SCHOOL OF CIVIL ENGINEERING

JOINT HIGHWAY RESEARCH PROJECT

JHRP-78-2

REINFORCED CONCRETE
GIRDER BRIDGE STUDY

Michael S. Black

PURDUE UNIVERSITY
INDIANA STATE HIGHWAY COMMISSION

Final Report
REINFORCED CONCRETE GIRDER BRIDGE STUDY

TO: J. F. McLaughlin, Director
Joint Highway Research Project

FROM: H. L. Michael, Associate Director
Joint Highway Research Project

February 28, 1978

Project: C-36-56U

File: 7-4-21

The attached Final Report titled "Reinforced Concrete Girder Bridge Study" is submitted on the JHRP research study titled "Behavior of Multiple-Span Reinforced Concrete Bridges". The research was performed and the report written by Mr. Michael S. Black, Graduate Assistant in Research on our staff, under the direction of Professors M. J. Gutzwiller and R. H. Lee.

The research reported herein originated with a request from the ISHC to investigate the causes and possible countermeasures which they should employ to minimize the structural distress which was occurring in approximately 60 multiple span reinforced concrete bridges. The research has been successful in finding cause of the distress and the report provides recommendations for minimizing the difficulties which could become serious and expensive.

The Report is forwarded for acceptance as fulfillment of the objectives of the research.

Respectfully submitted,

Harold L. Michael
Associate Director

HLM:ms

cc: A. G. Altschaeffl	G. K. Hallock	C. F. Scholer
O. M. Bevilacqua	D. E. Hancher	M. B. Scott
W. L. Dolch	K. R. Hoover	K. C. Sinha
R. L. Eskew	R. F. Marsh	C. A. Venable
G. D. Gibson	R. D. Miles	L. E. Wood
W. H. Goetz	P. L. Owens	E. J. Yoder
M. J. Gutzwiller	G. T. Satterly	S. R. Yoder

Digitized by the Internet Archive
in 2011 with funding from
LYRASIS members and Sloan Foundation; Indiana Department of Transportation

Final Report
REINFORCED CONCRETE GIRDER BRIDGE STUDY

by

Michael S. Black
Graduate Assistant in Research

Joint Highway Research Project

Project No.: C-36-56U

File No.: 7-4-21

Prepared as Part of an Investigation

Conducted by

Joint Highway Research Project
Engineering Experiment Station
Purdue University

in cooperation with the
Indiana State Highway Commission

Purdue University
West Lafayette, Indiana
February 28, 1978

ACKNOWLEDGEMENTS

The author wishes to express his appreciation to Professors M. J. Gutzwiller, R. H. Lee, and C. D. Sutton for their assistance throughout the course of this project.

The financial assistance granted by the Joint Highway Research Project is gratefully acknowledged.

Special thanks are extended to Messrs. W. B. Abbott and J. D. Mattis of the Indiana State Highway Commission Bridge Division and P. L. Owens of the Indiana State Highway Commission Research and Training Center for their technical assistance and consultation.

TABLE OF CONTENTS

	<u>Page</u>
LIST OF TABLES.	v
LIST OF FIGURES	vi
ABSTRACT.	x
CHAPTER 1: INTRODUCTION.	1
Parameters Affecting Girder Movement	6
Objective.	9
Scope.	9
Procedure.	10
CHAPTER 2: INSTRUMENTATION	14
Strain Measurement	14
Temperature Measurement.	26
Deflection Measurement	27
Measurement Considerations	35
CHAPTER 3: STRUCTURE I-94-41-4480 DATA	39
General.	39
Analysis and Interpretation of Strain Data	46
Analysis and Interpretation of Crack Data.	53
Analysis and Interpretation of Deflection Data	53
CHAPTER 4: THE FAILURE MECHANISM	57
General.	57
Dead Load Rotation	57
Thermal Rotation	57
End Rotation	63
Transformed Section.	68
Girder End Cracking.	70
Pier Cap Spalling.	71
Interior Versus Exterior Girders	72
CHAPTER 5: CONCLUSIONS	75
General.	75
Recommendations.	77

TABLE OF CONTENTS (Continued)

	<u>Page</u>
BIBLIOGRAPHY.	80
APPENDICES	
APPENDIX A: SAMPLE CALCULATIONS	81
Whittemore Temperature Compensation	81
Temperature Calculation	82
Theoretical Strain Calculation (Live Load).	82
Theoretical Deflection Calculation (Live Load).	84
Live Load Rotation.	85
APPENDIX B: STRUCTURE I-70-01-4868.	86
General	86
Gage Details.	86
Recorded Data	92
APPENDIX C: STRUCTURE I-65-54-4654.	97
General	97
Gage Details.	97
Recorded Data	102
APPENDIX D: STRUCTURE I-74-136-4331	105
General	105
Gage Location	105
Recorded Data	113
APPENDIX E: STRUCTURE I-74-134-4328	115
General	115
Gage Location	121
Recorded Data	121

LIST OF TABLES

<u>Table</u>		<u>Page</u>
1	I-94-41-4480 Recorded Strain.	40
2	I-94-41-4480 Recorded Strain (10" Whittemore)	41
3	I-94-41-4480 Concrete Surface Temperature	43
4	I-94-41-4480 Average Strain Across Cracks	44
5	I-94-41-4480 Recorded Deflections	45
6	I-94-41-4479 Recorded Deflections	45
7	End Rotations of Various Structures	65

AppendicesTable

B1	I-70-01-4868 Recorded Strain.	93
B2	I-70-01-4868 Concrete Surface Temperature	96
C1	I-65-54-4654 Concrete Surface Temperature	104
D1	I-74-136-4331 Concrete Surface Temperature.	114
E1	I-74-132-4328 Recorded Strain	124
E2	I-74-132-4328 Recorded Strain	125
E3	I-74-132-4328 Recorded Strain	126

LIST OF FIGURES

<u>Figure</u>		<u>Page</u>
1	Type of Bridge Under Study	2
2	Structural Distress at Supports.	3
3	Structural Distress at Supports.	4
4	Location of Some of the Problem Structures	5
5	Bridge Damage Characteristics.	7
6	I-94-41-4480 Moving Girders.	11
7	Structure I-94-41-4480 (Eastbound) Over Norris Ditch .	12
8	I-94-41-4480 Instrumentation	15
9	I-94-41-4480 Instrumentation	16
10	I-94-41-4480 Instrumentation	17
11	Gage Layout I-94-41-4480	18
12	Concrete Surface Preparation	19
13	Gage Installation.	20
14	I-94-41-4480 Instrumentation	21
15	Strain Measurement Apparatus	23
16	I-94-41-4480 Instrumentation	24
17	I-94-41-4480 Instrumentation	25
18	I-94-41-4480 Instrumentation	28
19	Deflection Measurement Apparatus	29
20	Electrical Schematics.	30
21	Deflection Measurement I-94-41-4480.	31

LIST OF FIGURES (Continued)

<u>Figure</u>		<u>Page</u>
22	Deflection Measurement I-94-41-4480.	32
23	Deflection Measurement I-94-40-4479.	33
24	Deflection Measurement I-94-40-4479.	34
25	Strain Plots I-94-41-4480.	47
26	Strain Plots I-94-41-4480.	48
27	Strain Plots I-94-41-4480.	49
28	Strain Plots I-94-41-4480.	50
29	Strain Plots I-94-41-4480.	51
30	Strain Plots I-94-41-4480.	52
31	Location of Girder Cracks I-94-41-4480	54
32	Dowel Bar Location at Pier 2	55
33	Conditions for Rotation Computation.	59
34	I-94-41-4480 Cross Section	60
35	I-70-01-4868 Cross Section	61
36	I-65-54-4654 Cross Section	62
37	I-94-41-4480 End Rotation.	67
38	I-94-41-4480 Transformed Cross Section	70
39	Traces of Moisture	73
<u>Appendices</u>		
<u>Figure</u>		
A1	Conditions for Computing Strain.	83
B1	I-70-01-4868 Instrumentation	87
B2	I-70-01-4868 Instrumentation	88
B3	I-70-01-4868 Instrumentation	89
B4	I-70-01-4868 Instrumentation	90
B5	I-70-01-4868 Instrumentation	91

LIST OF FIGURES (Continued)

<u>Figure</u>		<u>Page</u>
B6	Strain Plots I-70-01-4868.	94
B7	Strain Plots I-70-01-4868.	95
C1	I-65-54-4654 Instrumentation	98
C2	I-65-54-4654 Instrumentation	99
C3	I-65-54-4654 Instrumentation	100
C4	I-65-54-4654 Instrumentation	101
C5	Testing I-65-54-4654	103
D1	I-74-136-4331 Instrumentation.	106
D2	I-74-136-4331 Instrumentation.	107
D3	I-74-136-4331 Instrumentation.	108
D4	I-74-136-4331 Instrumentation.	109
D5	I-74-136-4331 Instrumentation.	110
D6	I-74-136-4331 Instrumentation.	111
D7	Testing I-74-136-4331.	112
E1	I-74-132-4328 Instrumentation.	116
E2	I-74-132-4328 Instrumentation.	117
E3	I-74-132-4328 Instrumentation.	118
E4	I-74-132-4328 Instrumentation.	119
E5	I-74-132-4328 Instrumentation.	120
E6	I-74-132-4328 Instrumentation.	122
E7	I-74-132-4328 Instrumentation.	123
E8	Strain Plot I-74-132-4328.	127
E9	Strain Plot I-74-132-4328.	128
E10	Strain Plot I-74-132-4328.	129

LIST OF FIGURES (Continued)

<u>Figure</u>		<u>Page</u>
E11	Strain Plot I-74-132-4328.	130
E12	Strain Plot I-74-132-4328.	131
E13	Strain Plot I-74-132-4328.	132
E14	Strain Plot I-74-132-4328.	133
E15	Strain Plot I-74-132-4328.	134

ABSTRACT

Black, Michael S., MSCE, Purdue University, May 1978. Reinforced Concrete Girder Bridge Study. Major Professors: M. J. Gutzwiller and R. H. Lee.

Personnel of the Indiana State Highway Commission, Bridge Division, contacted the structural staff of the Joint Highway Research Project concerning possible structural distress of approximately 60 multiple span reinforced concrete bridges. At the intermediate piers of these bridges, vertical deflection of the girders relative to the pier cap under vehicular load, spalling of the pier caps under the girder, and jamming of opposing girders were occurring.

These structures were instrumented in various ways to determine the failure mode. Girder strains were measured both electrically and mechanically. Concrete surface temperatures were measured on both the top deck and girders and deflections of the girders were measured, relative to the pier cap, both near the pier cap and also at midspan.

Data obtained from these measurements led to the conclusion that the vertical movement of the girders relative to the pier cap resulted from rotation of the girder ends induced by a temperature difference, between top deck and the bottom of the girder, of approximately 30°F. Pier cap spalling was the result of moisture seepage down through the expansion joints of these structures.

Recommendations included the extension of the diaphragms, at the intermediate piers, down to the top of the pier cap and the application of a waterproof membrane strip over each expansion joint with an overlay applied over the entire bridge deck.

