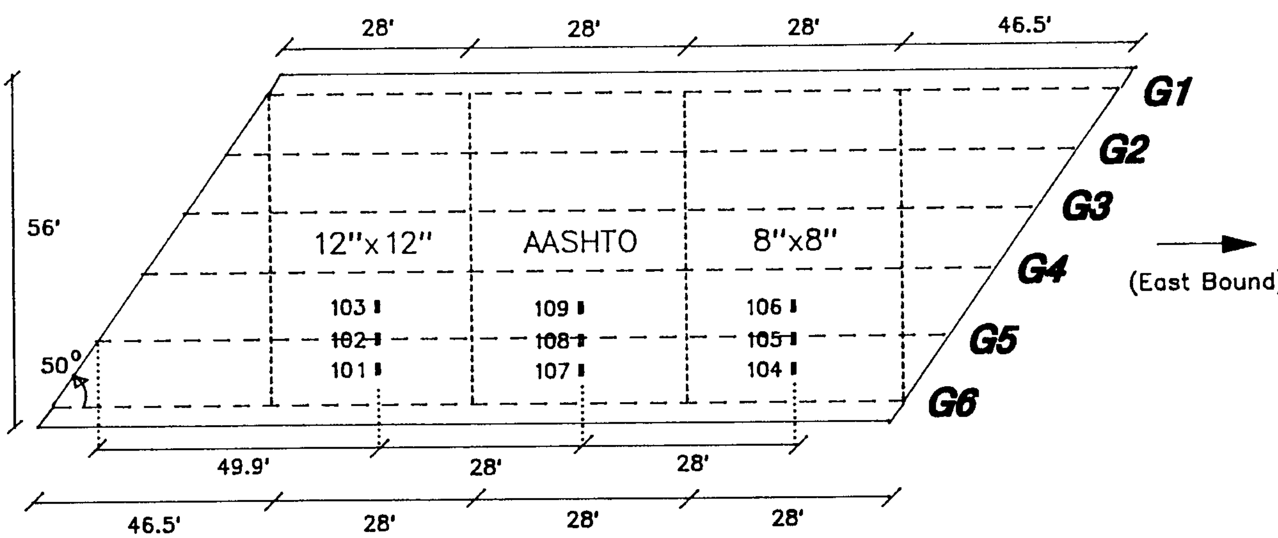
Figure 3. Gage locations at Sites A-1, I-1(12), and I-1(8): Rte 7 (Bayview Rd.).



NOTES:

1. All spans are simply supported.
2. ▬ = gage locations.
3. 16 four-arm strain gages of self-temperature compensating type for rebar uniaxial strain. Top transverse bars over girder G4: Nos. 1, 3, 5, 7, 10, 13, 15.
   Bottom transverse bars at center of interior bay: Nos. 2, 4, 6, 8, 11, 14, 16.
   Bottom longitudinal bars at center of interior bay: Nos. 9, 12.

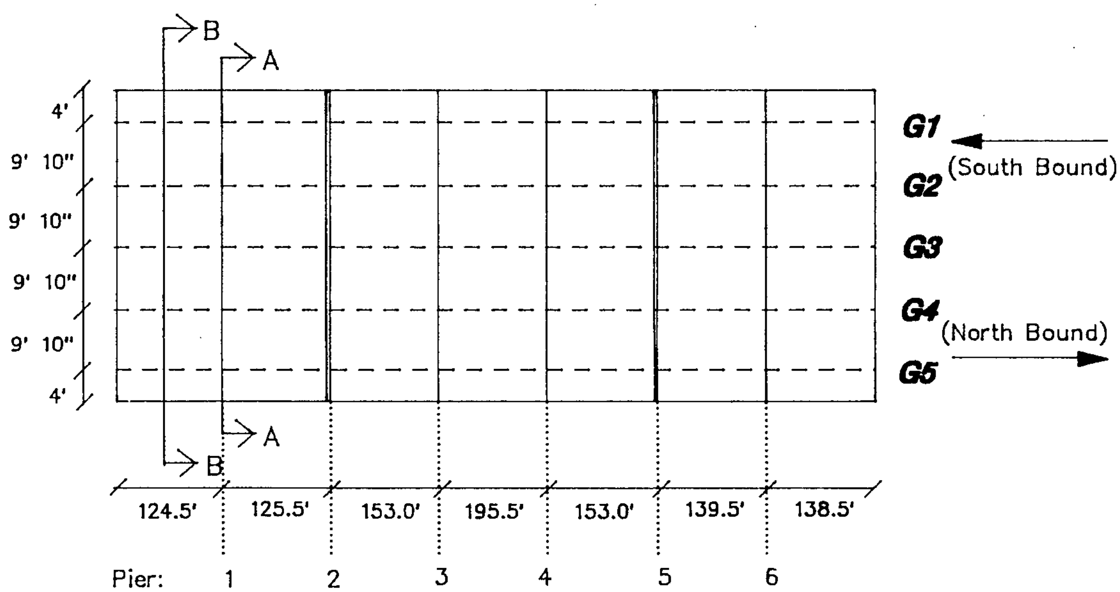
Figure 4. Gage locations at Sites A-2, I-1(12), and I-2(8): Rte 7 (Bayview Rd).



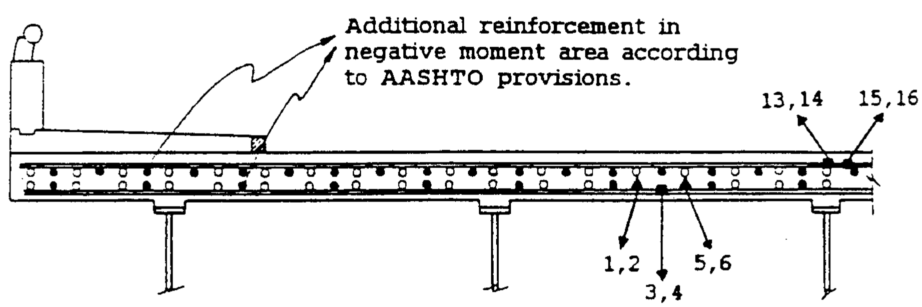
NOTES:

1. The span is simply supported.
2. ▮ = gage locations.
3. 9 four-arm strain gages of self-temperature compensating type for rebar uniaxial strain. Bottom transverse bars at center of fascia bay: Nos. 101, 104, 107.
   Top transverse bars over girder G5: Nos. 102, 105, 108.
   Bottom transverse bars at center of interior bay: Nos. 103, 106, 109.

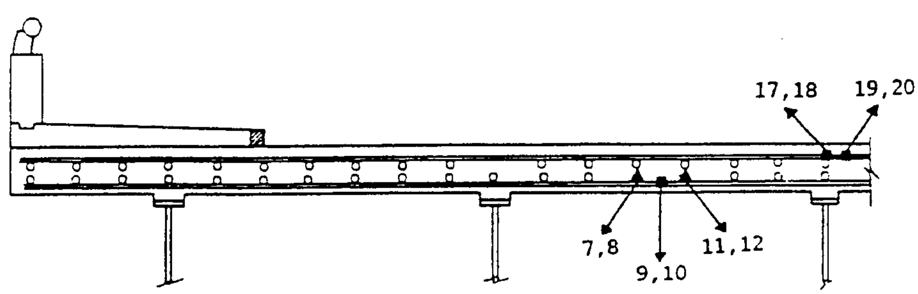
Figure 5. Gage locations at Site I-3: Rte 20.



OVER PIER A-A



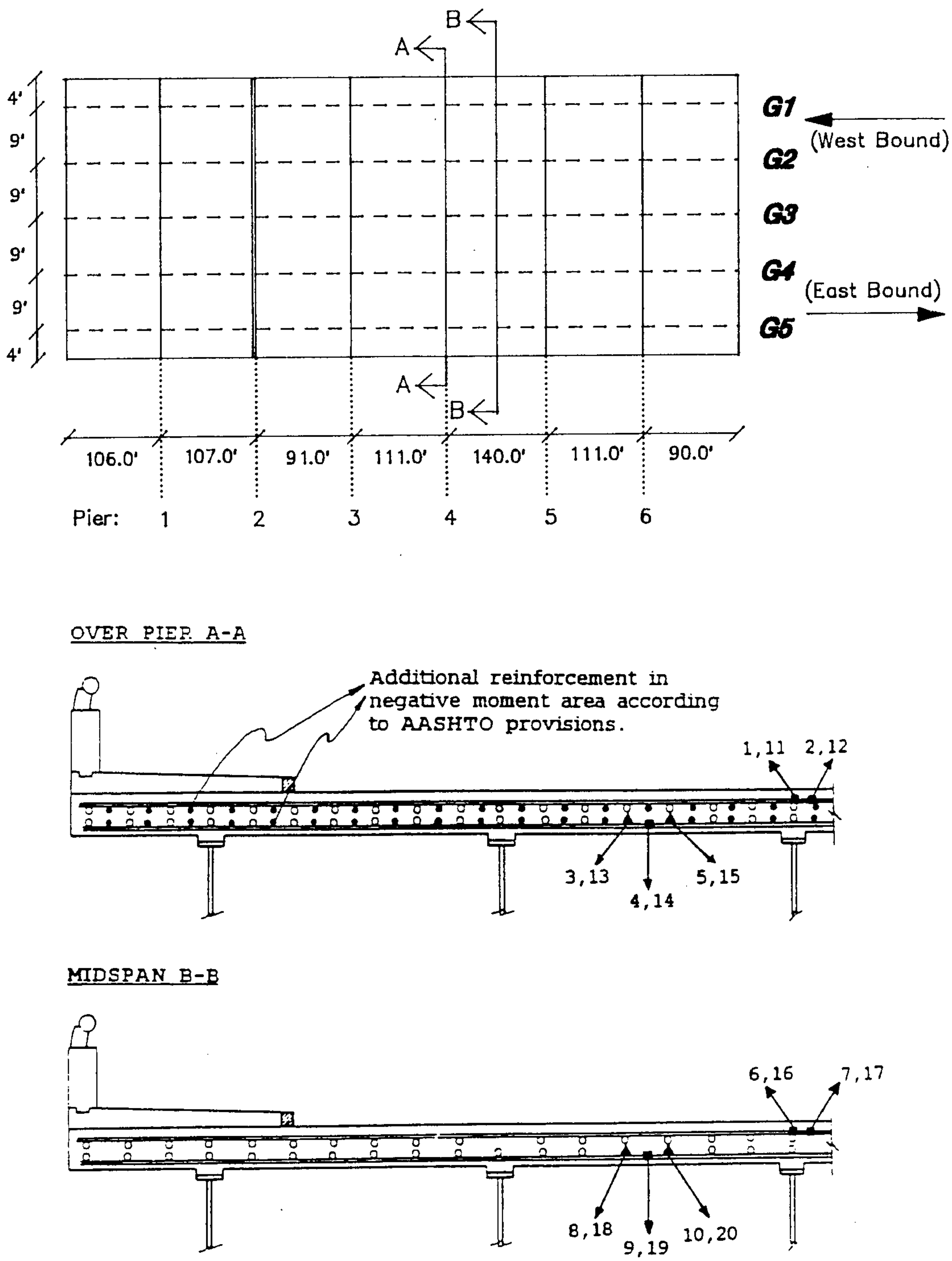
MIDSPAN B-B



Notes:

■ Gage on Transverse Rebar
▲ Gage on Longitudinal Rebar
Total of 20 weldable single-arm strain gages of self-temperature compensating type. At each location, two gages on diametrically opposite sides of a rebar for uniaxial strain.

