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16. Abstract

Traffic in lanes adjacent to a deck that is being widened or reconstructed causes deflections and vibrations in the fresh concrete deck. A study of the effects of these disturbances in concrete decks is reported here.

Decks in service for years were inspected for signs of deterioration; deflections and vibrations were measured during concrete placement and initial curing; bridge deck cores were analyzed for cracks and signs of bonding problems in the reinforcing steel and tested for strength and pulse velocity.

Laboratory beams were constructed and tested to simulate a transverse strip of a deck slab. Periodic deflections and vibrations were applied from time of casting to one day age.

No deterioration that could be attributed to traffic during construction and curing of the decks was found in existing decks. The study of the cores showed no difference in cracking in cores taken from disturbed areas of the deck from that in cores taken from undisturbed areas. There was evidence of creation of a void in the new concrete around certain rebars dowels bent at right angles in a horizontal plane upon emerging from the old concrete. This situation was found only in cores taken at the joint between old and new concrete.

The study shows that no detrimental effects in deck concrete supported by steel and prestressed concrete beams spanning up to about 100 ft should be expected when the decks are widened or reconstructed under normal traffic.

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BRIDGE SLAB CONCRETE PLACED ADJACENT TO MOVING LIVE LOADS

by

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Research Report Number 266-1F
Bridge Slab Concrete Placed Adjacent to
Moving Live Loads
Research Study Number 2-5-79-266

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State Department of Highways and Public Transportation
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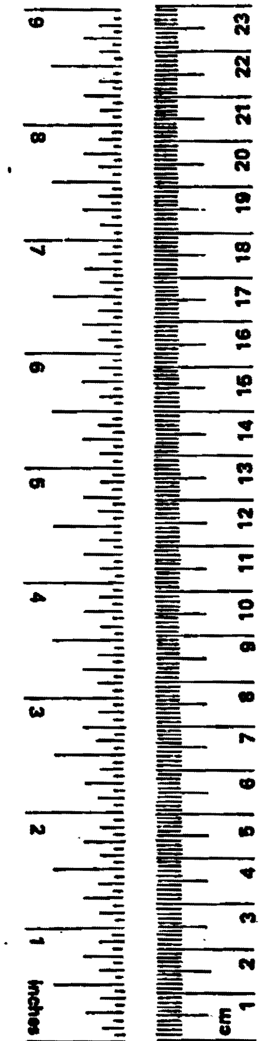
College Station, Texas

January 1981

METRIC CONVERSION FACTORS

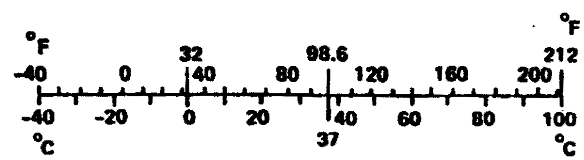
Approximate Conversions to Metric Measures

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



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



*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.

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PREFACE

This report was prepared in cooperation with the U. S. Department of Transportation, Federal Highway Administration.

The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

There was no invention or discovery conceived or first actually reduced to practice in the course of or under this contract, including any art, method, process, machine, manufacture, design or composition of matter, or any new and useful improvement thereof, or any variety of plant which is or may be patentable under the patent laws of the United States of America or any foreign country.

ABSTRACT

Traffic in lanes adjacent to a deck that is being widened or reconstructed causes deflections and vibrations in the fresh concrete deck. A study of the effects of these disturbances in concrete decks is reported here.

Decks in service for years were inspected for signs of deterioration; deflections and vibrations were measured during concrete placement and initial curing; bridge deck cores were analyzed for cracks and signs of bonding problems in the reinforcing steel and tested for strength and pulse velocity.

Laboratory beams were constructed and tested to simulate a transverse strip of a deck slab. Periodic deflections and vibrations were applied from time of casting to one day age.

No deterioration that could be attributed to traffic during construction and curing of the decks was found in existing decks. The study of the cores showed no difference in cracking in cores taken from disturbed areas of the deck from that in cores taken from undisturbed areas. There was evidence of creation of a void in the new concrete around certain rebars dowels bent at right angles in a horizontal plane upon emerging from the old concrete. This situation was found only in cores taken at the joint between old and new concrete.