CHAPTER 1

INTRODUCTION

Personnel of the Indiana State Highway Commission (ISHC), Bridge Division, contacted the structural staff of the Joint Highway Research Project (JHRP) concerning possible structural distress of approximately 60 multiple span reinforced concrete bridges (Figure 1). The problem with these structures is concentrated at the girder ends supported by the intermediate piers. At the intermediate piers, the portion of the pier cap directly underneath the girder is cracking and spalling. In some cases so much of the pier cap under the girder has spalled away, that reinforcing steel is exposed and bearing area is greatly reduced (Figure 2). In addition to spalling of the pier caps, adjacent girder ends suffer jamming together and cracking (Figure 3). This structural distress is further complicated by abnormal deflection of the girder ends relative to the pier cap under vehicular loads. This deflection under such loading is termed "bouncing" of the bridge.

Certain similarities characterize bridge damage. First of all, most of the damaged bridges are multiple span, simply supported, reinforced concrete bridges with the deck and girder cast monolithically. Other similarities are:

- 1) Damaged bridges are usually located on roadways with a high traffic volume coupled with a high percentage of truck traffic (Figure 4).

a) Three span concrete bridge

b) Three span concrete bridge

Figure 1. Type of bridge under study.

a) Spalling of pier cap and exposed rebar

b) Spalling of pier cap and exposed rebar

Figure 2. Structural distress at supports.

a) Cracking and jamming of girder ends

b) Cracking and jamming of girder ends

Figure 3. Structural distress at supports.

- 2) Tight approach pavement and structure joints causing joint filler material to be forced out of joint.
- 3) Spalling of concrete from sides of girders and piers.
- 4) Spalling of concrete from the ends of girders at the piers producing openings through the deck at girder ends.
- 5) A finely powdered mortar deposited on the face of the pier under girders where vertical movement of the end of the girder over its bearing area is taking place (Figure 5).
- 6) The vertical movements noted above are not only visible, but sometimes audible under passage of a heavy live load.
- 7) The top of one bridge deck over a pier will sometimes be significantly lower than that of the adjacent span.

Parameters Affecting Girder Movement

Although longitudinal movement is evident (Figure 3) in many of the girders of a particular structure, vertical movement is concentrated at the interior girders of the system. This girder movement (both vertical and longitudinal) is affected by various parameters. These parameters may include:

- 1) Temperature
- 2) Girder span length
- 3) Traffic volume
- 4) Percentage of truck traffic
- 5) Skew of the piers
- 6) Condition of approach slab
- 7) Moisture

b) Mortar coating deposited on pier cap face under moving girder

Figure 5. Bridge damage characteristics.

Among these parameters influencing girder movement, the most important is temperature. Temperature can contribute to girder movement in two ways. First, there is pavement growth due to high summer temperatures. This is coupled with the expansion and contraction of the girders themselves due to temperature change. Secondly, there is end rotation of the girders caused by the temperature difference between the top deck of the bridge and the girder bottom. This temperature difference can be appreciable when the top deck of a structure is exposed to direct sunlight and the girders underneath stay cool and shaded. This rotation is opposite to that of the live and dead loads on the girder and could cause "lifting off" at the girder ends if the rotation due to temperature overcomes the rotation due to dead load. The lifting off of the girder at the ends would reduce bearing area which would increase bearing stress, and when live load is applied, the girder might be slammed down to the pier cap causing possible deterioration and spalling.

Most of the other parameters listed above are self-explanatory as to how they affect girder movement with the exception of (6) and (7). Condition of the approach slab becomes critical to girder movement when settlement of this slab occurs. Soil settlement under the approach slab causes the slab to bow in a concave fashion. When vehicular traffic pass over this dip in the approach slab, the bridge deck then experiences bouncing live load which serves to increase the impact factor and possible "scooting" or longitudinal movement of the girders relative to the pier. Moisture and salt are always a problem on highway structures, especially when the structure is simply supported with

multiple spans. Seepage down through the expansion joints and underneath girder ends resting on the piers, combined with freeze-thaw cycles, can also make contributions to girder movement and pier cap distress.

Observation of the above list of parameters enables one to see how each element or combination of elements can have a detrimental effect on the bridge in the form of girder movement both horizontally and vertically.

Objective

The purpose of this study is to find the failure mechanism of these structures.

Scope

The several parameters affecting the failure mode of these bridges, makes the determination of the cause(s) of failure extremely difficult.

The structures experiencing distress are in various states of disrepair. Some of these bridges exhibit both excessive spalling of the pier cap and girder movement. Other bridges show very little (if any) spalling but are undergoing significant girder movement. This latter condition appears to be the incipient stage of the failure mode. The scope of this study will be confined to the incipient stage of failure as described above. If the failure mechanism is determined, study of this failure mode is most beneficial, since these relatively undamaged structures can be repaired before serious damage occurs. Seriously damaged structures will not be entirely neglected, however, some of

these bridges will be studied for the purpose of comparison.

Procedure

After discussing possible alternatives with the ISHC Bridge Department, it was decided to concentrate study on one particular structure and gather a limited amount of data on several other structures in various stages of distress. However, the structure under concentrated study had to be in the incipient mode of failure.

A bridge in the initial stages of failure was located with the cooperation of personnel of the ISHC. The structure is located on Interstate 94, eastbound, near Michigan City. Presently, this structure has four interior girders showing distress in the first span of the eastbound roadway as diagrammed in Figure 6. The girders numbered in Figure 6 are bouncing. Some of the characteristics of this structure are as follows:

- 1) Structure number I-94-41-4480
- 2) Three-span structure, 8 girders in each span
- 3) Each girder is simply supported, span length 29'-1"
- 4) Skew of the girders is 30° left
- 5) Structure was built in 1971

This structure is the subject of intense study and is shown in Figure 7.

Information sought concerning structural behavior took various forms,

- 1) Structure behavior under live load (did it conform to bending theory?)

EASTBOUND STRUCTURE

SECTION A-A

Figure 6. I-94-41-4480 moving girders.

Figure 7. Structure I-94-41-4480 (eastbound) over Norris Ditch.

- 2) Structure behavior under a temperature difference from top to bottom
- 3) Vertical girder deflection, both at the pier cap and girder center line
- 4) Horizontal girder deflection at the pier cap
- 5) Insight into girder end cracking

A reliable and accurate method of measuring girder strains, temperatures, and deflections was required to obtain the information described above.

CHAPTER 2

INSTRUMENTATION

Strain Measurement

Instrumentation of structure I-94-41-4480 consisted of three phases. The first of these involved strain measurement. Strain was measured both with electrical-resistance strain gages and the mechanical (Whittemore type) gage. Electrical strain measurement was taken on the top deck and the numbered girders of Figure 6. Mechanical strain measurement was taken at the top and bottom of each moving girder in Figure 6. Gage locations are shown in Figures 8, 9 and 10. Strains were measured at the girder center line because it would provide a convenient check point and be a good barometer of girder behavior. Both electrical and mechanical strain measurements were taken on several girders at the same gage line for comparison (Figure 11).

The electrical-resistance strain gages used were of the foil type and were temperature compensating. The gage length was 4 inches. The procedure for applying this type of gage on a concrete surface is quite lengthy and is best presented in the step-by-step form that follows:

- 1) Construct alignment marks on the concrete surface where gages are to be placed.
- 2) Grind the concrete surface within the alignment marks in both the longitudinal and transverse direction.
- 3) Clean the surface with acetone and gauze.

●=STRAIN GAGE
 ▲=TEMP. SENSOR

Figure 8. I-94-41-4430 instrumentation.

Figure 9. I-94-41-4480 instrumentation.

Figure 10. I-94-41-4480 instrumentation.

Figure 11. Gage layout I-94-41-4480.

b) Gage seat complete with clamp-tabs and Whittemore points

Figure 12. Concrete surface preparation.

a) Gages in place and clamped

b) Final assembly with leadwires attached

Figure 13. Gage installation.

a) Instrumentation set-up

b) Instrumentation set-up

Figure 14. I-94-41-4480 instrumentation.

- 4) Apply a coating of plastic resin epoxy to concrete surface sealing all voids and cracks (temperature must be greater than 60° F).
- 5) Allow 24 hours for epoxy to cure.
- 6) Repeat steps (4) and (5) if concrete surface is extremely rough.
- 7) Hand sand gage seat lightly.
- 8) Clean seat with acetone and gauze.
- 9) Reconstruct gage alignment marks.
- 10) Position strain gage (or temperature sensor) and solder tab on mylar tape.
- 11) Apply epoxy (same as in (4)) to back of gage and solder tab.
- 12) Position gage over alignment marks and place teflon and rubber cushion over gage, respectively.
- 13) Clamp gage to surface with a pressure of 1 psi and allow epoxy to cure for 24 hours.
- 14) Place solder dots on gage and tab.
- 15) Prepare wire leads and solder them to the gage.
- 16) Check continuity of connection with an ohmmeter.
- 17) Spray assembly with protective plastic film.

Figures 12 and 13 illustrate various steps of the concrete surface preparation. Figure 14 shows the electrical-strain test procedure for I-94-41-4480.

The other method of strain measurement consisted of mounting small metal tabs on the concrete surface (Figures 11 and 12). The metal tabs were 1 inch by 1 inch and had a small diameter hole drilled

a) Battery powered strain indicator and switch and balance unit (10" Whittemore gage in foreground)

b) 5" and 10" Whittemore gages with respective spacer bars

Figure 15. Strain measurement apparatus.

a) Girder 6 at pier 2

b) Close-up showing 5" Whittemore points

Figure 16. I-94-41-4480 instrumentation.

a) Top deck installation

b) Girder installation

Figure 17. I-94-41-4480 instrumentation.

in the middle. The metal tabs were set at a standard gage length of 10 inches with a spacer bar and attached to the concrete surface with epoxy. Figure 15 shows the Whittemore gage used for mechanical strain measurement as well as the apparatus used for electrical strain measurement.

Associated with strain measurement was the monitoring of crack expansion or contraction. The four bouncing girders of Figure 6 all had cracks at the ends resting on pier 2. A Whittemore tab was placed on each side of the crack on a 5-inch gage length. Crack movement was monitored with the use of a 5-inch Whittemore gage for all four girders. In Figure 16 the crack monitoring system is shown for a typical girder.

Temperature Measurement

The temperature sensors were also of the electrical-resistance type and were bonded to the concrete surface in the same manner as the electrical-resistance strain gages. The procedure for installing the temperature sensors was identical to that presented in the previous section on Strain Measurement for the strain gages. Temperature sensor location is shown in Figures 8 and 9. One important difference in temperature sensor installation was that of insulating the sensor from direct sunlight and ambient temperature. This was accomplished by placing a piece of sponge rubber over the gage and then taping the rubber to the surface of the concrete as shown in Figure 17.

Deflection Measurement

The final phase of the instrumentation system consisted of deflection measurements. A first attempt was made using the dial extensometer as shown in Figure 18 to measure vertical deflection at midspan. Since only a fraction of a second was available to read the extensometer, this system made it extremely difficult to obtain accurate, reliable deflection data due to live loads. Therefore this system was abandoned for a system using a linear potentiometer. The potentiometer and its associated hardware is shown in Figure 19. The advantages of this latter system over the former are that it is more accurate and provides a continuous readout with a strip chart recorder.