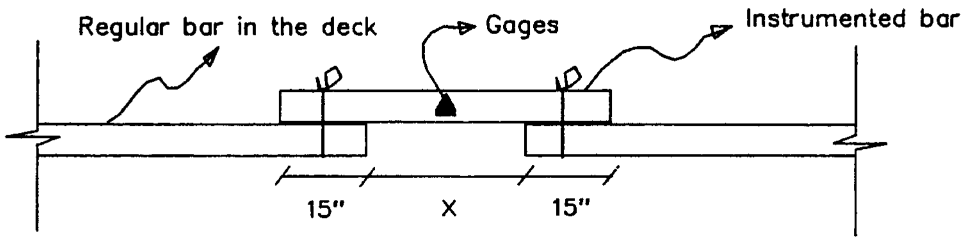
Figure 6. Gage locations at Site I-13: Rte 11.



Notes:

■ Gage on Transverse Rebar
▲ Gage on Longitudinal Rebar
Total of 20 weldable single-arm strain gages of self-temperature compensating type. At each location, two gages on diametrically opposite sides of a rebar for uniaxial strain.

Figure 7. Typical gage installations.



Description:	Length x
1. Sites I-1, A-1 (Bayview Road)	18"
2. Sites I-2, A-2 (Rte_7)	18"
3. Site I-3 (Westfield – between girders)	18"
4. Site I-3 (Westfield – over girders)	25.5"
5. Site I-13 (Chateaugay – between girders)	18"
6. Site I-13 (Chateaugay – over girders)	42"

III. BEHAVIOR OF REBAR STRESS UNDER SERVICE VEHICULAR LOADS

A. Critical Rebar Stresses and Associated Loadpaths

Various loadpaths were used in load testing to obtain the distribution of critical rebar strains/stresses. Figure 8 shows several loadpaths used in 1992 for three of the four instrumented deck slabs, when the fourth had no gages in working condition. A two-axle truck was used at each site for loading, with gross weight varying from 44.4 to 53.6 kips. Axle spacing was from 15 to 15.3 ft, and the rear axle weighed 30.9 to 38.9 kips. Table 3 compares maximum rebar stresses under these loadpaths, linearly adjusted to 16 kips of rear wheel load. It shows that the path with a wheel at the center of two girders (Path 2 in Table 3 and Figure 8) was the most critical with respect to maximum stresses in both transverse and longitudinal rebars at the center. The wheelpath straddling a girder (Path 3 in Table 3 and Figure 8) was critical for transverse rebars over a girder. However, their maximum stresses were much lower than those of transverse rebars at the center of two girders under Path 2 -- Path 3 was thus concluded to be less critical than Path 2.

B. Global and Local Effects in Critical Rebar Stresses

Figure 9 shows typical rebar stress influence lines, obtained in a dynamic influence line load test at Site I-3 (Rte 20) in 1990. These curves show stresses of rebars at midspan and pier sections (Fig. 5 shows more details of gage locations). In this test, a two-axle truck was used to apply load, having front and rear axles weighing 15 and 30 kips, respectively, with a 15-ft longitudinal spacing. The truck was driven on the southbound side of the bridge from Pier 2 to the south abutment at a speed of about 10 mph. This portion of the bridge is continuous over Pier 1 (Fig. 5). Stresses obtained under the load were then linearly scaled to a vehicular load having 16-kip rear wheels. The abscissa in each figure is the distance from the front axle to the starting point near Pier 2.

Figure 9 shows that dynamic effects of the moving vehicle were minimized by the crawl speed. It also shows that rebar stresses include global and local contributions. The local contribution is described by two sharp peaks induced by the front and rear wheels, successively. The global contribution is demonstrated by curves of relatively lower slopes before and after the adjacent sharp peaks, which describe the deck slab's participation in load-carrying as part of the bridge's cross-section. In these two figures, the local effect on the maximum rebar stresses is greater than the global effect.

Figure 8. Loading paths.

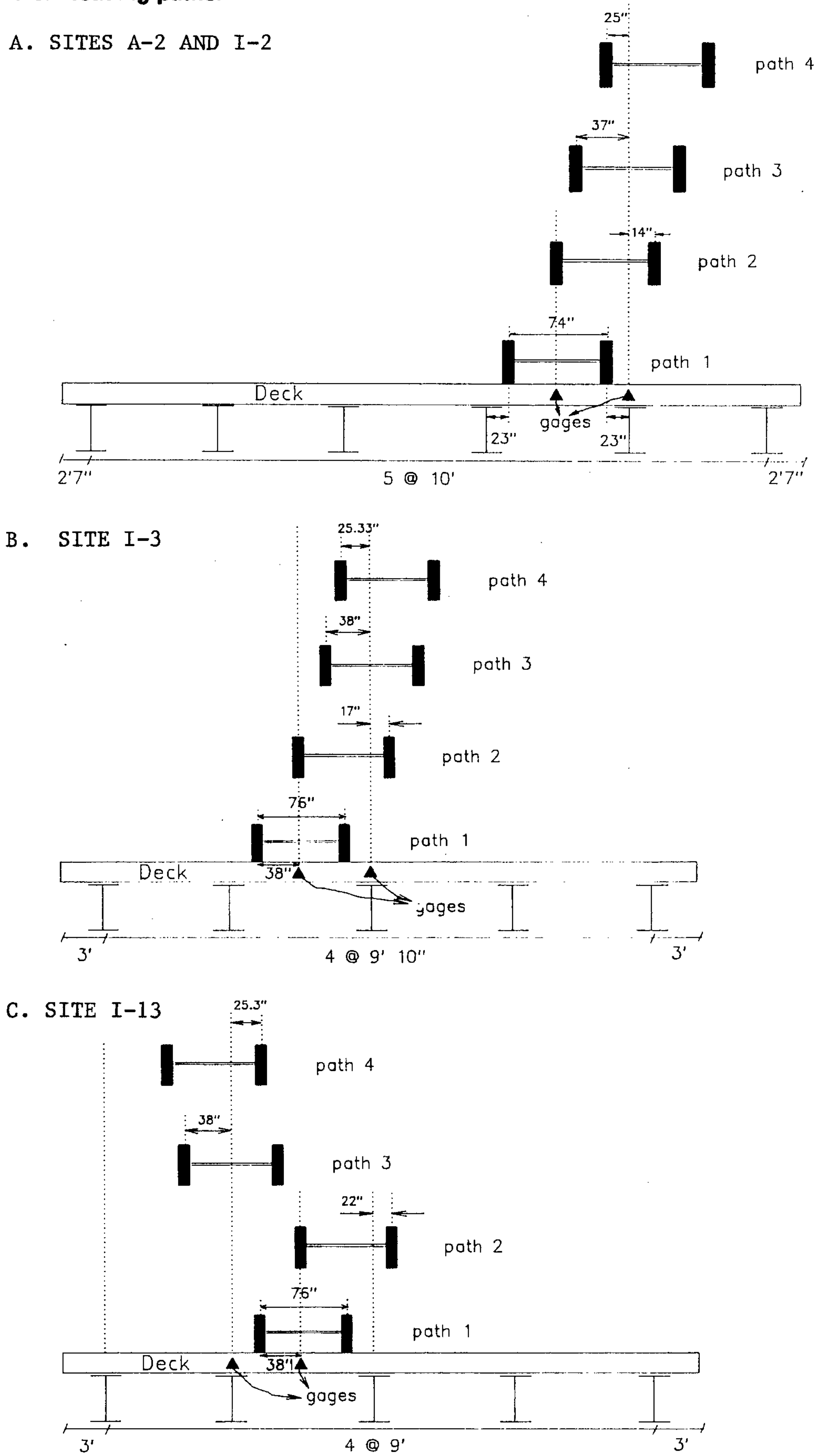


Table 3. Rebar stresses under various load paths (1992 tests).