The study shows that no detrimental effects in deck concrete supported by steel and prestressed concrete beams spanning up to about 100 ft should be expected when the decks are widened or reconstructed under normal traffic.

Key Words: bending curvature, bond, bridge, concrete, construction under traffic, cracks, damage, deck, fresh concrete, vibrations

SUMMARY

Introduction: Fresh concrete placed in bridge deck widening, repair, or replacement undergoes deflections from adjacent lanes carrying traffic. This study was made to determine if damages resulted to the concrete from such disturbance.

Procedure: Thirty bridges that had been widened under traffic and which had been in service for years were visually inspected. These bridges, typical of the highway system in Texas, had simple spans, continuous spans, steel beams, reinforced and prestressed concrete beams, and slabs. Spans ranged from 25 ft to 110 ft.

Deflections and vibrations were measured in bridges under reconstruction before, during, and for 24 hours after deck placement. The steel, the forms and the plastic concrete vibrated at the same frequency, therefore no bonding problems should be expected from traffic disturbances. Cores taken from these bridges, and other bridges, were studied for damage that might have resulted from adjacent traffic. Those from undisturbed regions of the decks were compared with those from disturbed regions to determine if cracking, bonding, and strength differences could be detected.

Laboratory beams simulating a one-foot wide transverse strip of bridge deck were constructed and tested. Periodic deflections and continuous vibrations were applied, and resulting overall deflections, cracks, and reinforcing movements were studied. Cores from these beams were taken to study crack depths and bonding of steel.

Findings: The visual inspection revealed no damages to the deck that could be attributed to widening under construction.

The bridge deck cores taken from disturbed regions were in the same good condition as those taken from undisturbed regions, with the exception of those taken at the joint between new and old deck. Five of these latter cores had a space around the reinforcing bar dowel extending from the old concrete into the

new concrete. The voids were produced when the bars, following the deflections and vibrations of the old concrete, displaced fresh concrete that could not flow back into position. All of these five cores came from bridges in which the dowel was bent at a right angle in a horizontal plane upon emergence from the old concrete. The dowels that extended straight, without bending, into the new concrete did not have the voids. In splitting the cores away from rebars, some poor imprint of bars in the concrete gave evidence that there had been some slight differential movement between steel and concrete at the joint. Most of the cores showed no sign whatsoever of distress.

The laboratory beams, supported on elastic supports, were periodically deflected at one end from the time of casting. Transverse cracking in the negative moment region of these beams occurred at about the time of set of the concrete. These cracks occurred at a curvature of approximately 0.36×10^{-4}/in., three times that found in the decks. Cores taken from these beams showed some blurred rebar imprints when the cores were split at the bars, but generally there was no indication of bonding problems.

The absence of flexural cracking in existing decks, and the finding that curvature in fresh concrete beams in the laboratory was three times that found from bridge deflection measurements indicates that no problem in flexure of new concrete due to traffic should be expected. The measurement of vibrations showed that the plastic concrete, the forms, and the reinforcing steel all vibrated at the same frequency and amplitude under traffic disturbances, therefore no problems of bonding should be expected.

Conclusions:

1. No evidence of problems in concrete placed and cured while traffic was maintained was found in bridges that have been in service for years after the new concrete was placed.

2. Traffic can be maintained during placement and curing without causing flexural problems in the fresh concrete.
3. Voids develop in fresh concrete around dowel bars bent at a right angle in a horizontal plane upon emerging from the old concrete. No surface evidence was found that these voids cause problems in the performance of the deck. No such voids were found in dowels that extended into the new concrete without bends.
4. Vibrations caused by normal bridge traffic have no detrimental effect on the concrete, the reinforcing steel, nor the interaction between the reinforcing steel and concrete.

Implementation:

The findings indicate that the current practice of maintaining traffic flow on bridges during deck concrete placement and cure has no widespread detrimental effect on the new deck.

Localized potential trouble areas exist at rebar dowels that are bent horizontally 90 degrees upon emergence from the old concrete. A wallowed out area was found in five cores from bridges using that detail for the dowel. The trouble was not found around dowels that extended straight, without bending, into the new concrete.