The principle behind the linear potentiometer is based on the direct relationship between a change in voltage of the potentiometer as its probe shortens or lengthens. It is nothing more than an electrical analog of the force-spring system ($F = KD$). A schematic of the linear potentiometer circuit is shown in Figure 20.

To obtain girder deflection the potentiometer probe was attached to the girder by means of a universal joint connecting the probe to a stud epoxyed to the girder. This allowed the probe to move with the girder as it moved. The purpose of the universal joint was to protect the probe from damage or breakage. This was especially important in the horizontal deflection measurement.

Deflection measurements using the linear potentiometer were taken on girders 1, 3 and 4 of Figure 6, both near pier 2 and at midspan. Horizontal and vertical deflection was measured relative to the pier cap at pier 2 while vertical deflection only was measured at midspan.

a) Deflection measurement

b) Close-up of dial gage
Figure 18. I-94-41-4480 instrumentation.

Figure 19. Deflection measurement apparatus. (Battery and electrical inverter, potentiometer, strip chart recorder, circuit box, digital volt-ohmmeter, left to right.)

a) Linear potentiometer circuit

b) Strain gage circuit

Figure 20. Electrical schematics.

Vertical deflection measurement at pier cap (Girder 4)

Vertical deflection measurement at midspan (Girder 1)
21. Deflection measurement I-94-41-4480.

a) Vertical deflection measurement at the diaphragm between girders 4 and 5

b) Horizontal deflection measurement at pier cap (Girder 4)
Figure 22. Deflection measurement I-94-41-4480.

a) Vertical deflection measurement at pier cap (Girder 4)

b) Vertical deflection measurement at midspan (Girder 4)
Figure 23. Deflection measurement I-94-40-4479.

a) Horizontal deflection measurement at pier cap (Girder 4)

b) Close-up of potentiometer (Girder 4)

Figure 24. Deflection measurement I-94-40-4479.

Vertical deflection of the diaphragm between girders 4 and 5, at pier 2, was also measured. Figures 21 and 22 show the deflection apparatus set up at the bridge site in various locations. For comparison purposes, deflection measurements were taken on another bridge. The structure chosen was I-94-40-4479 in close proximity to Structure I-94-41-4480; however, the girders of the former structure are presently not bouncing. Deflection measurements were taken on this structure for girder 4 (corresponding to girder 4 of I-94-41-4480) and the same locations (i.e., at the pier cap, horizontal and vertical, and at midspan). Figures 23 and 24 illustrate test set-up for the deflection measurements taken at Structure I-94-40-4479.

Measurement Considerations

With any instrumentation system, consideration must be given to various possible sources of error. First, consideration is given to the electrical-resistance strain gages. All of the strain gages used were in the active leg of a quarter-bridge Wheatstone bridge circuit. The other three legs of the bridge circuit were located within the strain indicator (Figure 14). It follows that the lead wires for the active gage were very long in comparison with the other legs of the bridge located in the strain indicator. The long exposed lead wires were more susceptible to temperature variations during the testing procedure. This situation can cause errors in observed strains. This error can be minimized by using the three-wire lead system shown in Figure 20. Another problem associated with the use of long lead wires is the error induced by the resistance of the lead wire itself. When the lead wire depends on the overall length and the American Wire Gage

(AWG) number. The instrumentation wire used was AWG 26 with a resistance of approximately 41-ohms per thousand feet. Lead lengths rarely were in excess of 100 feet and compared with the nominal resistance of the gages used (120-ohms $\pm$ .5%), this error (known as the desensitization error) was considered negligible.

The type of leadwire described above was also used for the temperature sensors. Leadwires were handled in the same way as the strain gage leadwires, with one significant difference. The three-wire system used with strain gages to eliminate temperature-induced resistance changes in the leadwires is not effective with the sensors. Changes in leadwire temperature are normally a minor source of error. A change of 50° F over the entire length of a 0.5-ohm copper leadwire circuit will create an indicated error of approximately 0.4° F when the sensor is near 75° F. This error decreases at higher sensor temperatures and increases at lower sensor temperatures. A two-wire lead was used for the temperature sensors since it was adequate and easier to attach than the three-wire lead. As was the case with the strain gage lead, the temperature sensor leadwire resistance is a possible source of error. The sensor leadwires were cut all the same length before hook-up (rarely longer than 25 feet) and the resistance of the leadwire recorded. Resistances from the sensors were then recorded during the test and adjusted to take into account leadwire resistance before calculating temperatures which were known functions of sensor resistance.

The strain recording system (Figure 15) was battery powered and significant error can occur with battery decay. What actually happens

to the strain indicator with time is that the voltage change of the battery causes the null or zero position of the bridge circuit to move or drift. If this zero drift is known, the strain readings can be adjusted accordingly. There are many methods for approximating zero drift of the strain indicator. The two used in this study were:

- 1) Attaching a strain gage to a part of the structure where strain will be zero (e.g. a bridge pier). Any reading on the indicator from this gage is assumed to be zero drift.
- 2) Take a reading on the active gage and then reverse the active and dummy gage and record another reading from the indicator. The readings should be the same but opposite in sign. If they are not, the difference is taken as zero drift of the indicator.

Measurement considerations concerning mechanical strain measurement consisted solely of temperature compensation. The Whittemore tabs were susceptible to contraction and expansion of the concrete due to temperature variation. A standard reading was recorded on a standard bar at a set temperature with the Whittemore gage. All readings that followed were then adjusted to this standard reading. A sample calculation for the Whittemore reading adjustment appears in Appendix A.

There are two points to consider in regard to deflection measurement. The first consideration involves making the frame holding the linear potentiometer rigid enough to allow no movement except the potentiometer probe itself. With this accomplished, the other consideration is the voltage source of the potentiometer circuit.

Mercury batteries were chosen to provide the source of voltage in the circuit because of their unique characteristic of constant output during their useful life. Since the voltage does not decay with respect to time, the circuit undergoes no variations which could lead to measurement error. The major source of error in the deflection measurement consisted of the play evident in the universal joint. The magnitude of this error was .001 inches and measured deflections were adjusted accordingly.

CHAPTER 3

STRUCTURE I-94-41-4480 DATA

General

Electrical strain measurements were taken on Structure I-94-41-4480 over a three day period during the month of September 1977. Strain was recorded intermittently throughout the test period in the manner presented in Chapter 2. For the most part, strain data were recorded on an hourly basis throughout the day until late evening when readings were terminated until the following morning. The test set-up appeared in Figure 14 of Chapter 2. The leadwires of all the strain gages remained attached to the strain indicator during the entire test period. Table 1 represents the tabulation of electrical strain measurement. All readings are in micro-inches-per-inch and have been adjusted for zero drift of the indicator. The values in Table 1 are all relative to the initial reading (i.e., the initial reading was taken as zero).

Mechanical strain measurement was undertaken during this time also. The Whittemore readings were taken in between the recording of electrical strain measurements. Whittemore data for I-94-41-4480 (10-inch gage) is presented in Table 2. Again all readings are in micro-inches-per-inch relative to an initial reading (not shown). The Whittemore readings have been adjusted for temperature and the method of correction is presented in Appendix A.

Table 1. I-94-41-4480 Recorded Strain

DATE TIME 8		TOP DECK		GIRDER 4		GIRDER 5		GIRDER 6		GIRDER 7	
SEPT. 7				T	B	T	B	T	B	T	B
12:00 p.m.	-6	+4	+10	+3	+3	+8	-4	+10	0		
12:45 p.m.	-10	+12	+22	+10	+3	+14	-4	+10	-4		
1:30 p.m.	-10	+30	+10	+10	-4	+10	-10	+6	-12		
3:00 p.m.	-6	+14	+62	+4	-5	+9	-9	+6	-8		
4:00 p.m.	+4	+5	+76	0	-8	+1	-5	-8	-8		
5:10 p.m.	+22	0	+125	-8	-7	-9	+4	-20	+10		
6:09 p.m.	+28	-2	+155	-20	-12	-18	-3	-30	+10		
7:10 p.m.	+14	-22	+164	-40	-25	-37	-15	-51	+2		
8:00 p.m.	+17	-27	+184	-32	-16	-32	-4	-50	+14		
9:00 p.m.	+22	-20	+214	-32	-14	-32	-4	-48	+17		
10:15 p.m.	+21	-20	+248	-31	-13	-33	-2	-45	+20		
SEPT. 8											
12:10 a.m.	+8	-29	+278	-38	-19	-40	0	-56	+21		
8:00 a.m.	+6	-62	+565	-43	-40	-66	+4	-82	+10		
10:00 a.m.	-25	-64	+623	-43	-45	-65	-10	-81	-9		
11:15 a.m.	-27	-68	+626	-48	-51	-67	-18	-78	-18		
12:10 p.m.	-28	-52	+646	-38	-42	-55	-20	-64	-23		
12:45 p.m.	-55	-60	+626	-46	-53	-58	-38	-68	-43		
2:00 p.m.	-70	-54	+625	-44	-56	-55	-50	-63	-58		
3:30 p.m.	-72	-54	+632	-40	-50	-42	-46	-46	-50		
4:30 p.m.	-54	-42	+650	-47	-51	-48	-48	-50	-50		
5:30 p.m.	-52	-54	+652	-62	-60	-60	-55	-64	-54		
6:30 p.m.	-34	-56	+669	-66	-59	-66	-49	-72	-44		
7:30 p.m.	-24	-76	+675	-72	-71	-82	-28	-82	-31		
8:15 p.m.	-14	-62	+698	-68	-60	-78	-38	-88	-29		
9:00 p.m.	-22	-68	+704	-74	-63	-82	-44	-92	-30		
10:00 p.m.	-22	-71	+711	-80	-68	-87	-47	-100	-33		
11:00 p.m.	-26	-72	+725	-81	-66	-90	-53	-104	-36		
SEPT. 9											
6:00 a.m.	-28	-79	+821	-90	-72	-103	-48	-112	-26		
8:00 a.m.	-45	-84	+837	-97	-86	-112	-68	-113	-45		
9:00 a.m.	-58	-88	+828	-94	-84	-104	-73	-111	-56		
10:00 a.m.	-80	-100	+822	-94	-91	-102	-80	-111	-68		
11:00 a.m.	-84	-93	+828	-87	-90	-96	-84	-104	-78		
11:30 a.m.	-151	-170	+760	-160	-156	-161	-144	-166	-136		
12:00 p.m.	-123	-121	+806	-118	-121	-130	-115	-135	-98		
12:30 p.m.	-120	-114	+813	-111	-114	-119	-103	-124	-92		
1:00 p.m.	-92	-105	+824	-107	-104	-110	-98	-115	-89		

Table 2. I-94-41-4480 Recorded Strain (10" Whittemore)

DATE TIME &	GIRDER 4		GIRDER 5		GIRDER 6		GIRDER 7	
SEPT. 2	T	B	T	B	T	B	T	B
1:00 pm	-20	-70	+30	-50	+30	-40	+20	-10
SEPT. 6								
4:45 pm	-6	-126	+54	-66	+44	-36	+44	-6
SEPT. 7								
12:30 pm	-17	-86	+43	-56	-9	-50	+13	-30
4:00 pm	+18	-67	+51	-57	+38	-10	+16	0
7:45 pm	+10	-67	0	-26	+17	+40	+7	+10
SEPT. 8								
12:00 noon	-45	-36	-45	-33	-37	+13	-17	+7
4:30 pm	-17	-73	+23	-55	-25	+40	+49	0
7:30 pm	+11	-75	+11	-36	+38	+49	+58	+30
SEPT. 9								
6:00 am	+13	-66	+13	-16	+20	+51	+40	+31
11:15 am	+3	-56	+13	-36	+41	+61	+51	+11

ALL READINGS $\times 10^{-6}$ (MICRO-STRAIN)

Temperature data were also taken during this time period and these are tabulated in Table 3. The temperatures recorded in this table are in degrees fahrenheit and represent the temperature of the concrete surface at the locations shown in Figures 8 and 9 of Chapter 2. The maximum thermal difference (between top deck and bottom girder) recorded in Table 3 was 25.6°F. A typical temperature calculation from sensor resistance data is explained in Appendix A. Ambient temperatures during this test period averaged 73°F underneath the bridge and 71°F near the bridge.