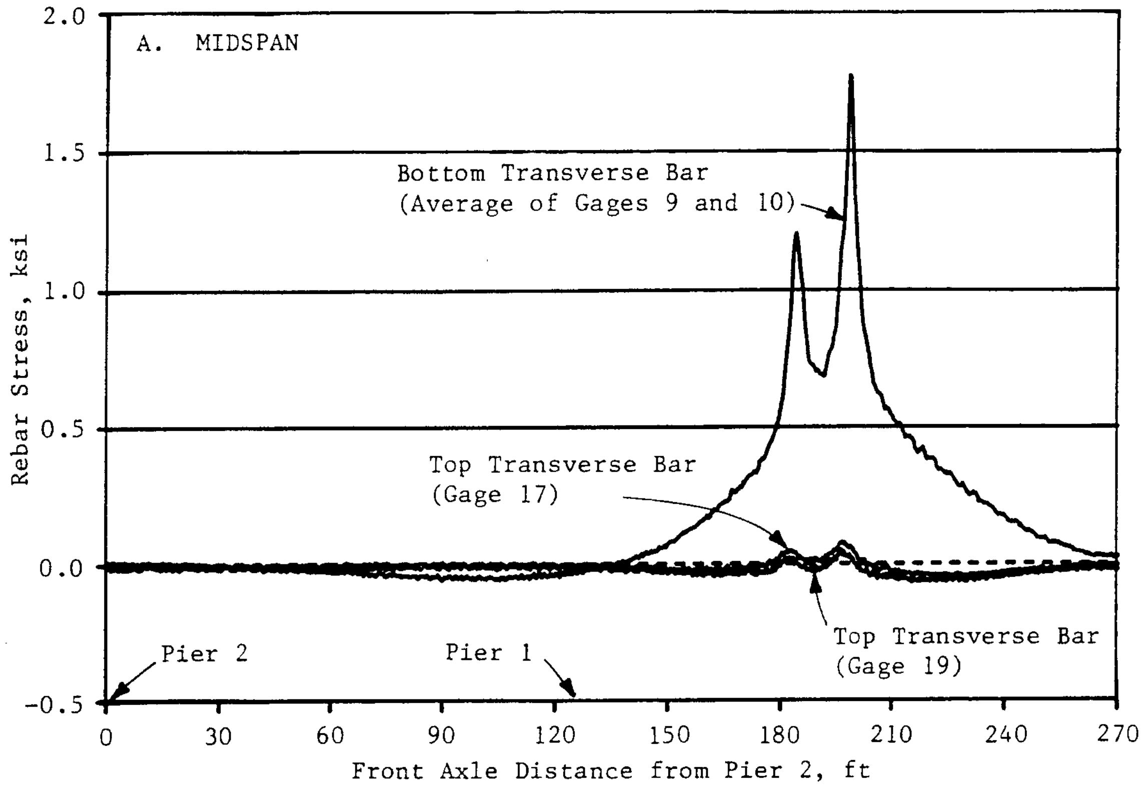
Locations	Stress, ksi Path 1	Path 2	Path 3	Path 4
Sites I-2(12),A-2[a]				
Gage 103 (12x12)	3.27	9.89[e]	2.90	0.61
Gage 109 (AASHTO)	1.25	2.97[e]	1.06	0.32
Site I-3[b]				
Gages 3 and 4	0.60	2.30[e]	0.26	0.51
Gages 5 and 6	1.61	3.08[e]	0.60	1.23
Gages 9 and 10	1.57	3.01[e]	0.94	1.47
Gages 17 and 18	0.14	0.07	0.22[f]	0.22
Site I-13[c]				
Gages 1 and 11	0.20	0.34	0.35[f]	0.26
Gages 2 and 12	0.13	0.31	0.35[f]	0.31
Gages 3 and 13	1.05	1.96[e]	1.80	1.06
Gages 4 and 14	0.15	0.87[e]	0.65	0.20
Gages 6 and 16	0.04	0.22	0.28[f]	0.17
Gages 7 and 17	0.04	0.12	0.16[f]	0.13
Gages 8 and 18	ERR[d]	2.23[e]	1.86	ERR[d]
Gages 9 and 19	0.77	1.92[e]	1.03	0.66

[a]See Fig. 4 for gage configurations and locations.
[b]See Fig. 5 for gage configurations and locations.
[c]See Fig. 6 for gage configurations and locations.
[d]Instrumentation error.
[e]Transverse and longitudinal rebars at center experiencing maximum stress under Load Path 2.
[f]Transverse rebars over a girder experiencing maximum stress under Load Path 3.

In Figure 9A, stress of the bottom transverse bars (average of Gages 9 and 10) at the midspan section was contributed mainly by the global stress, except between distances of 170 to 210 ft. When the load vehicle was on the adjacent span (between Piers 2 and 1), the bottom transverse bar at midspan was subjected to very low negative (compressive) stresses. This stress became positive (tensile) when the vehicle was on the gaged span (between Pier 1 and the south abutment). Figure 9A also shows that the top transverse bars over an interior girder (Gages 17 and 19) had much lower stresses than the bottom transverse bars, and yet experienced combined global and local effects. Since Gages 7, 8, 11, 12, and 20 failed due to age and Gage 18 was not functioning properly, rebar stresses of those gages are not shown. Previous results showed that their maximum was as negligibly low as about 0.64 ksi (*25*).

Figure 9B shows similar superposition of global and local effects of vehicular wheel loads at the Pier 1 section indicated in Figure 5. It can be seen that longitudinal bar stresses (Gages 1, 5, and 6) were more localized than those in the transverse bars (Gages 3 and 4), as indicated by sharper peaks. It can also be observed in Figure 9 that the bottom longitudinal bars experienced higher stress than the bottom transverse bar. This was caused by presence of a transverse crack on the top surface at the pier section. Without this crack, previous stress data showed that the opposite was true (*25*). Nevertheless, the highest stress shown in Figure 9 is less than 3 ksi, apparently far below an

Figure 9. Site I-3 influence lines under vehicular 16-kip rear wheel load.



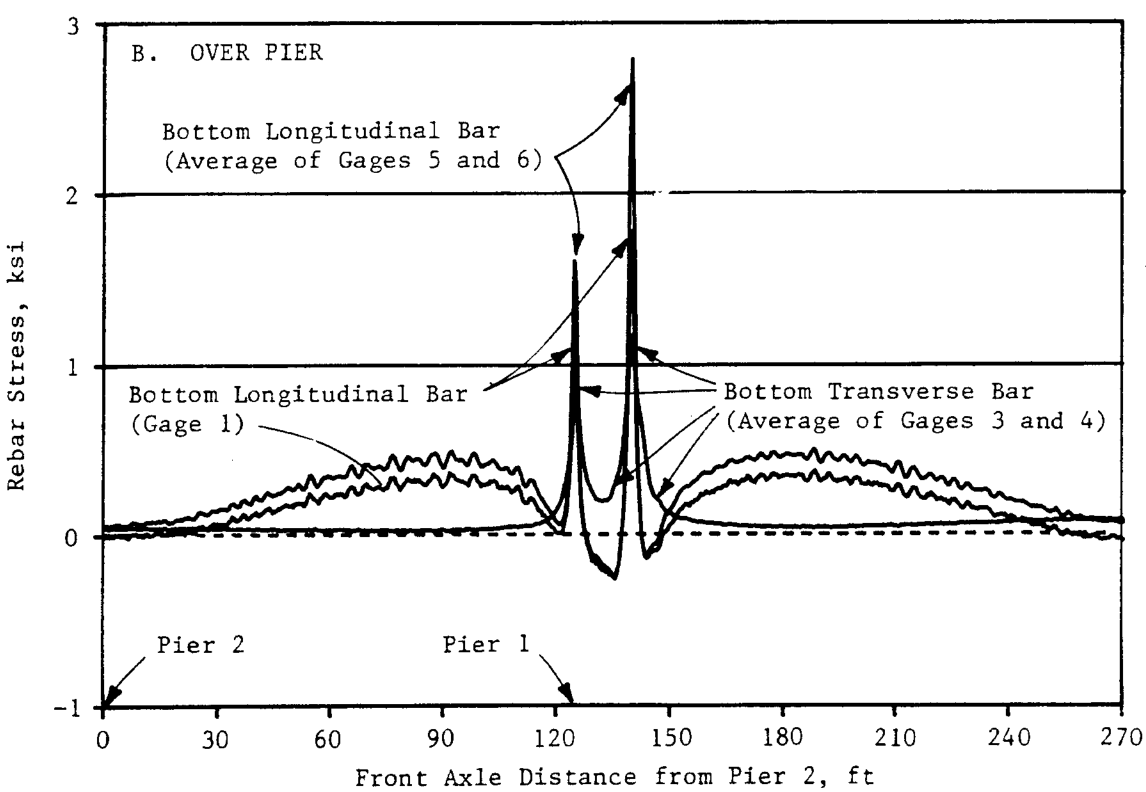


Table 4. Maximum rebar stresses in isotropic decks under 16-kip wheel load, compared to allowable levels.

	12x12 Grid				8x8 Grid	
Live-Load Stress, ksi	Site I-1(12)	Site I-2(12)	Site I-3	Site I-13	Site I-1(8)	Site I-2(8)
Allowable*	14.3	12.9	14.6	14.6	15.7	14.8
Measured**	9.1	10.4	3.0	2.3	7.9	6.7

*Allowable live-load stress = (24 ksi - dead-load stress)/ 1.3.

**Measured live-load stress = maximum stress over service life under 16-kip vehicular wheel load, obtained in load tests.

allowable level. Stresses at Gages 2, 13, 14, 15, and 16 were not obtained in the 1990 test due to gage failures. Note that this deck had been in service for 4 years by 1990, and previous rebar stresses were lower than those presented in Figure 9. This was also typically observed in other instrumented decks.

C. Rebar Stress Histories

Figure 10 shows evolution of maximum stresses in bottom transverse rebars with deck age, for the three reinforcement arrangements in the four instrumented deck slabs. Comparison of these curves indicates that the 12-in. grid experienced higher stresses than the other two patterns, which was expected since less steel was used in the 12-in. grid. Note that at the deck age of 5 years, two tests were performed in Sites I-2 and A-2, and two stress readings recorded. These different readings characterize variation of stress results over deck age, as shown in the graphs. This variation can be attributed to uncertainty in loading paths, possible nonlinearity of behavior under test loads of various magnitudes, and/or possible electrical noise influencing data recording (*26*). Such variation is inevitable in a field-testing environment, and is not a cause for concern. Nevertheless, it may be observed in Figure 10 that rebar stresses may increase noticeably in the first year or two of service but remain relatively constant thereafter, regardless of reinforcement arrangement.