It is recommended that the dowel be treated as follows in order to avoid future void areas in concrete around the dowels at the joint between the old and new concretes:

Extend all dowel reinforcing bars approximately 24 bar diameters straight into the new deck area. Lap splice these dowels 20 bar diameters with the reinforcing bars that serve as transverse reinforcing for the new concrete. Tie the top-mat steel system to the dowels to prevent relative vertical movement between the dowels and the top-mat steel.

This will provide a space between the end of the top rebars and the break joint for a temporary filler, sometimes used in construction by stages, and sufficient lap length for transfer of flexural stress. The ties are needed to insure that the dowels and the rebars move in harmony, thus preventing the occurrence of voids found at the bent dowels.

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INTRODUCTION

The Problem

The volume of traffic on major highways has reached a point where many bridges have become constrictions that delay traffic movement. Flow may be eased in these cases by widening the bridge to provide additional lanes for the vehicles. The demand for continuous passage at all times has led to the current practice of maintaining traffic on the bridge at the same time that it is widened. This practice has brought with it problems of traffic control, structural design, and construction techniques.

Concrete is in a somewhat fluid state when it is first placed in the forms. The state of this concrete changes from fluid to plastic to solid in a time interval of only a few hours, depending to some degree on temperature. During this period, the continuous passage of traffic on the usable portion of the bridge transmits vibrations and deflections to the fresh concrete deck. Various elements of the deck move with respect to other elements, causing a disturbance of the concrete in the area. The reinforcing steel mat is flexible, but it is very stiff when compared with the fresh mass of fluid to plastic concrete. In the hydration process the concrete gradually changes from plastic to solid, and particles that earlier could readily slide on one another now become locked together by the hydrating cement. The aggregates that could move with respect to the steel bars, or the bars, even minutely, with respect to the concrete, now become bound in place.

The bonds in the concrete are very weak at first, but they gain strength rapidly. A broken bond between particles might heal at this stage if it could remain relatively still long enough. Continuous or intermittent movement, however, could prevent healing, and cracks might develop in the concrete, and bonding deficiencies develop between steel and concrete.

The gradual stiffening of the concrete requires it to conform to the various deflection configurations of the forms as traffic moves across the structure. This causes shear and flexural stresses which could lead to cracking. Deterioration of concrete often begins at cracks, and it is best to avoid them if possible.

This investigation was carried out to study the problems discussed above in connection with widening or replacement of bridge decks while they carry traffic loads.

Objectives

To determine if bridge deck concrete placed and cured adjacent to traffic-carrying lanes is detrimentally affected by vibrations and deflections caused by that traffic.

DISCUSSION OF THE PROBLEM

The widening of a bridge and the replacement of a bridge deck in stages present the problem of attachment of the new concrete to the old. In both, widening and staged replacement, the older concrete is prepared for the joint to be formed between the old and new parts. In widening, the curb portion of the slab is broken away to form the joint. In stage replacement, preparation is made in an earlier stage for a joint to be formed in a later stage.

The joint between the old and new material has sometimes been made without reinforcement by casting the new concrete against the old, but more often the two parts are joined by the reinforcing steel carried across the joint. One of the bridges surveyed in the Texas system, Number 28 in Appendix A, has an open joint, and the State of California has some joints of this kind (1). The two parts of the deck using such a joint deflect different amounts, causing problems in steering vehicles and of spalling of material at the joint. The State Department of Highways and Public Transportation of Texas (SDHPT) does not use the joint anymore; and California, after a study in 1965 (1), adopted the policy of running steel across the joint to attach one part to the other (2). Lapped reinforcing steel across the joint is preferred over the use of dowel bars.

Closure pours are used in California to reduce the effects of plastic deformation and shrinkage. These pours vary in width and time after widening, the former being in the neighborhood of two feet, and the latter some 3 to 15 days. These pours are not used by SDHPT.

The joint between old and new material is sometimes made directly over a supporting beam, and sometimes between beams. SDHPT uses both methods, see Figure 1, whereas California preference is for forming the joint between beams. The survey of bridges listed in Appendix A found no problem that would indicate preference of one over the other.