Crack growth (see Chapter 2) was also monitored during this test period using the 5-inch Whittemore gage. This data is presented in Table 4. All measurements are in micro-inches-per-inch and are relative to an initial reading (not shown). Temperature adjustments were made in the same manner as for the 10-inch Whittemore data.

Live load deflection was not taken during the September testing but was gathered in late November-early December, 1977. This data was taken as described in Chapter 2, on a continuous basis, using the strip chart recorder. Center-line deflections were monitored over a 30-minute period while movement near the pier cap was monitored for a 1 to 2-hour period, both horizontally and vertically. Results are tabulated in Table 5 for Structure I-94-41-4480 and in Table 6 for Structure I-94-40-4479. All measurements are in inches and the readings have been adjusted for the slight deflection of the universal joint. It should be noted that the location of girder 4 in Structure I-94-41-4480 corresponds exactly to the location of girder 4 in Structure I-94-40-4479.

Table 3. I-94-41-4480 Concrete Surface Temperature

DATE TIME &	TOP DECK	GIRDER 4		GIRDER 7	
SEPT. 7		T	B	T	B
10:30 a.m.	59.3	52.4	51.0	57.9	57.2
12:00 p.m.	67.4	54.5	51.7	59.3	58.6
1:45 p.m.	72.3	55.9	46.7	61.4	60.7
3:45 p.m.	73.6	57.9	53.1	63.5	60.0
6:15 p.m.	66.1	58.6	53.1	64.8	60.0
8:05 p.m.	61.4	59.3	52.4	64.1	58.6
SEPT. 8					
12:10 a.m.	56.6	57.2	52.4	62.8	57.2
8:00 a.m.	44.8	48.2	46.8	53.8	52.4
11:20 a.m.	48.9	46.8	45.4	52.4	51.7
12:45 p.m.	58.6	49.6	47.5	54.5	54.5
2:00 p.m.	67.4	51.7	48.9	56.6	55.9
4:30 p.m.	71.6	54.5	51.	60.7	58.6
6:30 p.m.	67.5	57.2	51.7	63.5	59.3
8:00 p.m.	62.8	57.9	52.4	63.5	59.3
9:00 p.m.	60.7	57.9	52.4	63.5	59.3
SEPT. 9					
6:00 a.m.	50.3	53.8	51.7	59.3	57.2
10:00 a.m.	54.5	53.1	51.7	57.9	57.9

ALL READINGS IN DEGREES FAHRENHEIT

Table 4. I-94-41-4480 Average Strain Across Cracks (5" Whittemore)

DATE & TIME	GIRDER 4	GIRDER 5	GIRDER 6	GIRDER 7
SEPT. 2				
1:05 pm	-60	-40	-100	-60
SEPT. 6				
5:00 pm	+48	+28	-72	+8
SEPT. 7				
12:15 pm	-14	+26	-100	-40
3:45 pm	+4	+1264	-62	-42
6:15 pm	+4	+1364	-64	-44
SEPT. 8				
12:10 am	-14	+1286	-60	-60
12:00 noon	-128	+1272	-54	-34
4:30 pm	-72	+1288	-80	-40
8:00 pm	-96	+1304	-102	-62
SEPT. 9				
6:00 am	-116	+1224	-58	-58
11:30 am	-96	+1244	-58	-58
OCT. 22				
11:00 am	-136	+1244	-136	-36

ALL READINGS $\times 10^{-6}$ (MICRO-STRAIN)

Table 5. I-94-41-4480 Recorded Deflections

GIRDER 1			GIRDER 3			GIRDER 4		
PIER CAP		CENT. LINE	PIER CAP		CENT. LINE	PIER CAP		CENT. LINE
V	H		V	H		V	H	
0	none mea- sured	0.006	0.004	none mea- sured	0.062	0.013	none mea- sured	0.048
VERT. DIAPHRAGM DEFLECTION (BET. GIRDS. 4 & 5) = 0.007								

Table 6. I-94-40-4479 Recorded Deflections

GIRDER 4		
PIER CAP		CENT. LINE
V	H	
0	none mea- sured	0.047

V = VERTICAL

H = HORIZONTAL

ALL MEASUREMENTS IN INCHES

Additional strain and temperature data, for various structures, appear in Appendices B through E.

Analysis and Interpretation of Strain Data

Plots of the strain data in Tables 1 and 2 are shown in Figures 25 through 30. The plots show the strain recorded from the girders as a function of time. Solid lines indicate the electrical-resistance strain gage data (Table 1). Dotted lines indicate the Whittemore strain data (Table 2). Strain at the top and bottom fiber of a typical I-94-41-4480 interior girder cross section was calculated. The value of strain, theoretically, for the top fiber was 55 micro-inches-per-inch in compression. The bottom fiber strain was calculated to be 122 micro-inches-per-inch in tension. These computations were based on bending theory due to HS-20 live load conditions only, and the method of calculating the above strain is presented in Appendix A. Tabulated values of the top deck strain (Table 1) would correspond with the theoretical top fiber computation. In this respect, correlation is quite good. Correlation of the strain data for the top of the girders in Tables 1 and 2 is too high according to bending theory predictions, since this location is near the neutral axis and strains should be close to zero. The data of Table 1 also make a decisive "jump" between 11:00 and 11:30 a.m. on September 9, for no apparent reason. The unusual behavior coupled with the large compressive strains along the bottom fiber (Tables 1 and 2) leads to the belief that the girders must be experiencing axial loading at the ends in the form of opposing girders possibly jamming together.

Figure 25. Strain plot I-94-41-4480.

Figure 26. Strain plot I-94-41-4480.

Figure 27. Strain plot I-94-41-4480.

Figure 28. Strain plot I-94-41-4480.

Figure 29. Strain plot I-94-41-4480.

Figure 30. Strain plot I-94-41-4480.

Analysis and Interpretation of Crack Data

The location of girder cracks is shown in Figure 31. The distance to the crack is measured from the pier 2 end along the length of the girder. Crack movement was monitored as described in Chapter 2 and the results are presented in Table 4. The strains of this table differ from those of Table 2 in that Table 4 strains were based on a 5-inch gage length while Table 2 strains were based on a 10-inch gage length. Observation of Table 4 leads to the conclusion that the cracks seem to be contracting in the case of girders 4, 6 and 7 while the girder 5 crack is expanding. No explanation is offered for these phenomena but it was interesting to note that three of the four girders involved exhibited movement in the direction of traffic (Figure 31). This leads to the conclusion that longitudinal girder movement might be related to the direction of traffic flow. Also of note is the relationship between the crack location and the dowel bar(s) at the fixed ends (piers 2 and 3). Figure 32 shows a diagram of this detail made from the I-94-41-4480 drawings. All measurements shown in Figure 32 are along the length of the girder. It should also be noted that Figure 32 does not represent a complete steel detail of the girder. The purpose of Figure 32 is to compare dowel bar location to crack location. From comparison of Figures 31 and 32, no obvious conclusions were evident, except that the crack location does fall in the general location of the dowel bar.

Analysis and Interpretation of Deflection Data

The theoretical deflection of a typical I-94-41-4480 interior girder, at the point of maximum moment, due to HS-20 loading, was

Figure 31. Location of girder cracks I-94-41-4480.

Figure 32. Dowel bar location at pier 2.

calculated to be 0.054 inches. The explanation and procedure of the computation is presented in Appendix A. Comparing the theoretical deflection to the center-line deflection of Table 5 the results are quite good.

CHAPTER 4

THE FAILURE MECHANISM

General

Examination of the end rotations at the girder ends leads to insight into the failure mechanism of the girders. This can be accomplished by comparing end rotations caused by dead load, live load, and temperature.

Dead Load Rotation

Consider the vertical distance between point B on the deflection curve shown in Figure 33 for a simply supported girder and the tangent drawn to this curve at point A. This vertical distance is denoted by Δ in the diagram. For an elastic structure, Δ is $wL^4/24EI$, giving the tangential rotation θ_A as $wL^3/24EI$ (for small deflection theory) in a clockwise direction. The rotation at end B will be equal to that at end A (due to symmetry) but opposite in sign.

Thermal Rotation

When the sun shines on the top deck of a bridge, the top deck undergoes expansion while the bottoms of the girders stay relatively cool. As seen from the temperature data, this temperature difference may be as high as 30°F. Under thermal expansion, the strain diagram for a typical girder cross section will be as shown in Figure 33 where h is the depth, α is the coefficient of linear expansion (5.5×10^{-6} in/°F),

a) Second moment area theorem

b) Strain diagram for temperature difference

c) Diagram for virtual work expression

Figure 33. Conditions for rotation computation.

and ΔT is the temperature difference from top to bottom. The tangent of angle ϕ is equal to:

$$\frac{\alpha \Delta T}{h}$$

and from small angle geometry $\tan \phi$ is approximately equal to ϕ , therefore:

$$\phi = \frac{\alpha \Delta T}{h}$$

The angle of the strain diagram (Figure 33), ϕ , is equal to curvature and from this relationship ($\phi = -M/EI$) a substitution can be made yielding:

$$\frac{-M}{EI} = \frac{\alpha \Delta T}{h}$$

or, the moment induced due to the temperature difference would be:

$$M = \frac{-\alpha \Delta T EI}{h}$$

The end rotation can be obtained by employing the virtual work expression:

$$\theta_A = \int \frac{Mm}{EI} ds$$

The expression for M has already been established and the expression for m can be determined from Figure 33. A unit rotation is employed at end A of Figure 33 in the direction shown. The expression for m , at any point is then:

$$m = \frac{x}{L} dx$$

Figure 34. I-94-41-4480 cross section.

Figure 35. I-70-01-4868 cross section.

Figure 36. I-65-54-4654 cross section.

The virtual work expression becomes:

$$\theta_A = \int \frac{-\alpha \Delta T EI}{hL EI} x dx$$

The above expression, after simplifying and integrating with respect to x, yields:

$$\theta_A = \left[\frac{-\alpha \Delta T}{2hL} x^2 \right]_0^L$$

Substituting the limits for x (from 0 to L) leaves:

$$\theta_A = - \frac{\alpha \Delta T L}{2h}$$

The negative sign denotes the end rotation at A is opposite to the direction of the unit rotation assumed, or counter-clockwise.