Figure 11 shows histories of maximum stress in bottom longitudinal rebars at midspan between two girders for the 12x12 grid at Sites I-3 and I-13. These stresses also show a trend of stress evolution similar to that of the bottom transverse rebars, although stress levels were lower. Comparison of Figures 10 and 11 indicates that the stresses in transverse rebars are generally higher than those in longitudinal bars or approximately equal to them over the service lives of these decks. Thus, only transverse rebar stresses are compared here with their allowable levels for evaluation of serviceability.

Global maximum rebar stresses of the isotropic decks shown in Figure 10 are listed in Table 4. Using a simplified model of transversely continuous beams and the AASHTO allowable stress method (*1*), dead-load stresses in transverse rebar were found for the instrumented decks, under a uniformly distributed dead load

Figure 10. Maximum stress histories of bottom transverse rebars under 16-kip wheel load.

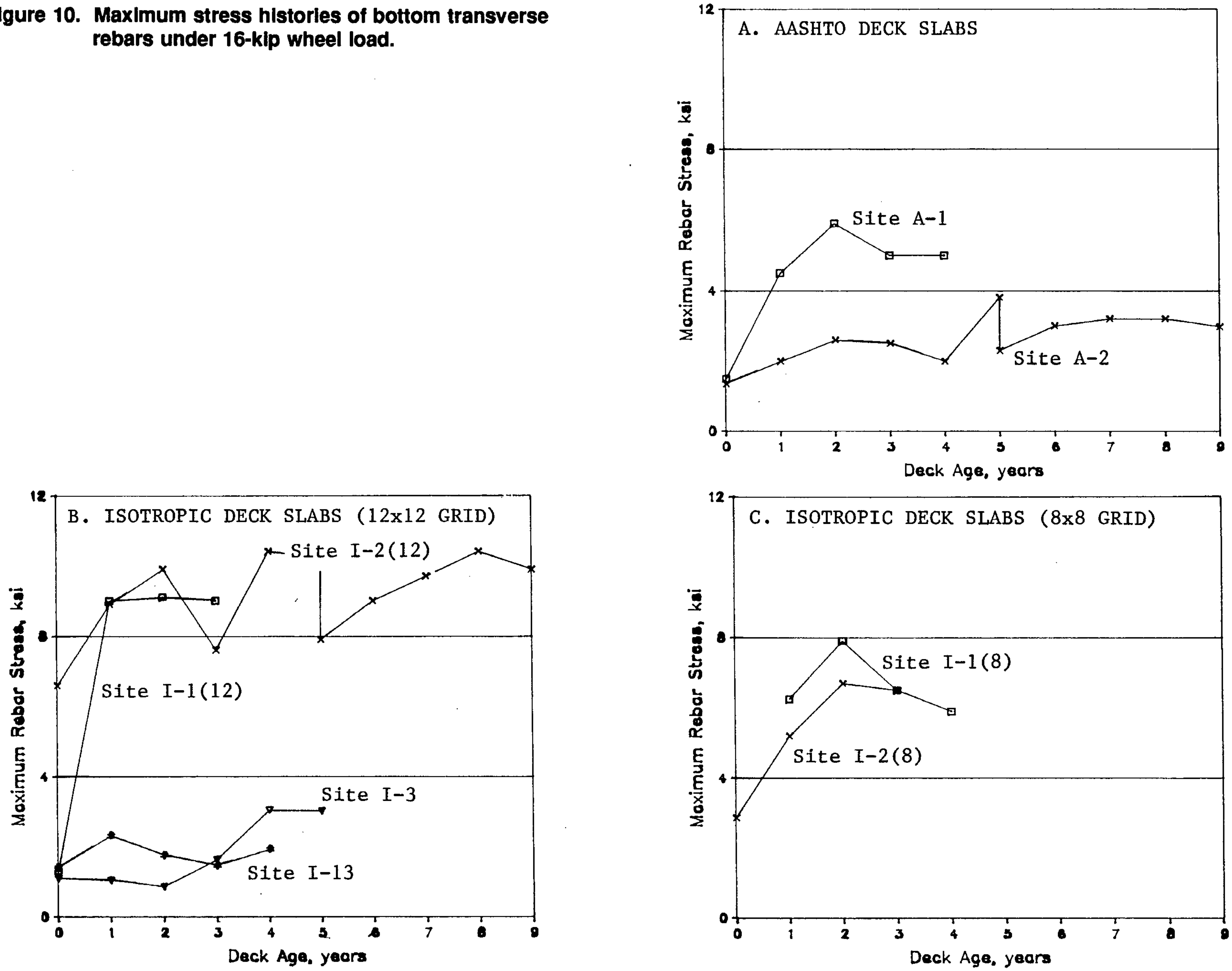
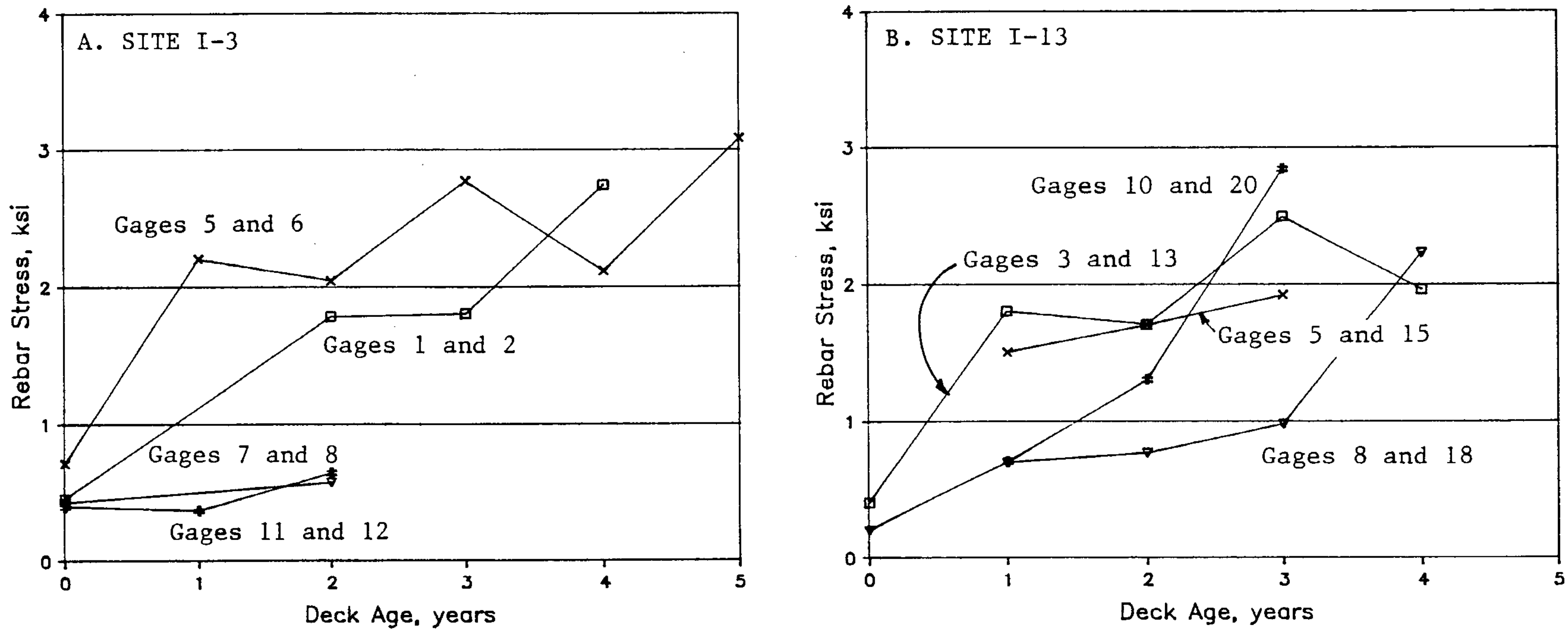


Figure 11. Maximum stress histories of bottom longitudinal rebars under 16-kip wheel load (12x12 grid).



of 142 psf (accounting for weights of the deck, stay-in-place forms, and future overlay). Allowable live-load rebar stresses are computed as differences between a total allowable stress of 24 ksi (for Grade 60 steel) and dead-load stresses, with a conservative maximum impact factor of 1.3 according to the AASHTO code (*25*). These are also listed in Table 4 for comparison. Measured maximum stresses are all lower than allowable levels, based on this conservative analysis, for the two isotropic reinforcement arrangements. Table 4 also shows much wider margins available for 8x8 grids than for 12x12 grids in the same deck slabs (I-1 and I-2).

IV. SURFACE CONDITION AND CRACKING

A. General Overview

From 1986 to 1991, the first 13 experimental isotropic decks (Sites I-1 to I-13 and Sites A-1 and A-2) were inspected annually for possible deterioration affecting serviceability. They were examined for cracking, spalling, and delamination by visual and sonic (chain-drag) methods. Both top and bottom deck surfaces were inspected. No spalling or delamination was observed. Generally, cracking was judged to be minor, and the decks have performed satisfactorily (*27,28*). In 1991, for performance comparison between the isotropic and regular decks, 11 AASHTO decks (Sites A-3 to A-13) were added to the sample. Note that this formed a 13-to-13 comparison of the two types of reinforcement, because Sites A-1 and A-2 were already included in the original inspection program. These added decks are included in Table 1B and their locations indicated in Figure 2. These sites were selected for their similarity to the isotropic decks in age, structural type (a concrete slab on multiple steel stringers), and traffic volume, since these factors were considered important to performance of RC bridge decks. This similarity can be seen by comparing these two groups in Tables 1A and 1B. The initial selection was made using the state bridge inventory, and Site A-5 was found later during field inspection to have been overlaid with asphalt. It thus could provide no comparative data. In 1991, the rest of isotropic decks in service (Sites I-14 to I-28) were added to the inspection program, for a complete statewide overview of their performance and condition, with the results also included here. Table 5 gives inspection histories of the isotropic and AASHTO bridge decks.

At Sites I-1, I-2, A-1, and A-2, a few bottom transverse cracks highlighted by efflorescence were visible from the ground. Note that these cracks were present in similar intensities in all sections regardless of reinforcement arrangements. Sites I-7 and I-8 had more intensive bottom transverse and longitudinal cracks, which were highly visible. These two sites will be discussed and compared later with Sites A-7 and A-8, along with top surface inspection data. Sites I-1, I-2,

Table 5. Isotropic and AASHTO deck slabs inspected 1986-91.