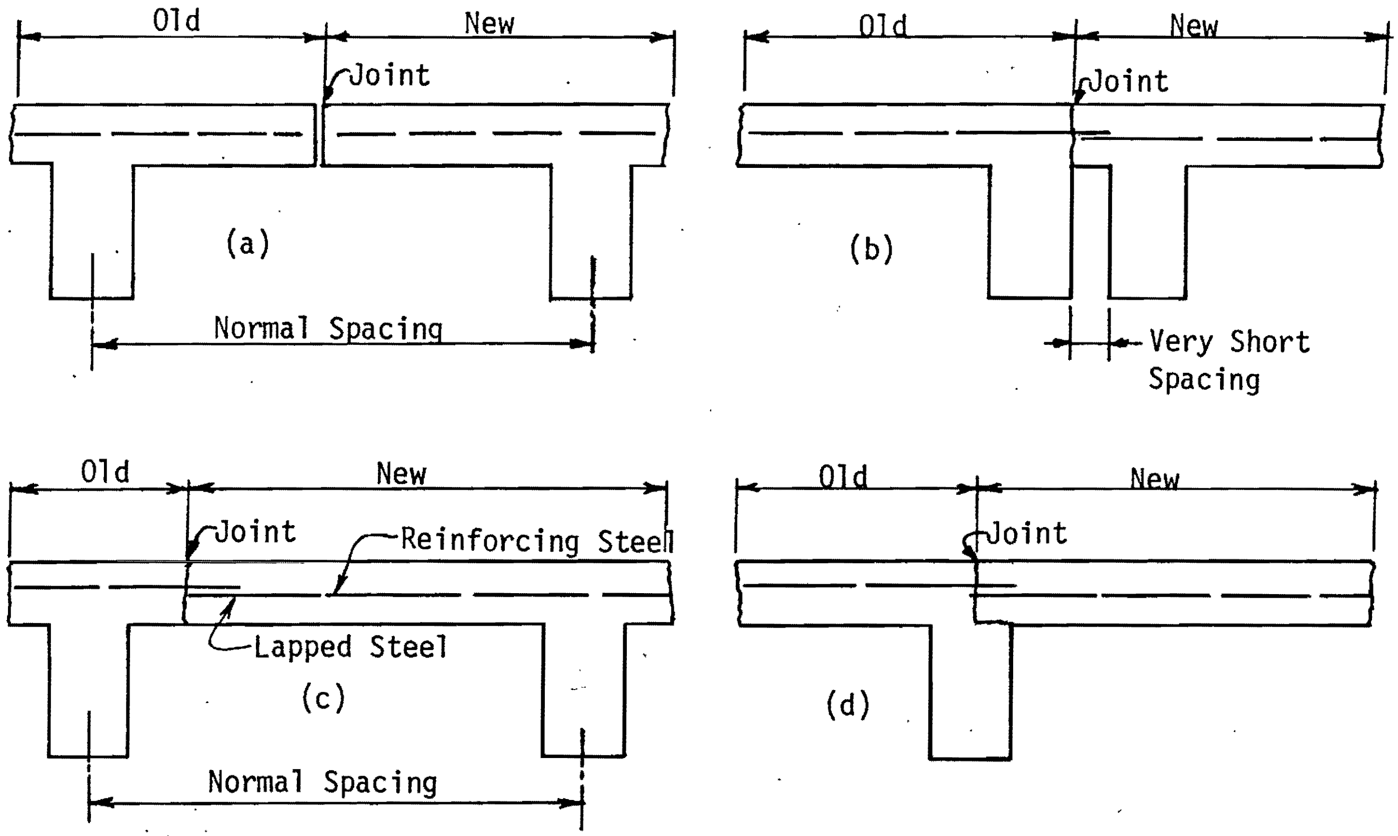


Figure 1. Types of Construction that Connect New Concrete to Existing Structure in Bridges Listed in Appendix A.

Regardless of whether the joint is made over a beam or between beams, the deflection when traffic moves over the structure causes movement of the fresh concrete. If a closure pour is used, the movement would occur in the fresh material of the closure strip. Figure 2 illustrates this type of movement. The reinforcing steel crosses over from the joint and is embedded in concrete when the new portion of the deck is cast. This bar might be lap tied to reinforcement placed for the new slab, or it might simply cantilever out from the existing material. When a vehicle crosses the bridge it deflects the existing slab. The added part is deflected some, because of diaphragm action, but not nearly as much as the old deck. The deck forms between the old and new are supported on a stiff spanner between beams, and the deflection of that form is linear between the beams. If the form is stay-in-place steel, it behaves in a similar way.