End Rotation

By employing the methods developed in the preceding sections, rotations were computed for three of the structures tested. These structures were I-94-41-4480, I-70-01-4868, and I-65-54-4654, of span lengths 29.1, 33.8, and 40 feet respectively. The end rotations were readily computed for these structures because drawings were on hand. The cross sections used for the moment of inertia computation appear in Figures 34, 35 and 36. All lengths were taken from the respective drawings for each structure. The figures also show how each section was divided into individual areas (dotted lines) for the moment of inertia of the composite section. It should be noted that the curbing monolithic with the structure was included in the moment of inertia computation. Curb extension, sidewalls, etc., were not included in

the moment of inertia consideration since, in most cases, these extensions are not monolithic with the structure; however, the extensions were included in the dead load computation with the exception of the guardrail. The gross moment of inertia was computed for each cross section in Figures 34, 35, and 36. Moment of inertia of the gross section was used because of the requirement in Section 909(c) of ACI 318-63 ($\rho f_y \leq 500$ psi). Moments of inertia, dead loads, and accompanying end rotations appear in Table 7. Structures in Table 7 are arranged by their span lengths, the shortest span first. Temperature rotations also appear in this table and are based on a 30°F temperature difference between top deck and bottom girder. The last column of Table 7 represents the percentage difference between $\theta_{d.l.}$ and θ_{temp} relative to θ_{temp} . The most important aspects of Table 7 are that the temperature rotation is greater than dead load rotation and temperature rotation and dead load rotation approach equality as the span length increases.

Of primary concern, however, is whether the end rotations can initiate enough movement, relative to the pier cap, to cause the vertical deflections of the girder, at the pier cap, in Table 5, Chapter 3. In Figure 37, a diagram of a typical I-94-41-4480 girder, at the pier cap, is shown. The rotations shown in Figure 37 have been greatly exaggerated for clarity. The horizontal measurement (1'-5.3") represents the length of the girder (measured along the girder) that rests on the pier cap. Assuming unrestrained conditions, (i.e., no dowel bars between girder and pier cap) the girder was free to rotate through angle $\theta_{d.l.}$ after construction of the bridge was completed and

Table 7. End Rotation of Various Structures

STRUCTURE	NEUTRAL AXIS DEPTH (from girder bottom) (inches)	DEPTH OF SECTION h (inches)	DEAD LOAD w lb/in	MOMENT OF INERTIA (inches ⁴)	DEAD LOAD ROTATION (radians)	THERMAL ROTATION (radians)	$\frac{\theta_{TEMP} - \theta_{D.L.}}{\theta_{TEMP.}} \times 100 \%$
I-94-41-4480	24.72	35.4	789.8	1008323	0.00042	0.00081	48.1
I-70-01-4868	21.96	33.0	782.7	988046	0.00066	0.00101	34.7
I-65-54-4654	23.88	34.0	764.2	1094223	0.00101	0.00116	12.9

shoring removed. This rotation was accommodated because of the Celotex or tar paper between the girder and the pier cap or because of the build-up of the girder soffit. During hot weather, thermal differences between top deck and bottom girder occurred and the girder rotated through angle θ_{temp} . Using the values computed in Table 7 the difference between these two rotations, $\Delta\theta$, amounts to 0.00039 radians for Structure I-94-41-4480. Applying small angle geometry ($\tan\theta \approx \theta$), Δ (Figure 37) is equivalent to 0.007 inches. Now Δ corresponds to the vertical deflection at the pier cap face due to $\Delta\theta$ (difference between θ_{temp} and $\theta_{d.l.}$). Comparing this deflection to those obtained by the linear potentiometer in Table 5, Chapter 3, a good correlation is established. In other words the vertical deflection relative to the pier cap of the girder, induced by the temperature difference, generally correspond to those same deflections obtained experimentally.

Another interesting comparison is rotation associated with the live load. The live load rotation was obtained by the conjugate beam approach. The procedure is presented in Appendix A. Since the location of the particular loading involved was asymmetrical, Appendix A, Figure A1, rotations were not equal. These rotations were 0.00053 and 0.00049 radians and are in the same direction as the dead load rotations. In either case, the live load rotation is sufficient to overcome $\Delta\theta$ caused by the temperature rotation. It follows then, that during periods of hot weather, when the temperature difference between top deck and bottom girder is approximately 30°F, these short span girders are lifting off slightly and then are being hammered back down to the pier cap by the live loads produced from truck traffic. So

Figure 37. I-94-41-4480 end rotation.

in this way, the observable girder movement, at the pier cap, is generated.

Transformed Section

Although code specifications (AASHO, Section 1.5.2(8), ACI 318-62, Section 909(c)) advise use of the gross concrete section in computations, it is interesting to look at the behavior of the transformed section as it applies to moment of inertia computations. The moment of inertia of the transformed section of Structure I-94-41-4480 was computed. First of all, the cracking moment for I-94-41-4480 was calculated and compared with dead and live load moments. The cracking moment as defined by ACI 318-71, Section 9.5.2.2 is:

$$M_{cr} = \frac{f_r I_g}{y_t} \quad \text{where } f_r = 7.5\sqrt{f'_c}$$

Using the value for f'_c (3000 psi) from previous computations (Appendix A), and the gross moment of inertia for I-94-41-4480 (Table 7):

$$M_{cr} = \frac{(7.5\sqrt{3000})(1008323)}{(24.7)(1000)} = 16778 \text{ in-kips}$$

The dead load moment for the I-94-41-4480 cross section is:

$$M_{d.l.} = \frac{wL^2}{8} = \frac{(789.8)(349)^2}{8(1000)} = 12025 \text{ in-kips}$$

So it is seen that the dead load moment is less than the cracking moment. The live load moment should also be checked. From Appendix A the maximum live load moment for an I-94-41-4480 interior girder was computed to be 1947 in-kips. For comparison, the cracking moment for an I-94-41-4480 interior girder is:

$$M_{cr} = \frac{(7.5\sqrt{3000})(116951)}{(2.03)(12)(1000)} = 1973 \text{ in-kips}$$

The moment of inertia and centroidal distance that appear in the equation above were computed previously (Appendix A, Figure A1). Although the live load and dead load moments individually do not exceed the cracking moment, the total moment of the combination of the two loading conditions will exceed the cracking moment, so it would be advantageous to use the transformed section for moment of inertia calculation for Structure I-94-41-4480. The transformed I-94-41-4480 section appears in Figure 38. The transformed steel area consisted of the 8-#11 rebars that made up the positive moment steel in the girder bottom. The modular ratio, n , used in transformation was based on 29×10^6 psi for the steel elastic modulus and 3320558 psi (computed in Appendix A) for the concrete elastic modulus. The modular ratio (E_s/E_c) is therefore $29000000/3320558$, or 8.73, say 9. AASHTO (1969 ed. Section 1.5.2(4)) advises using a modular ratio of 10 for computations of strength and 8 for computations of deflection, so the value of 9 lies within that range. The distance to the neutral axis from the top of the slab (C in Figure 38) was 7.45 inches. This means that the neutral axis is in the slab and the corresponding moment of inertia of the I-94-41-4480 transformed section was computed to be 717407 in^4 . To find the corresponding dead load rotation, the data in Table 7 was used and $\theta_{d.l.}$ was 0.00059 radians. This represents a percentage difference (between $\theta_{d.l.}$ and θ_{temp} relative to θ_{temp} of Table 7) of 27.2% for I-94-41-4480. Although the dead load rotation is increased, it still is overcome by the temperature rotation and the problem of

Figure 38. 1-94-41-4480 transformed cross section.

vertical girder movement still exists (to a lesser degree). Actual vertical deflection from the rotation difference, Δ , equals $(.00081 - .00059)(17.3)" = .004$ inches.

Girder End Cracking

The dowel bars used at the fixed end to tie the girder ends to the pier cap must also resist the temperature rotation just described in the previous section. The deflection due to the thermal rotation causes appreciable deflection of the girder at the pier cap face as seen in Figure 37. Since the girder is restrained from rotation due to temperature by the dowel bars, strains are initiated throughout the girder end. Of course these strains become concentrated at the points of restraint (dowel bars). The girder cracks fall in the range of the dowel bar as witnessed by Figures 31 and 32. The cracks lie between the points of restraint of the dowel bar in the region of the "U" formed by the dowel. This leads to the belief that the strain in the concrete, initiated by the thermal rotation, is exceeded in this region causing the cracks.

Pier Cap Spalling

The majority of the observed pier cap damage occurred at the fixed ends of the spans. The dowel bars (Figure 32) were placed at these ends to tie the girder to the pier cap. In some of the damaged structures observed, these dowel bars were exposed (Figure 2). Much of the damage to the pier cap is brought about by moisture aided by girder movement. Moisture combined with salt (in winter months) seeps down through expansion joints and underneath the girder and slowly drains. From observation of various structures, the draining water

leaves traces of mud, silt, and salt on the pier cap face once the water evaporates (Figure 39). Of course if the girder is moving vertically at the pier cap, this movement permits the water mixture to seep underneath the girders more freely. The dowel bars again become the source of trouble. Once the moisture reaches the underside of the girder it thus makes contact with the dowel and initiates corrosion. When corrosion is underway, expansion of the dowel bar occurs, resulting in concrete spalling around the area of the dowel bar. The above phenomenon is more closely associated with bridge deck spalling, but in the case of these pier caps, the principle is the same.

Interior Verses Exterior Girders

The question that remains unanswered, however, is one of why the vertical movement of the girders is concentrated at the interior girders of a structure. In the case of Structure I-94-41-4480, four of the interior girders are moving (Figure 6, Chapter 2). From studying the cross section of Figure 23, it is seen that the interior girders incorporate most of the top slab surface area. The effective flange width design criteria is a good indicator of how much surface area of the top slab each interior and exterior girder incorporates. The effective flange width of an I-94-41-4480 interior girder has been computed before and appears in Figure A1 of Appendix A. The interior effective flange width is 7.3 feet. The exterior effective flange width was computed from the criteria presented in AASHO Specification (1969 ed.) Section 1.5.5(A) (AASHO criteria is the same as ACI 318-63); and the girder span length divided by 12 controlled. The effective flange width for the exterior girder was 2.42 feet, measured from the

a) Mud deposits left by moisture draining from underneath girder

b) Salt deposits left by moisture drainage

Figure 39. Traces of moisture.

face of the girder. From Figure 34, another 12 inches must be added to allow for the part of the top slab directly over the exterior girder. This brings the total effective flange width, for surface area, equal to 3.42 feet. Of the total top slab surface area ($51.83 \text{ ft}^2/\text{ft}$ in Figure 34), $2(3.42)$ or $6.84 \text{ ft}^2/\text{ft}$ can be attributed to the exterior girders. The remaining top slab surface area, $(51.83 - 6.84) = 45 \text{ ft}^2/\text{ft}$ is attributed to the interior girders. The interior girder portion represents 87% of the total top slab surface area. So when the sun shines on the top slab, the interior girders experience greater exposure to temperature based on the greater top slab surface area the interior girders incorporate.