Year	Sites I-1 to I-13	Sites I-14 to I-28	Sites A-1 to A-2	Sites A-3 to A-13
1986	Yes	No	Yes	No
1987	Yes	No	Yes	No
1988	Yes	No	Yes	No
1989	Yes	No	Yes	No
1990	Yes	No	Yes	No
1991	Yes	Yes	Yes	Yes

Table 6. Summary of 1991 Inspections.

A. ISOTROPIC DECKS

I-1. Bay View Road over I-590, Monroe County (built 1982)
Both isotropic patterns were used (12x12 and 8x8), with cracking reported as 11 and 6 in./sq yd, respectively. Cracking generally located near load plates, reported as "very fine" with 0.003 in. max width for both transverse and longitudinal cracks; highlighted by efflorescence visible beneath the deck in all four bays.

I-2. Rte 7 over Elm St, Albany County (built 1983)*
Both isotopic patterns were used (12x12 and 8x8). Cracking had increased for both patterns, but was greater in the 8x8, most being localized and appearing to be an isolated problem. Maximum widths in the 12x12 section were 0.007 in. transverse and 0.009 longitudinal. Stay-in-place forms were removed in the test sections.

I-3. Rte 10 over Chatauqua Creek, Chautauqua County (built 1987)*
All types of cracking increased, with largest increase reported as longitudinal; most were random and averaged 2 to 3 ft long. Cracking intensity was noted to vary from pour to pour. Maximum widths were 0.003 in. transverse and 0.006 longitudinal.

I-4. Hutchinson River Parkway Southbound over Mamaroneck River, Westchester County (built 1985)*
No change in cracking from the previous year, most having appeared in the gore area shortly after construction. Maximum widths in the driving lanes were 0.005 in. transverse and 0.003 longitudinal.

I-5. Hutchinson River Parkway Northbound over Mamaroneck River, Westchester County (Built 1986)*
No significant change in cracking reported; maximum widths were 0.005 in. transverse and 0.009 longitudinal.

I-6. Rte 5 over Oswegatchie River, Lewis County (built 1986)*
Longitudinal cracking predominated, generally with two hairlines located over the girders, ranging in width from 0.001 to 0.003 in.

I-7. Rte 104 Eastbound over Hard Road, Monroe County (built 1985)
This deck had highest total crack density (25 in./sq yd) of all bridges studied; transverse and longitudinal cracks were present throughout the deck surface and transverse cracks followed the skewed reinforcement pattern. Crack density had not changed significantly from the previous year, and was highlighted by efflorescence in all four bays. Maximum widths were 0.007 in. transverse, 0.010 longitudinal, and 0.007 diagonal.

I-8. Rte 106 Westbound over Hard Road, Monroe County (built 1985)
This deck ranked second in total crack density (22 in./sq yd), with both transverse and longitudinal cracks present throughout the deck surface. Maximum widths were 0.009 in. transverse, 0.007 longitudinal, and 0.010 diagonal. No significant change in density was reported from the previous year. Here, too, they were highlighted by efflorescence visible under the deck in all four bays.

I-9. Rte 17 over Rte 219, Cattaraugus County (built 1986)*
This deck was in very good condition, with a few small longitudinal cracks over the girders at each end of the span and a maximum width of 0.007 in.

I-10. US 62 over Barge Canal, Erie and Niagara Counties (built 1986-87)*
No significant changes reported. Due to settlement of approach slabs beginning in 1987, transverse cracks were present in both lanes at both ends of the bridge. Faulting at the settled northbound approach slab resulted in a "launch pad" for motorists, but thus far no damage to the deck. Stay-in-place forms were removed in the center bay under the closure pour area.

I-11. Rte 446 over Cuba Lake Outlet, Allegany County (built 1986)*
Deck in excellent condition with only a few small shrinkage cracks visible; maximum longitudinal crack width reported was 0.005 in.

I-12. Versailles Rd over Cattaraugus Creek, Cattaraugus and Erie Counties (built 1986)*
Deck in good condition with no significant change in cracking from the previous year; most cracks located along the deck's north fascia with maximum widths of 0.016 in. transverse and 0.013 in. longitudinal.

I-13. Rte 11 over Chateaugay River, Franklin County (built 1988)*
Deck in very good condition with a few small cracks present; maximum widths were 0.005 in. longitudinal and 0.007 diagonal.

I-14. Rte 470 over Mohawk River, Albany County (built 1988)*
Deck in very good condition with a few small cracks present; maximum widths were 0.005 in. transverse and 0.003 longitudinal.

I-15. Rte 32 over Conrail, Albany County (built 1990)*
Only transverse cracks present, located mostly at midspan; maximum width reported was 0.016 in. -- the widest among all isotropic decks -- but average widths were 0.006 in.

I-16. Rte 7 over D&H Railroad, Schenectady County (built 1989)*
No cracks, spalls, or delaminations were reported.

I-17. Rte 4 over Champlain Canal, Washington County (built 1989)*
Deck in very good condition, with one small diagonal crack.

I-18. Shells Bush Rd over West Canada Creek, Herkimer County (built 1988)*
No cracks, spalls, or delaminations reported.

I-19. Rte 31 over Conrail, Oneida County (built 1990)*
Finishing after pour placement apparently left bumpy, wavy surface that had to be ground smooth, resulting a surface in poor condition with polished aggregate in many areas. Transverse cracks followed the skewed reinforcement pattern, with maximum width of 0.004 in.

I-20. Horseshoe Island Rd over Erie Canal, Onondaga County (built 1989)*
Isolated transverse cracking occurring only at ends of both approach pours; maximum transverse crack width reported was 0.007 in.

I-21. Rte 5 over Little Canada Way, Chautauqua County (built 1989)*
Deck in very good condition, with a few small cracks present at each end of the span; maximum widths were 0.010 in. transverse and 0.005 in. diagonal.

I-22. Rte 16 over Cazenovia Creek in Holland, Erie County (built 1988)*
No cracks, spalls, or delaminations reported.

I-23. Rte 16 over Cazenovia Creek in Wales, Erie County (built 1989)*
All crack types present,with longitudinal most prevalent, the widest being reported as 0.040 in. -- the record for all decks studied (although measured at the crack's tip as shown in Fig. 2). Steel girders were noted as being embedded into the abutments, apparently creating a stiffer end joint and possibly explaining the high cracking at both ends of the span.

I-24. Washington Ave over Conrail, Chemung County (built 1988)*
A few small transverse and longitudinal cracks were present at various locations, with maximum widths of 0.005 in. transverse and 0.002 longitudinal.

I-25. Madison Ave over Chemung River, Chemung County (built 1988)*
Crack intensity varied from pour to pour, with many fine cracks in specific areas. Maximum transverse crack width was 0.009 in., and longitudinal cracks averaged 1 to 2 ft long.

I-26. County Rd 541 over Susquehanna River, Broome County (built 1990)
Deck in very good condition with only two transverse cracks, the wider being 0.007 in.

I-27. South Grand St over Cobleskill Creek, Schoharie County (built 1989)*
No cracks, spalls, or delaminations reported.

I-28. Rte 97 over Callicoon Creek, Sullivan County (built 1987)*
Most cracking reported was transverse, located mostly at midspan, with maximum widths of 0.010 in. transverse and 0.009 longitudinal.

B. AASHTO DECKS

A-1. Bay View Rd over I-590, Monroe County (built 1982)
Cracking increased slightly over previous year. Transverse cracks followed the skewed reinforcement pattern. Maximum widths were 0.005 in. transverse and 0.003 longitudinal, highlighted by efflorescence visible in all four bays.

A-2. Rte 7 over Elm St, Albany County (built 1983)
Cracking increased slightly from previous inspection, with maximum transverse crack width of 0.009 in. Stay-in-place forms were removed from the test sections, revealing cracks highlighted by efflorescence.

A-3. West Third Street over Chadakoin River, Chautauqua County (built 1987)*
Most common cracks reported were transverse; crack intensity varied from pour to pour, with one of the seven pours having densities as high as 18 in./sq yd. Maximum transverse crack width (measured at the tip as shown in Fig. 2) was 0.025 in -- the highest reported among all bridges studied.

A-4. Hutchinson River Parkway Southbound over Mamaroneck Ave, Westchester County (built 1985)*
Deck in good condition with a few cracks present in each span; maximum widths were 0.013 in. transverse and 0.007 longitudinal.

A-5. Hutchinson River Parkway Northbound over Mamaroneck Ave, Westchester County (built 1985)*
Structure had an asphalt driving surface and thus was omitted from the comparisons.

A-6. Rte 122 over Trout River, Franklin County (built 1986)*
No cracks, spalls, or delaminations reported.

A-7. Rte 104 Eastbound over Holt Rd, Monroe County (built 1985)
Had highest crack density (11 in./sq yd) of all 13 AASHTO-reinforced decks. Transverse and longitudinal cracking were reported throughout the deck surface, with maximum widths of 0.009 in. transverse and 0.006 longitudinal, highlighted by efflorescence visible beneath the deck in all four bays.

A-8. Rte 104 Westbound over Holt Rd, Monroe County (built 1985)
Reported crack density was 6 in./sq yd, with only transverse cracking (maximum 0.007 in. width) in the either the driving or passing lanes, highlighted by efflorescence visible in all four bays.

A-9. Rte 17 over Allegheny River, Cattaraugus County (built 1986)*
Eastbound and westbound structures are separate; crack density, although minor, was higher on the eastbound structure, with maximum 0.007 in. width for both transverse and longitudinal.

A-10. Forest Rd over Thruway, Erie County (built 1988)*
Most cracking was longitudinal, located mostly at midspan and both ends, with maximum transverse crack width of 0.005 in.

A-11. Rte 21 over Canacadea Creek, Allegany County (built 1985)*
Two diagonal cracks at one end of the span, the wider being 0.007 in.

A-12. Hasting Rd over Olean Creek, Cattaraugus County (built 1985)*
No spalls or delaminations reported; one crack 0.002 in. wide and 2 ft long at one end of the span.