The stub bar from the existing material moves down with the old slab; it has little or no bending between its cantilevered end and the joint. In the overall movement, the end of this bar moves closer to the bottom form, and in doing so it displaces plastic concrete. The same general behavior can be predicted for the transverse reinforcing, in the new deck, which very likely extends all the way to the joint, overlapping the stub bar.

The deck steel is more complicated than that shown in Figure 2, which is simplified for clarity. Bars are supported from the deck form at frequent intervals, and they are tied to one another at laps and crossings at many points. The resulting steel system forms a mat which, although flexible, has some rigidity which gives the steel some degree of resistance to the movement of the concrete. If everything is in harmony the plastic concrete follows the movement of the forms, and the steel follows the movement of the concrete.

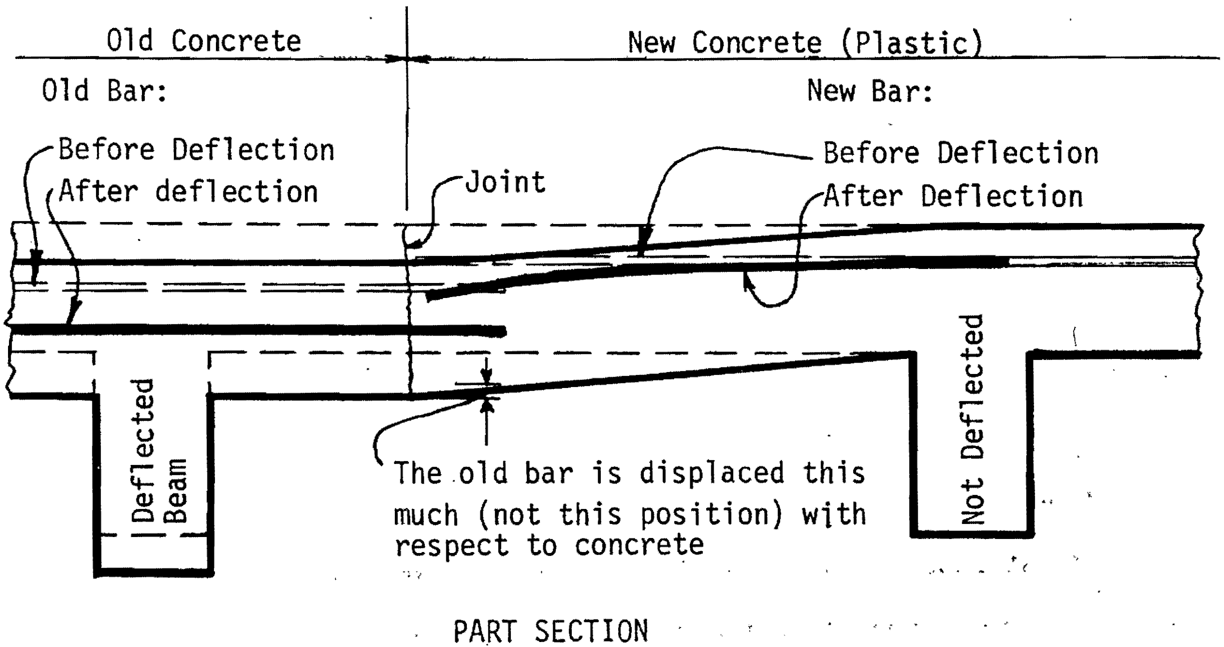


Figure 2. Relative Movements Between Old and New Concretes When Traffic Moves Across the Bridge.

There is the possibility that bars will bend between tie points to cause differential movement between steel and concrete, but that movement would be quite small, and its likelihood of occurrence is probably very low.