Bending in the longitudinal direction (parallel to the span) due to temperature differences between the top deck and bottom of the girders has been discussed. However, these temperature differences cause bending in the transverse direction (perpendicular to the span) also. Therefore there can be a thermal gradient from one side of the bridge to the other. In this study, this effect was not considered. This effect may contribute to why the interior girders exhibit vertical movement at the pier caps that the exterior girders do not.

CHAPTER 5

CONCLUSIONS

General

It is clear that a detailed explanation of the failure mode of this type of structure is difficult. However, general conclusions and recommendations can be derived from the results of this study. It is evident that the vertical and horizontal girder movement relative to the pier cap, jamming of opposing girders, and pier cap spalling cannot be caused by a single phenomenon. It is rather a combination of various phenomena that produce the various stages of the failure mode. From the deflection data and end rotation computations, it is evident that the resulting vertical movement of the girders at the pier caps is temperature related. Observation of the strain data and inspection of opposing girders and expansion joints leads to the belief that longitudinal movement of the girders occurred. Whether this longitudinal movement is due to season to season (i.e., winter-summer) temperature changes or "scooting" of the girders caused by heavy trucks bouncing on the bridge deck due to approach pavement settlement is not known. Longitudinal girder movement may be a combination of both effects. Had time permitted, long term horizontal deflection measurements as taken on I-94-41-4480 could have given insight into the contribution of temperature and scooting to the failure mechanism. Generally speaking, a temperature change of 100°F

is quite possible over a typical Midwest winter-summer period. This change in temperature would represent a change in length of a typical I-94-41-4480 girder of $(0.0000055)(100)(349)$, or 0.2 inches. If this effect was combined with girder scooting or temperature rotation (temperature rotation would cause opposing girders to rotate toward each other) jamming could quite conceivably occur. Since the effects described above are the results of long term phenomena, a great deal of time and patience would be required to gain useful horizontal deflection data.

Observation and inspection of the 1 inch joint between the approach slab and the bridge deck at the first bent led to conclusions concerning pavement growth and its effect on girder jamming. This 1 inch joint was closed on the bridges observed and checking with ISHC personnel, who were involved with many more distressed structures than the few documented in this study, the opinion was that this joint was tight and closed. This was due to either

1. expansion of the pavement due to high summer temperatures thus closing the joint
2. settlement of the soil under the end bent causing the end bent to rotate toward the approach slab which could be further enhanced by girder rotation at this end due to a temperature difference from top deck to girder bottom.

Raveling of the approach slab or bridge deck at the end bent was not observed for the bridges of this study. Since this end bent is usually an expansion end for the girders, the girders would be free to translate under forces exerted by pavement expansion thus causing jamming of opposing girders at the first intermediate pier.

The question that remains, however, is one dealing with pavement relief joints. From discussion with ISHC personnel, it is evident that pavement relief joints were placed in the pavement near structures during construction. Due to pavement expansion under high temperatures these joints were not adequate and larger joints were cut. In the meantime, however, girder jamming possibly occurred due to the pavement growth producing the damage as shown in Figure 3. Obviously the pavement relief joints are to keep the pavement from buckling but in the case of these simply supported structures, movement of the individual spans due to pavement expansion is likely to occur.

Observation and inspection of the distressed structures also gave insight into the spalling of the pier caps induced by moisture and enhanced by vertical girder movement. The elimination of moisture from underneath the girder is a problem of prime importance.

Recommendations

A remedy for the failure pattern described in the previous section is possible but difficult. The remedy is difficult because the structures involved are in various stages of distress. Structure I-94-41-4480 is undergoing vertical girder movement at the pier cap with very little pier cap spalling while Structure I-70-01-4868 exhibits excessive pier cap spalling but little vertical girder movement at the pier cap. Still other bridges show no signs of distress yet. These bridges are the most important in that action needs to be taken before structural distress occurs.

All structures exhibiting the failure characteristics or exhibiting the potential to do so should be categorized as to their span lengths and cross sections. Then end rotations due to dead load and temperature can be computed for each category. The percentage difference, relative to the temperature rotation, (as computed in Table 7) can be calculated. A percentage difference, say 10%, can be set as a standard for which any category of structures exhibiting a percentage difference of dead load rotation and temperature rotation greater than 10% should be repaired. The repair procedure should include extending the diaphragms, at the intermediate piers, down to the top of the pier cap. This procedure could be accomplished by using a pneumatically applied mortar mix using expansive cement. Any further girder movement will be eliminated because the extension of the diaphragms will increase the bearing area of the girders. Vertical girder movement that is occurring presently would also be eliminated by this procedure.

The next step is to eliminate the moisture seepage through the expansion joints. This can be accomplished by a waterproof membrane strip, 6-10 feet wide, placed over each expansion joint and an overlay applied over the entire bridge deck. An alternate method would be to chip away concrete around the expansion joint, place a water seal in the chipped out area and place a sealer over the water seal then apply an overlay over the entire bridge deck. In this case the water seal and sealer replace the membrane. This repair procedure is presented as an economical solution to use for structures that have not exhibited the failure characteristics to the degree that they need

extensive repair. The procedures used to repair extensively damaged structures are quite expensive so it is hoped that the repair procedure presented here, if used on slightly damaged structures or structures presently not exhibiting the failure characteristics, will prevent extensive damage from occurring. However, the repair procedure described above does not keep the structure from rotating due to temperature. A repair procedure involving a method to keep the girders from lifting off due to a temperature difference would be quite expensive and would not be any more economical than the current extensive repair procedure of making the structures continuous.

BIBLIOGRAPHY

BIBLIOGRAPHY

1. Dally, James W., Riley, William F., Experimental Stress Analysis, McGraw-Hill, Inc., New York, 1965.
2. American Association of State Highway Officials, Standard Specifications for Highway Bridges, AASHO General Offices, Washington, D. C., 1969.
3. American Concrete Institute, Building Code Requirements for Reinforced Concrete, ACI, Detroit, 1963.
4. Urquhart, Leonard C., O'Rourke, Charles E., and Winter, George, Design of Concrete Structures, McGraw-Hill Inc., New York, 1958.
5. Wang, Chu-Kia, Statically Indeterminate Structures, McGraw-Hill Inc., New York, 1953.
6. Tonini, D. E., and Dean, S. W., Jr., Chloride Corrosion of Steel in Concrete, American Society for Testing and Materials, Philadelphia, 1977.
7. Perry, C. C., Lissner, H. R., The Strain Gage Primer, McGraw-Hill Inc., New York, 1955.

APPENDICES

APPENDIX A

APPENDIX A

SAMPLE CALCULATIONS

Whittemore Temperature Compensation

The initial 10-inch Whittemore reading for I-94-41-4480 was taken on August 30, 1977, at 8:40 a.m.

Initial Whittemore reading	0.04610
Concrete Surface Temp.	77.6°F
Whittemore Std. Rdg.	0.04600
Bar Temperature	83°F
Temp. Difference	$(83-77.6)^{\circ}\text{F} = 5.4^{\circ}\text{F}$ (contraction)
Thermal Elong.	$(0.0000055) (-5.4) = -0.00003$
Adj. Init. Rdg.	$0.04610 + 0.00003 = 0.04613$ (thermal elongation as contraction has to be added in as expansion to compensate and vice versa)
Elongation	$0.04613 - 0.04600 = +0.00013$
Avg. Strain	$\frac{+0.00013}{10} = +0.000013$
Second Whittemore Rdg.	0.04600
Concrete Surface Temp.	77.°F
Whittemore Std. Rdg.	0.04600
Bar Temperature	83.°F
Temp. Difference	-6°F
Thermal Elong.	-0.00003
Adj. Second Rdg.	+0.04603

Elongation	+0.00003
Avg. Strain	+0.000003
Change in Strain (relative to initial strain)	$+0.000003 - 0.000013 = -0.000010 \text{ in/in}$ (or $-10\mu \text{ in/in}$)

The value computed above (-10 micro-inches-per-inch) appears in Table 2, girder 7, bottom, first reading in column.

Temperature Calculation

The manufacturer of the temperature sensors provided a table of sensor resistance that is proportional to temperature in degrees Fahrenheit. This table is necessary since the relationship between sensor resistance and temperature is not linear. Interpolations between values given in the manufacturer's table were made in a linear fashion; since the increments were small.

Theoretical Strain Calculation (Live Load)

The following calculations are based on the information contained on the drawings for Structure I-94-41-4480.

The absolute maximum live load moment will occur with an HS-20-44 truck on the bridge in the position shown in Figure A1. Note that the first wheel load is onto the next span before the two remaining wheel loads. Therefore there are only the two wheel loads shown on any one span at any one time. Each interior girder must support the stringer spacing, S , divided by 6.0 (AASHO 1969 ed. Sect. 1.3.1). The spacing of the I-94-41-4480 girders is 7.3-feet, so W of Figure A1 is equal to 1.22 (16-kips) or 19.5-kips. Therefore:

a) HS-20-44 loading for absolute maximum moment

b) I-94-41-4480 interior girder cross section

c) Stress diagram

Figure A1. Conditions for computing strain.

$$R_L = 19.5/29.1(29.1 - 2.9 + 12.2) = 25.7\text{-kips}$$

$$R_R = 19.5/29.1(29.1 - 12.2 + 2.9) = 13.3\text{-kips}$$

$$M_{\max, L.L.} = (13.3)(12.2) (12) = 1947 \text{ inch kips}$$

The moment of inertia for a typical I-94-41-4480 interior girder was computed using the cross section shown in Figure A1. The effective flange width was controlled by the center to center spacing of the girder (AASHTO, 1969 ed., Section 1.5.5(A)). The gross moment of inertia was computed to be 116,951 in⁴. The gross moment of inertia was used due to ACI 318-63 Section 909(c) ($\rho f_y \leq 500$ psi) and AASHTO 1969 ed. Section 1.5.2 (8) (which assume no cracking).

The stress diagram for the girder cross section appears in Figure A1. From bending theory it is known that $\sigma = Mc/I$, so for the top fiber of the stress diagram shown in Figure A1, $\sigma = (1947)(11)/116951$ or 0.183 ksi in compression. The bottom fiber stress is equal to $(1947)(24.4)/116951$ or, 0.406 ksi in tension. It is also known that stress is related to strain, in the elastic region, by the formulation $\sigma = E\epsilon$. The modulus of elasticity, E, was computed from ACI 318-63 Section 1102 ($E = w^{1.5} 33/f'_c$). From the I-94-41-4480 drawings $f'_c = 1200$ psi (WSD). In AASHTO, 1969 ed., Section 1.5.1(c) $f'_c = 0.4 f'_c$, so $f'_c = f'_c/0.4$ and $E = 3320558\text{-psi}$. Thus, the corresponding strains are 55 micro-inches-per-inch, compression, for the top fiber and 122-micro-inches-per-inch, tension, for the bottom fiber.

Theoretical Deflection Calculation (Live Load)

The same loading condition applies (as Section A.3, Figure A1) for the computation of deflection for an I-94-41-4480 interior girder.