A-13. Rte 3 over Black River, Jefferson County (built 1987)*
No cracks, spalls or delaminations reported.

*Remaining stay-in-place forms in good condition, showing no signs of rust or leaking.

Table 7. Crack densities on deck slabs in 1991.

Location	Deck Area, sq ft	Crack Density, in./sq yd		
		Total	Transverse	Longitudinal
A. ISOTOPIC DECKS				
I-1(12)	710	11.1	7.5	3.7
I-2(12)	2,864	1.8	1.2	0.2
I-3	49,000	2.2	0.7	1.4
I-4	12,180	2.8	2.6	0.2
I-5	7,632	1.2	1.0	0.2
I-6	8,366	3.3	0.2	3.1
I-7	6,300	25.8	16.0	8.8
I-8	6,300	22.3	13.5	8.0
I-9	5,865	1.2	0.4	0.6
I-10	22,900	2.0	1.0	0.9
I-11	4,900	0.8	0.0	0.8
I-12	14,000	3.2	3.0	0.2
I-13	32,045	0.2	0.1	0.1
	Average[a]	2.89	1.61	1.04
I-1(8)	710	6.4	5.2	1.2
I-2(8)	2,822	3.8	2.2	1.5
I-14	8,288	0.3	0.2	0.1
I-15	11,040	2.5	2.5	0.0
I-16	4,960	0.0	0.0	0.0
I-17	12,274	0.0	0.0	0.0
I-18	7,866	0.0	0.0	0.0
I-19	6,028	1.2	1.1	0.1
I-20	7,304	2.9	2.9	0.0
I-21	2,371	1.2	0.6	0.3
I-22	3,920	0.0	0.0	0.0
I-23	5,060	3.6	0.7	2.0
I-24	5,068	1.2	0.4	0.8
I-25	23,016	2.3	1.1	1.2
I-26	17,730	0.1	0.1	0.0
I-27	4,050	0.0	0.0	0.0
I-28	28,170	2.0	0.1	0.1
B. AASHTO DECKS				
A-1	3,595	7.1	7.0	0.1
A-2	1,624	2.7	2.3	0.2
A-3	42,136	3.6	3.0	0.6
A-4	5,357	0.5	0.2	0.2
A-5	4,861	NA	NA	NA
A-6	2,148	0.0	0.0	0.0
A-7	5,320	10.4	9.0	1.3
A-8	5,320	6.0	6.0	0.0
A-9	60,120	0.7	0.7	0.0
A-10	7,447	2.4	0.5	1.3
A-11	5,100	0.5	0.0	0.1
A-12	5,496	0.1	0.0	0.1
A-13	17,800	0.0	0.0	0.0
	Average[b]	1.76	1.37	0.26

NA = not applicable.

[a] Average for first 13 isotropic decks (I-1 to I-13) excluding Sites I-7 and I-8.

[b] Average for 13 AASHTO decks (A-1 to A-13) excluding Sites A-7 and A-8.

I-7, I-8, A-1, A-2, A-7, and A-8 were the only ones with forms removed either completely or partially among the sites addressed here, and this is noted in Table 1. Top surface cracking at all sites will now be discussed in more detail.

Table 6 gives narrative descriptions of deck conditions and shows that no spalling or delamination were found on the 28 isotropic decks. Cracking was observed on most decks but was generally considered minor, with greatest severity at Sites I-7, I-8, A-7, and A-8 as discussed in the next section. Crack widths were selectively measured using a graduated crack-width card with comparators ranging from 0.002 to 0.06 in. They were measured at the root of each crack, instead of its traffic-worn tip, with only a few exceptions (*28*). Maximum crack widths are also reported in Table 6. Stay-in-place (SIP) forms where used were all in good condition, showing no signs of rusting or leaking.

Cracking observed on the upper deck surface is classified into three types according to direction of extension: transverse, longitudinal, or diagonal. Transverse cracks are defined as perpendicular to traffic flow, longitudinal as parallel, and all others noted as diagonal. Examination and recording of cold-joint cracking at the ends of concrete pours were discontinued in 1989, since cold joint cracking was considered unavoidable and irrelevant to reinforcement arrangement. Cracking density, defined as crack length per unit area (in./sq yd), is used for quantitative measurement of cracking severity.

Table 7 gives deck area and total, transverse, and longitudinal crack densities for all the inspected deck slabs as of 1991. Total crack densities ranged from 0 to 25.8 in./sq yd. Except for Sites I-1, I-2, I-7, and I-8, total crack densities were no higher than 3.6 in./sq yd. Sites I-1 and I-2 had been subjected to severe test wheel loads (as high as about 30 kips over an 8- by 20-in. loading plate) and most cracks were found near areas where those plates had been. Sites I-7 and I-8 showed highest densities and this also will be addressed in the next section.

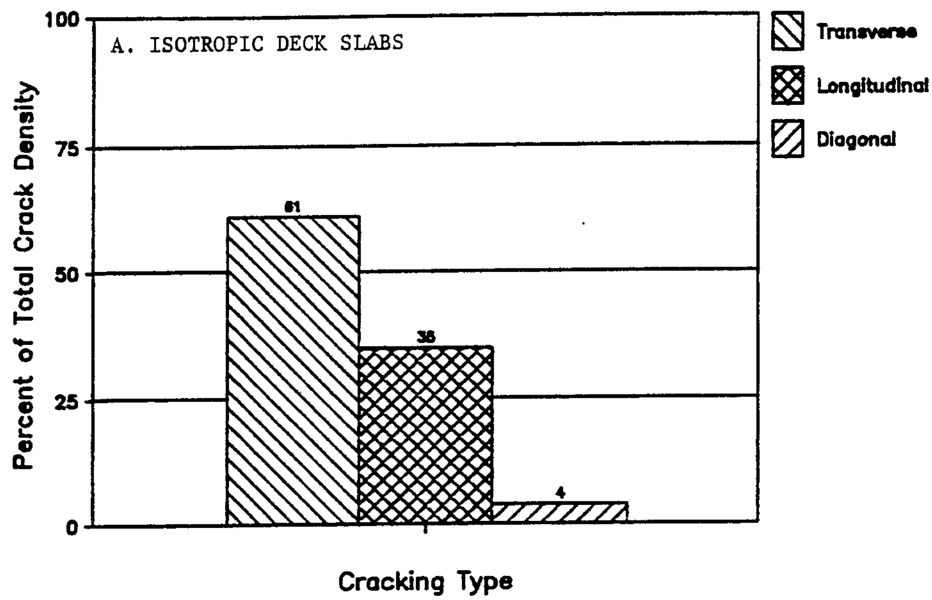
Figure 12 shows crack distribution by types as of 1991, with transverse cracking predominant and longitudinal much less frequent for both isotropic and AASHTO decks. On the other hand, this is even more pronounced for the latter, which will also be discussed later.

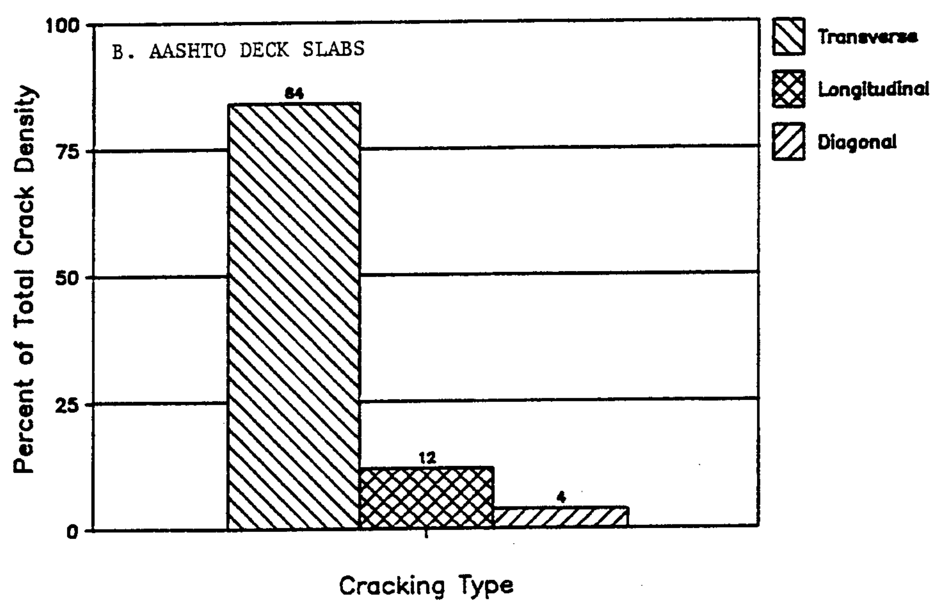
B. Most Severely Cracked Deck Slabs (Sites I-7, I-8, A-7, and A-8)

Table 7 shows that these sites exhibited most cracking for their respective reinforcement schemes (except for Sites A-1 and I-1 where all three reinforcement patterns were present). Table 1 indicates that these four bridges were included in the same construction project, built by the same contractor on the same route in 1985. This issue is addressed here, due to the significant differences in crack severity when compared to other deck slabs within each reinforcement group.

A special investigation attempted to identify causes of this relatively severe cracking. Two cores were taken in 1990 from Site I-7 (Rte 104 eastbound). Core 1 was at an intersection of longitudinal and diagonal cracks (and also of longitudinal and transverse bars), and Core 2 over a longitudinal crack (also

Figure 12. Cracking densities in isotropic and AASHTO deck slabs.





over a longitudinal bar) along the centerline between two interior girders. The cores were then examined by Materials Bureau personnel. Epoxy coating of top bars in both cores was found to be intact, and no corrosion was observed. In Core 1, the crack extended from the top surface into about one-quarter of the slab's depth. Multiple aggregate fractures were found in Core 2, with the crack penetrating its full depth. This indicates that the longitudinal cracking occurred after concrete had developed its strength, and was load-related. Early concrete strengths at both sites were also found to have been obtained after the contractor's request for early opening of the bridges. This is generally done for the contractor's convenience in moving heavy construction equipment. It is possible that such movements caused the observed longitudinal cracks. It is interesting to note that longitudinal cracking was not prominent at Sites A-7 and A-8, but the opposite was true at Sites I-7 and I-8. Further, note that Sites I-7 and I-8 exhibited more longitudinal cracking, attributable to possible overloading and use of less steel. Sites I-7 and I-8 actually experienced higher traffic volumes than Sites A-7 and A-8, because of an entrance after Site A-8 and before Site I-8, and an exit after Site I-7 but before Site A-7.