The movement such as that illustrated in Figure 2 is of greater concern because of its relative magnitude. When movement of the bar with respect to the surrounding concrete occurs, particles of concrete -- aggregates, mortar, water -- are forced from their positions. If other like particles do not move into the void left by those forced out, water pockets and voids will be formed. As long as the concrete is fluid, particles that are displaced will be replaced by others, but they might be segregated from overvibration if the movement is repeated many times. When the material becomes stiffer, with time, the displacement of particles requires more force, causing the steel to bend. There will be some displacement, however, and the displaced particles cannot be replaced because the concrete is too stiff. Action of this kind might leave voids where future deterioration could begin.

Another area where problems might develop is in the negative moment region, in the neighborhood of the beam on the right side of Figure 2. That area is shown in Figure 3, from work of Antoni and Corbisiero (3), at beam 1'. It is clear from this deflection pattern that the slab bends in the plane of the figure. The added portion of the slab has no stiffness when it is placed, and particles merely slide with respect to one another when the slab is deformed. As the concrete gains stiffness in its transformation to a solid state, the particles gradually bind to one another and movement becomes more restricted. The negative bending in the region of beam 1', Figure 3, is of interest here.

In an elastic system the flexural stress is related to the curvature of a beam by the well-known flexure formula:

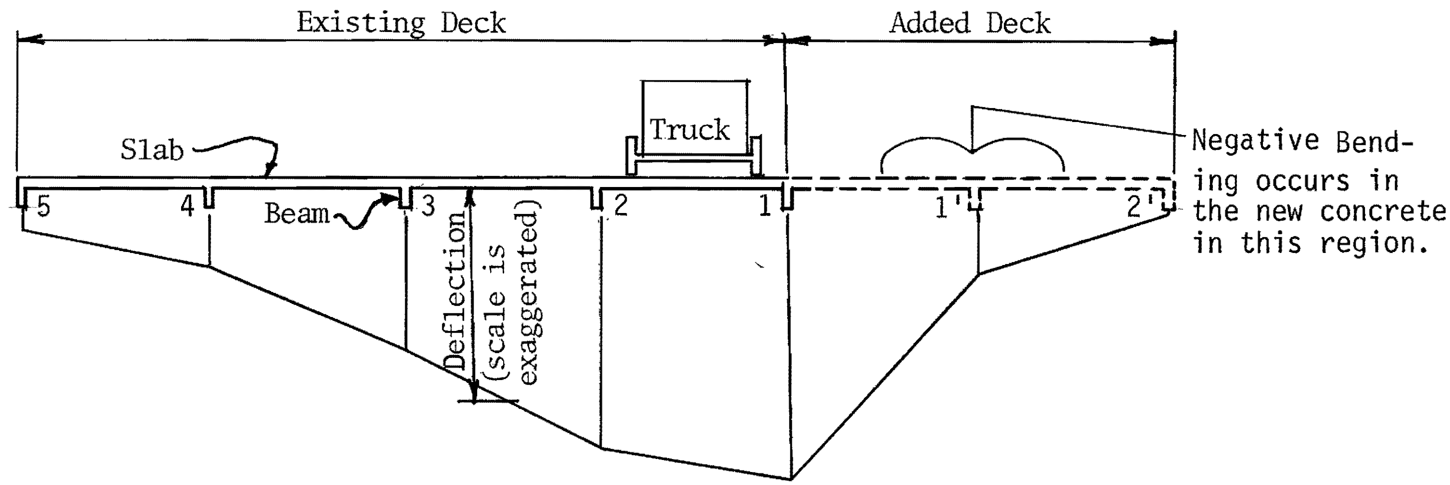


Figure 3. Deflection Pattern at a Transverse Section of a Bridge at Midspan.

$$\text{Curvature,} \quad \frac{1}{R} = \frac{M}{EI}$$

$$M = \frac{\sigma I}{c}$$

$$\frac{1}{R} = \frac{\sigma}{Ec}$$

where R is radius of curvature of the bent beam.

M is the bending moment at the section.

E is the modulus of elasticity of the material.

I is the moment of inertia of the section.

c is the distance from the neutral axis to the outer fibers of the beam, which is 1/2 depth for a rectangular section.

The curvature, $\frac{1}{R}$, that causes cracking in the fresh concrete of a widened or replaced deck can be determined if all terms on the right side of the equation are known. If the curvature that initiates cracking were known, traffic could be controlled so that cracking would not occur and a more durable slab could be the result.