In the conjugate beam approach, the deflection of the actual beam at any particular point is the actual moment of the conjugate beam when loaded with the M/EI diagram. The girder in Figure A1 was loaded with the M/EI diagram based on the HS-20-44 loading condition shown and the moment of that diagram taken about the point of maximum moment, computed. This value was equivalent to $21129649/EI$ and is in fact the deflection of the beam at the point of maximum moment. Using the same assumptions and cross sections as previously used and taking E in ksi and I in inches⁴, the deflection is 0.054 inches downward.

Live Load Rotation

In computing the rotation due to HS-20-44 live loading, the loading condition in Figure A1 is used. In the conjugate beam approach, it is known that the shear computed in the conjugate beam is the rotation of the actual beam. Therefore the maximum shears computed for the conjugate beam, and hence the maximum live load rotations, occur with the loads shown as in Figure A1 and will be equivalent to the reactions R_L and R_R . Loading Figure A1 with the M/EI diagram and computing R_R was done in Section A.4 for the deflection calculation. Since R_R was known, forces for the conjugate beam in the vertical direction were equated and R_L solved for. The two reactions, R_R and R_L are equivalent to $-191796/EI$ and $204353/EI$ respectively where E is in KSI and I is in In⁴. Using the same E and I as used in Section A.3, the end rotations then are $\theta_R = 0.00049$ radians counter-clockwise and $\theta_L = 0.00053$ radians clockwise.

APPENDIX B

APPENDIX B

STRUCTURE I-70-01-4868

General

Structure I-70-01-4868 was the first structure to be tested. Strain and temperature measurements were taken. Strain measurements were confined to the use of electrical-resistance type gages only. Temperature measurements were made with the temperature sensors. Measurements were taken on girder 5 (at midspan) of the westbound structure as shown in Figure B1 and on the top deck as illustrated by Figure B2. Strain and temperature data were recorded continuously over a 24-hour period during the latter part of June, 1977. The weather during the test was hot and humid with an average temperature of 85°F, over the 24-hour period.

Gage Details

Location of strain gages and temperature sensors are shown in Figures B2 and B3. The temperature sensors used are of the same variety as those used on Structure I-94-41-4480 but the strain gages are of the type EA-06-20CBW-120 with a 2-inch gage length. Installation details and test set-up are shown in Figures B4 and B5 respectively.

WESTBOUND STRUCTURE

SECTION A-A

Figure B1. I-70-01-4868 instrumentation.

WESTBOUND STRUCTURE

●=STRAIN GAGE

▲=TEMP SENSOR

PLAN VIEW

Figure B2. I-70-01-4868 instrumentation.

Figure B3. I-70-01-4868 instrumentation.

a) Girder and deck gage seats

b) Top deck installation

Figure B4. I-70-01-4868 instrumentation

a) Girder instrumentation

b) Test set-up

Figure B5. I-70-01-4868 instrumentation

Recorded Data

Strain gage data is presented in Table B1 and has been adjusted for zero drift of the strain indicator. All measurements are in micro-inches per inch. The circled numbers appearing in Table B1 correspond to the circled numbers adjacent to the gage location pictured in Figures B2 and B3. Location description is as follows:

Circled Number	Location
1	Top Deck (Figure B4)
2	Bottom Deck
3	Top Girder (side)
4	Bottom Girder (side)
5	Bottom Girder

Plots of the strain data of Table B1 are shown in Figures B6 and B7 and again circled numbers of the plots correspond to tabular data of Table B1. Typical tensile-compressive bending behavior of the girder is evident from Figure B7.

Temperature data is tabulated in Table B2. All readings are in degrees Fahrenheit. The circled number system is again employed to correlate locations in Figures B2 and B3 to the columns of Table B2. By comparing the difference of column 1 and column 5, a maximum difference (ΔT) of 28.4°F is realized (from top deck to bottom girder).

Table B1. 1-70-01-4868 Recorded Strain

DATE & TIME	①	②	③	④	⑤
JUN.28					
10:10 pm	0	0	0	0	0
11:10 pm	0	+33	-57	+6	+64
JUN.29					
12:10 am	+3	+37	-55	+8	+65
1:10 am	+5	+44	-47	+13	+67
2:10 am	-4	+48	-46	+16	+67
3:10 am	-7	+49	-46	+16	+69
4:10 am	-7	+49	-46	+18	+70
5:10 am	-7	+49	-45	+18	+71
6:10 am	-10	+51	-44	+16	+72
7:10 am	-25	+59	-36	+19	+72
8:10 am	-27	+63	-30	+21	+74
9:10 am	-17	+72	-19	+20	+75
10:10 am	-8	+75	-12	+20	+73
11:10 am	-6	+75	-12	+20	+73
12:10 pm	-20	+75	-5	+21	+68
1:10 pm	-51	+60	-18	+7	+53
2:10 pm	-60	+56	-17	+8	+54
3:10 pm	-60	+50	-21	+7	+54
4:10 pm	-64	+51	-20	+9	+56
5:10 pm	-51	+45	-24	+10	+57
6:10 pm	-40	+43	-29	+15	+55
7:10 pm	-32	+41	-36	+16	+60
8:10 pm	-56	+77	-18	+41	+7
9:10 pm	-26	+69	-25	+39	+10
10:10 pm	-15	+74	-18	+38	+13

ALL READINGS $\times 10^{-6}$ (MICRO-STRAIN)

Figure B6. Strain plots I-70-01-4868.

Figure B7. Strain plots I-70-01-4868.

Table B2. I-70-01-4868 Concrete Surface Temperature

DATE & TIME	①	②	③	④	⑤
JUN. 28					
10:10 pm	95.0	87.0	84.3	76.3	75.7
11:10 pm	91.7	85.0	83.0	75.0	75.0
JUN. 29					
12:10 am	93.7	85.7	83.0	75.0	75.7
1:10 am	91.7	84.3	84.3	77.0	77.7
2:10 am	90.3	83.0	83.7	76.3	77.7
3:10 am	89.7	81.7	83.7	77.0	77.0
4:10 am	86.3	77.7	79.7	73.6	74.3
5:10 am	88.3	79.7	79.0	73.6	73.6
6:10 am	87.0	78.3	80.3	75.7	76.3
7:10 am	88.3	78.3	80.3	72.3	72.9
8:10 am	88.3	77.7	77.0	72.3	73.6
9:10 am	90.3	79.7	79.7	75.7	76.3
10:10 am	92.3	82.3	80.3	76.3	76.3
11:10 am	92.3	82.3	78.3	74.3	74.3
12:10 pm	95.0	85.0	79.7	75.0	75.0
1:10 pm	98.9	90.3	81.7	77.0	75.7
2:10 pm	101.7	93.7	84.3	77.7	77.0
3:10 pm	101.7	98.3	85.7	77.7	77.7
4:10 pm	104.8	98.3	87.0	79.0	77.7
5:10 pm	105.5	98.9	88.3	79.0	78.3
6:10 pm	104.8	100.3	89.0	79.0	77.7
7:10 pm	106.1	98.9	89.0	79.0	77.7
8:10 pm	101.5	97.6	90.3	80.3	80.3
9:10 pm	98.3	95.7	89.0	78.3	77.7
10:10 pm	98.9	95.0	89.0	77.7	77.7

ALL READINGS IN DEGREES FAHRENHEIT

APPENDIX C

APPENDIX C

STRUCTURE I-65-54-4654

General

Structure I-65-54-4654 was the only four span bridge tested and was the only bridge where data was gathered on an intermediate span. On all other structures, damage or movement was occurring in the first span. On this bridge damage had occurred at the first intermediate span (Figure C1). Both electrical-resistance strain gages and temperature sensors were used on this structure. Measurements were taken on this structure at midspan and near the ends of girder 4 pictured in Figure C1. Difficulties with the strain gages and inclement weather resulted in no recorded strain data; however, temperature measurements were still taken. The test of this bridge occurred during the latter part of July, 1977. The weather during testing was hot and humid turning to rain with an average ambient temperature of 81°F.

Gage Details

Location of strain gages is pictured in Figures C2 through C4. Several different ideas were introduced here. In an effort to gain a complete curvature diagram for the girder, strain gages were located along the entire girder length as illustrated by Figure C3. Also an attempt was made to place a gage on a 45° angle (Figure C4)

Figure C1. I-65-54-4654 instrumentation.

Figure C2. I-65-54-4654 instrumentation.

Figure C3. I-65-54-4654 instrumentation.

Figure C4. I-65-54-4654.

to measure strains (and thus stresses) along that plane. Since the girder ends were cracking in that plane it was thought that the cracking was a result of a force which was a resultant of the girder-pier cap frictional force and a force perpendicular to the frictional force caused by the neighboring girder "riding up" on the other.

Unfortunately this bridge was very high above the ground (Figure C5) and gage installation was extremely difficult. Evidently, poor bond of the gage to the concrete surface resulted. An attempt was made to balance the bridge of the strain indicator (Figure C5) for some of the gages but readings were too erratic. The temperature sensors, however, responded adequately.

Recorded Data

Observing the I-70-01-4868 temperature data (Table B1), it was noted that the hottest part of the day occurred during late afternoon. So in order to get the maximum thermal difference from top to bottom, the test was begun during late afternoon. The numbered columns of Table C1 correspond to the circled numbered gage locations shown in Figures C2 and C3. All readings are in degrees Fahrenheit. By comparing the difference between column 1 and column 5, a maximum difference (ΔT) of 23.4°F is realized (from top deck to bottom of girder).

a) Test set-up

b) Attempting to record data

Figure C5. Testing I-65-54-4654.

Table C1. I-65-54-4654 Concrete Surface Temperature

DATE & TIME	①	②	③	④	⑤
JUL.21					
5:30 pm	117.7	107.4	100.9	93.0	94.3
6:30 pm	115.2	108.1	101.5	93.0	94.3
7:30 pm	112.0	106.1	101.5	93.0	94.3
8:30 pm	110.0	104.8	101.5	92.3	93.0
11:30 pm	104.8	99.6	98.9	90.3	91.7
JUL.22					
6:30 am	97.6	91.0	93.7	88.3	89.0
8:30 am	101.5	91.0	93.7	89.7	90.3
9:30 am	106.8	93.0	93.7	90.3	91.0

ALL READINGS IN DEGREES FAHRENHEIT

APPENDIX D

APPENDIX D
STRUCTURE I-74-136-4331

General

Structure I-74-136-4331 differed from the previous bridges tested in that three girders were tested. Measurements were taken on various girders of both structures. The top deck of the westbound structure as well as an interior and exterior girder of the westbound structure were instrumented (Figures D1, D3, D5). An interior girder of the eastbound bridge was also instrumented (Figure D2). Both electrical-resistance strain gages and temperature sensors were used on these structures; however, difficulties with strain gage installation and inclement weather once again resulted in no useful strain data. The temperature sensors worked adequately. These bridges were tested during the middle of August, 1977, and the weather was hot and humid changing to rain with an average ambient temperature of 86°F.

Gage Location

Figures D1, D4, and D6 illustrate the location of the strain gages and temperature sensors. All girder locations are at midspan of the individual girders and on the shoulder of the top deck for the westbound bridge. Figure D7 shows the test set-up at the bridge site. The strain indicator was used in an attempt to balance the strain gages, but, once again, efforts failed as the readings became too

• = STRAIN GAGE
 ▲ = TEMP. SENSOR

Figure D1. I-74-136-4331 instrumentation.