A total of eight cores (two per deck) were later extracted in 1991 from Sites I-7, I-8, A-7, and A-8 for further examination. For each deck, one was extracted at a transverse crack and another 1 ft away from the crack (not on a crack). All cores were tested and analyzed by the Department's Materials Bureau. Uncracked cores were tested for compression strength, unit weight, chloride content, and absorption, with no results considered significant since they were normal. The four cores with transverse cracks were carefully examined and found to have cracked as a result of shrinkage. Construction records also show that at each site, two of eight tested slumps exceeded the maximum allowable level of the New York State construction specifications (*24*). Higher slump may cause more transverse cracking due to more severe concrete shrinkage. Further, transverse cracking was predominantly higher at all four sites within their respective groups of reinforcement type (see Table 7), indicating that certain adverse factors (possibly improper concrete placement) affecting transverse cracking were present for all these sites.

It is concluded that the relatively heavy cracking at Sites I-7 and I-8 was due to improper construction procedures and possible overloading. Despite the abnormal cracking observed, Sites I-7, I-8, A-7, and A-8 were rated 5 in routine inspection as noted in Table 8, meaning "minor deterioration" and "functioning as originally designed." These ratings were assigned based on riding quality across the bridges with respect to spalling, delamination, and cracking (*29*).

C. Comparison of Lightly Reinforced and AASHTO Deck Slabs

As described earlier, 13 AASHTO decks were selected for comparison with the first 13 isotropic decks. This section compares results for these two groups.

Table 7 lists crack densities on both the isotropic and AASHTO decks for comparison. Average crack densities for the two comparison groups are also given, excluding the abnormal Sites I-7 and I-8, and A-7 and A-8. Average

Table 8. Condition ratings of deck slabs.

Location	Inspection Date	Condition Rating[a]
A. ISOTROPIC DECKS		
I-1	5/90	6
I-2	8/90	5
I-3	8/90	7
I-4	3/90	7
I-5	3/90	7
I-6	3/89	7
I-7	10/90	5
I-8	10/90	5
I-9	11/90	6
I-10	11/90	7
I-11	3/90	7
I-12	8/90	7
I-13	4/89	7
	Average[b]	6.6
I-14	10/89	7
I-15	12/90	7
I-16	12/89	7
I-17	10/89	7
I-18	4/89	7
I-19	8/90	7
I-20	10/89	7
I-21	4/90	7
I-22	10/90	7
I-23	1/90	7
I-24	5/90	7
I-25	5/90	7
I-26	1990	7
I-27	1/90	7
I-28	1/90	6
B. AASHTO DECKS		
A-1	5/90	6
A-2	8/90	5
A-3	8/89	6
A-4	2/90	7
A-5	2/90	NA
A-6	10/89	7
A-7	10/90	5
A-8	10/90	5
A-9	12/90	5
A-10	12/90	6
A-11	5/89	7
A-12	9/89	7
A-13	10/89	7
	Average[c]	6.3

NA = not applicable (covered by asphalt overlay).

[a] Inventory and Inspection System files:
1 = potentially hazardous
2 = used to shade between ratings of 1 and 3.
3 = serious deterioration or not functioning as originally designed.
4 = used to shade between ratings of 3 and 5.
5 = minor deterioration, functioning as originally designed.
6 = used to shade between ratings of 5 and 7.
7 = new condition.

[b] Average for first 13 isotropic decks (I-1 to I-13) excluding Sites I-7 and I-8.

[c] Average for 13 AASHTO decks (A-1 to A-13), excluding Sites A-7 and A-8).

transverse crack densities were 1.61 and 1.37 in./sq yd, respectively, for the isotropic and AASHTO decks, without substantial difference. However, average longitudinal crack densities were 1.04 in./sq yd for the isotropic decks and 0.26 for the AASHTO decks, showing a relatively larger difference. This is also reflected by a higher percentage of longitudinal cracking in isotropic decks in Figure 12, compared to AASHTO decks.

Table 8 gives condition ratings for these two groups of deck slabs. Average ratings were 6.6 and 6.3, respectively, for isotropic and AASHTO decks, showing no significant difference. (Note that Sites I-7, I-8, A-7, and A-8 were excluded in averaging, because of their extreme cracking density.)

Two types of longitudinal cracking were observed -- between girders (probably due to positive moment by wheel loads) and over girders (probably due to negative moment by wheel loads). Table 9 gives distributions of longitudinal-crack types between the two deck groups, with respect to crack locations relative to the girders. Two types are considered here, namely over (or close to) a girder or between (or far from) two girders. They are so distinguished for their respective negative and positive moment origins. Average ratios of the two types of longitudinal crack are 63 percent over-girder cracking versus 37 percent between-girder cracking for isotropic decks (excluding abnormal Sites I-7 and I-8), and 74 versus 26 percent for AASHTO decks (excluding abnormal Sites A-7 and A-8), without notable difference.

These comparisons are for the complete deck sample (Sites I-1 to I-13 versus Sites A-1 to A-13) with various but comparable individual ages, without referring to specific ages of each deck pair that may be important in deterioration, which is addressed now. Figures 13, 14, and 15 show cracking development with age. Figure 13 shows total crack densities over age for the two comparison groups. Figures 14 and 15 show transverse and longitudinal cracks separately. Top-surface cracking generally increases with age in both isotropic and AASHTO decks. It is observed, however, that crack densities once actually decreased with age for Sites I-2, I-6, and I-13. This is apparently either because of inspector inexperience or reduced crack visibility with less moisture near the deck surface when inspected. Because most observed cracks are hairline or smaller, these two factors did affect inspection results for crack density. Sites I-1(12), I-1(8), I-7, I-8, A-1, A-7, and A-8 had the highest transverse cracking as shown in Figure 14. Figure 15 shows the relationship of longitudinal cracking to deck age. Site I-6 (Rte 3) had relatively higher longitudinal cracking (excluding Sites I-1(12), I-7, and I-8). (Much of this involved single hairline cracks no wider than 0.003 in., extending through the span over one girder.)

Taking an age section across all the deck slabs included in Figures 13, 14, and 15 and comparing crack densities at the same age, the effect of age on cracking of the two deck slab groups can be clearly seen. Excluding Sites I-7, I-8, A-7, and A-8 in Figure 14, transverse cracking is equivalent on isotropic and AASHTO deck slabs. Results of an earlier comparison showed similar transverse cracking between New York isotropic and North Carolina AASHTO decks (*27,30*).

Table 9. Longitudinal crack distribution related to girder location.

		Cracking					
Location	Deck Area, sq ft	Over Girders, ft[a]	Between Girders, ft	Over Girders, in./sq yd[a]	Between Girders, in./sq yd	% Over Girders	% Between Girders
A. ISOTROPIC DECKS							
I-1(12)	710	20	4	3.0	0.6	83	17
I-2(12)	2,864	5	0	0.2	0.0	100	0
I-3	49,000	198	447	0.4	1.0	31	69
I-4	12,180	2	19	0.0	0.2	0	100
I-5	7,632	13	0	0.2	0.0	100	0
I-6	8,366	217	23	2.8	0.3	90	10
I-7	6,300	75	441	1.3	7.6	15	85
I-8	6,300	55	412	0.9	7.1	12	88
I-9	5,865	32	3	0.6	0.1	91	9
I-10	22,900	75	112	0.4	0.5	40	60
I-11	4,900	16	20	0.4	0.4	44	56
I-12	14,000	19	10	0.1	0.1	66	34
I-13	32,045	13	14	0.0	0.0	48	52
					Average[c]	63	37
B. AASHTO DECKS							
A-1	3,595	2	2	0.1	0.1	50	50
A-2	1,624	3	0	0.2	0.2	100	0
A-3	42,136	116	119	0.3	0.3	49	51
A-4	5,357	9	3	0.3	0.2	75	25
A-5[b]	4,861	NA	NA	NA	NA	NA	NA
A-6	2,148	0	0	0.0	0.0	--	--
A-7	5,320	48	14	1.0	0.3	77	23
A-8	5,320	0	0	0.0	0.0	--	--
A-9	60,120	2	6	0.0	0.0	25	75
A-10	7,447	88	5	1.3	1.1	95	5
A-11	5,100	7	0	0.1	0.0	100	0
A-12	5,496	2	0	0.0	0.0	100	0
A-13	17,800	0	0	0.0	0.0	--	--
					Average[d]	75	26

NA = not applicable.

[a]Within 2 ft of girder centerline.
[b]Deck had asphalt overlay.
[c]Average for isotropic decks, excluding Sites I-7 and I-8.
[d]Average for AASHTO decks, excluding Sites A-7 and A-8.

Figure 13. Total crack densities on deck top surfaces.

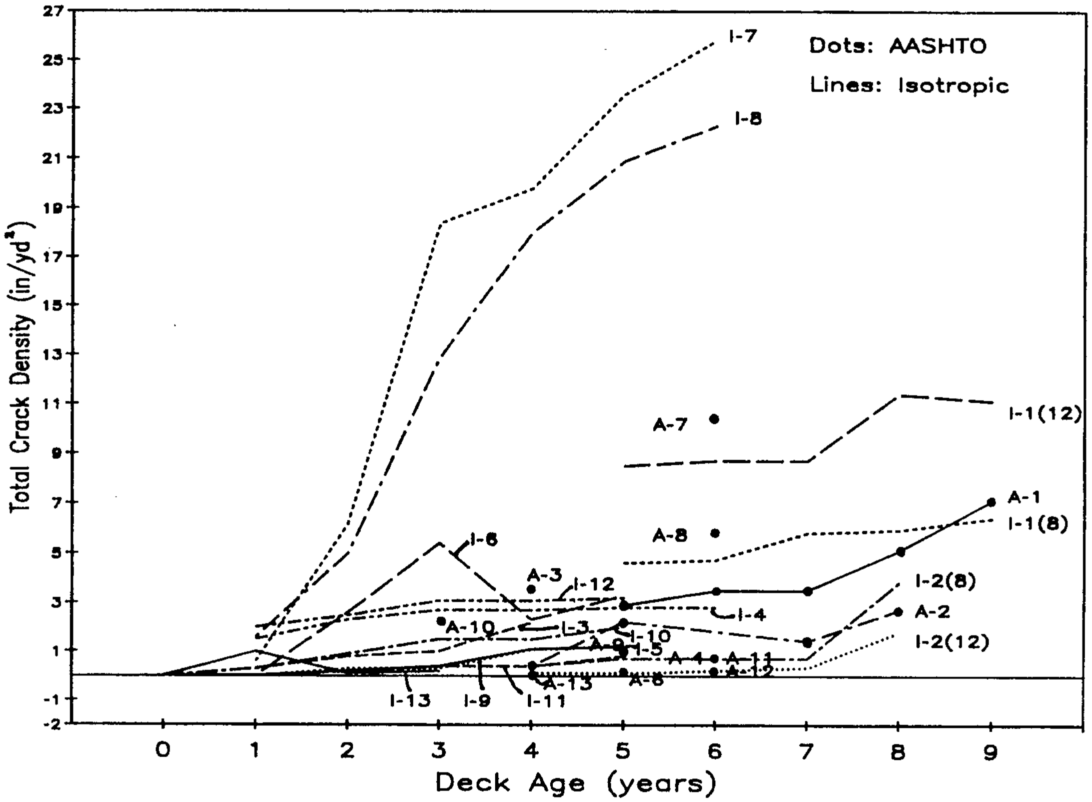


Figure 14. Transverse crack density on deck top surfaces.

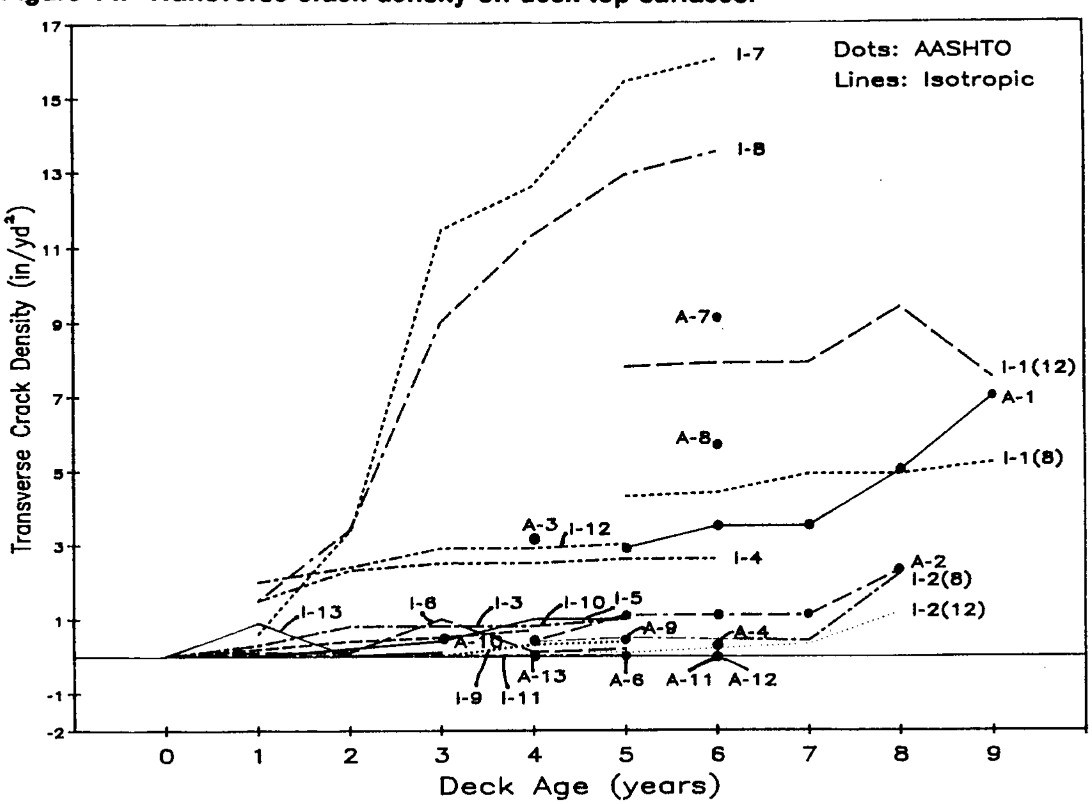
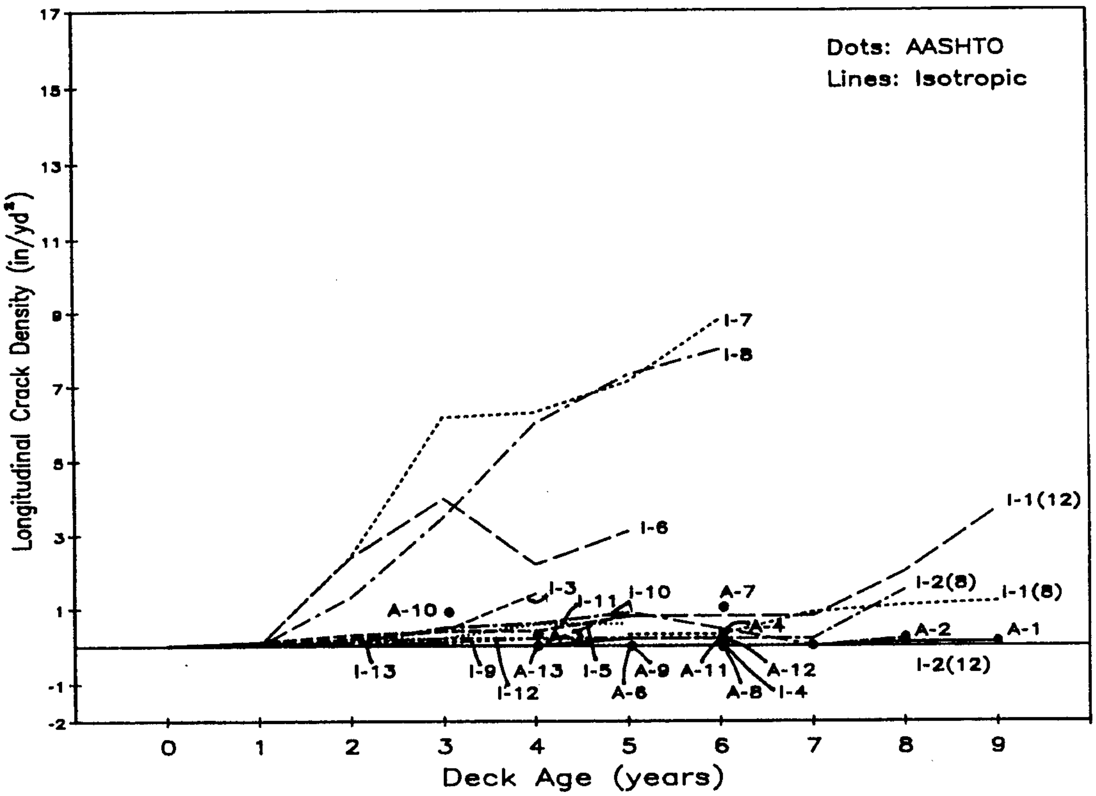


Figure 15. Longitudinal crack density on deck top surfaces.



On the other hand, Figure 15 shows slightly higher longitudinal cracking in isotropic than in AASHTO deck slabs for various ages (excluding Sites I-7, I-8, A-7, and A-8), although most had densities lower than 2 in./sq yd. This is attributable to using less steel in the isotropic decks. Thus, a higher reinforcement ratio than 0.24 percent is recommended to reduce such cracking due to possible overloading and to increase bridge deck durability. A reinforcement ratio closer to 0.3 percent is preferred, because of the experience with 0.24 and 0.36 percent presented in this report, and the positive experience with Ontario practice (*23*). A rebar spacing of 9 in. at centers is suggested, without further change to the present design as described earlier. This will result in a reinforcement ratio of 0.32-percent using an effective depth of 6.5 in.

An annual saving of $1.3 million may be realized if this reinforcement pattern is implemented in New York. This estimate is based on 1993 quantities and bid prices for deck slab concrete.

V. CONCLUSIONS AND RECOMMENDATIONS

Strength of empirically designed isotropic deck slabs was verified in previous research as adequate for current wheel loading. Their long-term serviceability when subjected to severe service fatigue is addressed here. Experimental isotropic slabs in New York State have been examined periodically by both load test and general inspection over their service lives (the longest being 10 years) and compared with AASHTO deck slabs.

Maximum rebar stresses in those isotropic decks under the AASHTO wheel load of 16 kips were always lower than allowable levels based on conservative analyses. Maximum transverse rebar stresses under this wheel load might increase noticeably for the first year or two of service, but remained relatively constant thereafter, regardless of reinforcement pattern.

No spalling or delamination has been found in annual inspections over 6 years, and cracking is generally judged to be minor. Isotropic deck slabs experienced transverse cracking comparable to AASHTO slabs. Higher longitudinal cracking was found on isotropic slabs than on the AASHTO slabs and was concluded to be load-related. Thus, a higher reinforcement ratio, closer to 0.3 percent, is recommended, to reduce possible cracking and increase bridge deck durability.

ACKNOWLEDGMENTS

Assistance and cooperation of New York State Department of Transportation personnel in this study over the years are gratefully appreciated. Discussions with D. B. Beal and M. J. Loftus of the Structures Design and Construction Division were most helpful. I. A. Aziz, W. J. Deschamps, E. W. Dillon, and R. D. Wright of the Engineering Research and Development Bureau assisted in load testing and inspection. P. J. St. John, J. S. Candib, and T. E. Pericak of the Materials Bureau examined the cores. Many regional personnel assisted in traffic control during load testing and inspection over the years, and that work could not have been completed without their efforts.

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