WEST BOUND STRUCTURE

SECTION A-A

Figure D3. I-74-136-4331 instrumentation.

Figure D4. I-74-136-4331 instrumentation.

WESTBOUND STRUCTURE

SECTION A-A

Figure D5. I-74-136-4331 instrumentation.

Figure D6. I-74-136-4331 instrumentation.

a) Structure I-74-136-4331 over Sand Creek

b) Test set-up

Figure D7. Testing I-74-136-4331.

erratic due evidently to poor bonding between the gages and the concrete surface.

Recorded Data

Data were recorded as long as possible for the temperature sensors. This data appears in Table D1. All readings are in degrees Fahrenheit. Testing had to be discontinued at the occurrence of a rainstorm since thermal difference between the top and bottom of the bridge was altered. Looking at the values in column 1 of Table D1, it is seen that between 2 and 4:00 p.m. a drastic change in temperature (26.9°F) occurred. This was due to the rain shower which cooled off the top deck rapidly as the rain fell. In fact a negative temperature difference occurs (between top deck and bottom girder) because the girder bottom is hotter than the deck since the girder is not exposed to the rain. By comparing columns 1, 3, and 5 of Table D1, a maximum positive thermal difference of 15.3°F is attained.

Table D1. I-74-136-4331 Concrete Surface Temperature.

DATE & TIME	TOP DECK	GIRDER 1		GIRDER 6	
		T	B	T	B
AUG. 16					
9:00 am	105.5	102.2	104.2	115.2	108.1
2:00 pm	132.4	102.2	117.1	118.4	118.4
4:00 pm	105.5	102.2	115.2	104.2	108.1
6:00 pm	101.7	101.5	113.9	103.5	106.8
7:30 pm	98.9	98.3	109.4	99.6	100.3

ALL READINGS IN DEGREES FAHRENHEIT

APPENDIX E

APPENDIX E
STRUCTURE I-74-134-4328

General

Structure I-74-134-4328 was the last structure to be tested. Since the previous two structures (I-65-54-4654 and I-74-136-4331) were failures as far as strain data were concerned, the major thrust of this test was to accumulate some useful strain data.

Various girders of both structures were instrumented, both at midspan and near the ends. Figure E1 shows two interior girders of the westbound bridge. Figure E2 pictures an interior and exterior girder of the same structure but in different spans. Both of these girders were instrumented. Figure E3 shows an interior girder of the eastbound structure that was instrumented. It should be noted that vertical movement of the girder with respect to the pier cap was evident in all girders except the exterior girder.

These bridges were tested in early October, 1977, over a three day period on a continuous basis. Readings were recorded until dark and then terminated until the following day with all leadwires attached to the strain indicator over the entire test period. Weather during the test varied from mild and temperate to rainy drizzle with an average ambient temperature of 55°F.

WESTBOUND STRUCTURE

SECTION A-A

Figure E1. I-74-132-4328 instrumentation.

WESTBOUND STRUCTURE

SECTION A-A

Figure E2. I-74-132-4328 instrumentation.

EASTBOUND STRUCTURE

SECTION A-A

Figure E3. I-74-132-4328 instrumentation.

Figure E4. I-74-132-4328 instrumentation.

Figure E5. I-74-132-4328 instrumentation.

Gage Location

Strain gages used were of the type EA-06-20CBW-120 with a 2-inch gage length. All of the girders were instrumented at midspan with the exception of girder 3 (Figure E1) of the westbound structure. This girder had gages installed at midspan and near the ends as shown in Figure E4. In addition to this arrangement, the girder opposite girder 3 in the intermediate span had a gage placed on its bottom the same distance from the pier cap as girder 3 (Figure E5). This was done in an effort to pick-up unusual girder behavior in the form of jamming of two opposing girders. If one girder was jamming against the other larger strains should be evident. The remaining strain gage locations are shown in Figure E6. The exterior girder instrumentation is shown in Figure E7 along with an end view of girder 3 instrumentation.

Recorded Data

Strain gage data are presented in Tables E1, E2, and E3. All readings are in micro-inches per inch. The data have been adjusted for zero drift of the strain indicator. The first column of Table E3 shows tabulated data for the top of girder 3 eastbound only. This is because the filament of the bottom strain gage was faulty and this was not noticed until after the gage was epoxied to the concrete surface. The data of Tables E1 through E3 were plotted in Figures E8 through E15. The strain plot of Figure E8 shows the strain at the bottom of the girder opposing girder 3. Since the strains approach the ultimate strain of concrete, structural distress (in the form of

Figure E6. I-74-132-4328 instrumentation.

a) Exterior girder installation

b) Girder 3 east end installation

Figure E7. I-74-136-4431 instrumentation.

Table E1. I-74-132-4328 Recorded Strain.

DATE and TIME	GIRDER 3-M W.B. WEST END PIER 2	GIRDER 3 W.B. EAST END PIER 2		GIRDER 3 W.B. CENTER-LINE		GIRDER 3 W.B. WEST END BENT 1	
		T	B	T	B	T	B
OCT. 5	B	T	B	T	B	T	B
9:00 p.m.	-6	-20	+8	-4	+20	-15	-6
OCT. 6							
8:15 a.m.	+4	+125	+48	-50	+18	-66	-49
9:45 a.m.	-2	+93	+36	-30	+24	-40	-50
11:00 a.m.	-4	+46	+27	-22	+8	-22	-58
1:00 p.m.	-4	+82	+10	-11	+5	-2	-65
2:10 p.m.	-15	+85	+2	-23	+4	-1	-72
3:00 p.m.	-34	+70	-12	-37	-14	-28	-86
4:00 p.m.	-32	+66	-10	-52	-18	-42	-92
5:00 p.m.	-26	+24	+4	-55	-15	-53	-84
6:00 p.m.	-29	-10	+10	-71	-18	-75	-83
7:00 p.m.	-24	-44	+26	-74	-10	-86	-76
8:15 p.m.	-15	-70	+39	-79	-14	-97	-78
9:30 p.m.	-24	-60	+37	-87	-10	-97	-76
OCT. 7							
9:15 a.m.	-13	+237	+30	-59	-3	-85	-72
11:00 a.m.	-21	+174	+18	-54	-8	-74	-86
1:45 p.m.	-29	+148	+10	-54	-12	-69	-81
3:45 p.m.	-31	+133	+6	-74	-18	-78	-90
4:05 p.m.	-27	+113	+11	-69	-15	-81	-86
6:00 p.m.	-28	+89	+14	-69	-10	-83	-82
8:15 p.m.	-18	+66	+20	-66	+2	-94	-80
OCT. 8							
9:05 a.m.	-924	+113	+23	-11	+29	-101	-108
11:05 a.m.	-1532	+127	+2	0	+28	-98	-109
12:30 a.m.	-1974	+212	-238	+146	+92	-52	-82
2:00 p.m.	-2508	+302	-246	+182	+86	-52	-87
3:00 p.m.	-2706	+618	-240	+180	+90	-54	-94
5:00 p.m.	-2930	+567	-218	+172	+97	-62	-92
7:00 p.m.	-3212	+489	-178	+146	+76	-102	-112
9:30 p.m.	-2932	+393	+162	+18	+19	-142	-133
OCT. 9							
8:00 a.m.	-4	+332	+377	+98	+33	-73	-70

Table E2. I-74-132-4328 Recorded Strain

DATE and TIME	GIRDER 4 W. B. PIER 2	
	T	B
OCT. 5		
9:00 p.m.	-12	0
OCT. 6		
8:15 a.m.	-44	-32
9:45 a.m.	-36	-36
11:00 a.m.	-30	-40
1:00 p.m.	-24	-45
2:10 p.m.	-23	-44
3:00 p.m.	-24	-64
4:00 p.m.	-34	-71
5:00 p.m.	-56	-72
6:00 p.m.	-66	-67
7:00 p.m.	-72	-62
8:15 p.m.	-83	-68
9:30 p.m.	-78	-64
OCT. 7		
9:15 a.m.	-63	-67
11:00 a.m.	-54	-70
1:45 p.m.	-57	-69
3:45 p.m.	-57	-68
4:05 p.m.	-67	-69
6:00 p.m.	-69	-66
8:15 p.m.	-74	-68
OCT. 8		
9:05 a.m.	-83	-100
11:05 a.m.	-80	-101
12:30 a.m.	-68	-94
2:00 p.m.	-50	-108
3:00 p.m.	-56	-106
5:00 p.m.	-66	-94
7:00 p.m.	-93	-133
9:30 p.m.	-123	-161
OCT. 9		
8:00 a.m.	-49	-34

Table E3. I-74-132-4328 Recorded Strain

DATE and TIME	GIRDER 3 E.B. PIER 3	GIRDER 1 W.B. PIER 3		GIRDER 4 W.B. PIER 3	
	T	T	B	T	B
OCT. 5					
9:00 p.m.	0	+16	+22	+12	+9
OCT. 6					
8:15 a.m.	-11	+19	+62	0	-56
9:45 a.m.	+36	+25	+61	+3	-74
11:00 a.m.	+49	+32	+58	+14	-85
2:10 p.m.	+10	+15	-12	0	+3
3:00 p.m.	+12	+35	-32	-10	+33
4:00 p.m.	+14	+35	-24	-13	+39
5:00 p.m.	-4	+21	-30	-34	+36
6:00 p.m.	+1	+17	-25	-43	+37
7:10 p.m.	-10	+16	-10	-36	+60
8:15 p.m.	0	+22	-8	-39	+60
9:30 p.m.	0	+17	+2	-36	+66
OCT. 7					
9:20 a.m.	+8	+16	+14	-14	+56
11:00 a.m.	+27	+61	+23	+46	+46
1:45 p.m.	+47	+110	+34	+126	+47
2:45 p.m.	+44	+115	+26	+125	+42
4:10 p.m.	+41	+115	+31	+138	+39
6:00 p.m.	+40	+138	+30	+164	+53
8:15 p.m.	+19	+239	+98	+267	+82

Figure E8. Strain plot I-74-132-4328.

Figure E9. Strain plots I-74-132-4328.

Figure E10. Strain plots I-74-132-4328.

Figure E11. Strain plots I-74-132-4328.

Figure E12. Strain plot I-74-136-4331.

Figure E13. Strain plot I-74-132-4328.

Figure E14. Strain plots I-74-132-4328.

Figure E15. Strain plots I-74-132-4328.

cracking) should be evident. No cracking was observed at this point so the gage must have malfunctioned. The general conclusions from observation of the strain plots are that the girders do not exhibit typical bending behavior at the center-line. The typical compression-tension behavior (from top to bottom of girder) expected gives way to tension-tension (Figure E14) or compression-compression (Figure E12) in some cases. This leads to the belief that the girder is experiencing some type of axial loading produced possibly by girder jamming or some other loading condition. "Jumps" in the data similar to I-94-41-4480 are also evident in this case also. Generally speaking, the strain plots, at the girder center-line, are similar to those for I-94-41-4480.

COVER DESIGN BY ALDO GIORGINI