Hilsdorf and Lott (4), in studies on plane-of-weakness cracking, found that cracking occurred in 6 in. deep slabs at 2½ to 4-hour age at a curvature approximately 5×10^{-4}/in. Cracks in the 4-hour concrete were wider than those in the 2½-hour material, indicating that modulus of elasticity increased faster than tensile strength.

Shidler (5) found that the ratio of modulus of elasticity to strength was essentially constant at early ages beginning at one day age, but it increased after the material aged. No data were given for ages less than one day.

The ACI Building Code (6) provides a formula for calculating modulus of elasticity based on the compressive strength of the concrete. That formula was developed from mature concrete, however, and is not intended for use of concrete which has not reached its design strength.

The information that is required for a theoretical study of the response of concrete, before it sets, to deformations and relative movement with respect to reinforcing steel has not yet been developed. Tests are needed to develop information to be used with the theory for these purposes.

Hansen (7) shows that the modulus of elasticity of concrete depends on the modulii and Poisson's ratios of the matrix and the particles (aggregates), and the volume concentration of the particles. For a given concrete, all of the values become constants except the modulus of elasticity and Poisson's ratio of the matrix. These values increase as the hydration progresses with time. Hansen also shows the relationship between strength and maturity of concrete, and, from this, a reasonable value of strength can be calculated at any time. But, the only way to determine the modulus of elasticity, at any age, is to collect the necessary data from tests. No such data are available for concrete at an age of initial set or thereabouts.

Only the information developed from Hilsdorf and Lott shown earlier -- strength ÷ modulus at 2-hour age is greater than at 4-hour age -- can be applied here in determining when the new deck concrete might first crack in flexure. From that information it appears that a deck might go uncracked for a few hours after casting as vehicles move across the bridge, but it might crack later, possibly at or about initial set, under the same traffic conditions as before.

The material and the behavior of decks replaced in stages or widened were the objects of this study. Defects that might have been caused by the movement of traffic on the bridge at the time of deck placement and curing were sought.

In the search, particular attention was given to cracks in the region judged to be the most disturbed by traffic, and the condition of concrete in cores taken from those regions.

The inspections and tests that were made, and which are described in separate sections of this report, were: on-site inspections; bridge deflection and vibration measurements; inspection of cores for cracks, voids, and bonding of steel; and laboratory beams under simulated highway loading.

TEST AND INSPECTION PROCEDURES

Introduction

A visual inspection of 30 bridges that had been widened or constructed in stages under traffic was made. Deflection and vibration measurements were carried out on nine bridges, cores from the decks of these bridges were inspected for the condition of the concrete, and laboratory beams were studied for concrete behavior as they were repeatedly deflected during and after casting. The details of each of these tests appear below in this order:

Visual Inspection of Bridges

Analysis of Cores from Bridge Decks

Field Measurements

Laboratory Beam Tests

Laboratory Beam Core Study

Visual Inspection of Bridges

Visual inspections were made of 30 bridges to determine if defects that might be traced to traffic during construction were evident. All of the bridges had been widened after they had carried normal traffic over a period of time. Reinforced concrete, prestressed concrete, and steel beam bridges were included along with a few slab bridges. Span types included simple, continuous, and overhanging beams; and span lengths ranged between 25 and 110 feet. This inspection consisted of a careful observation of the overall condition of bridges, the condition of the joints between old and new concrete, and the condition of the concrete in the vicinity of the joints. Distance measurements only were made in this inspection. Both the top and bottom surfaces were inspected, and no traffic control was used in the process. The bridges are listed in Table 1, and inspection notes are given in Appendix A.

TABLE 1. LIST OF BRIDGES THAT WERE VISUALLY INSPECTED.

	Bridge	Beam Type Old	Beam Type New	Traffic Maintained during Construction	Restrictions on Traffic during Construction: Weight	Restrictions on Traffic during Construction: Speed	Position of Traffic with Respect to New Concrete
1.	US 290 & GC&SF RR, Brenham	Stl	Stl	Yes	None	None	Adjacent Lane
2.	I-35 & Ave. D, Temple	Stl	Stl	Yes	None	30 mph	"
3.	I-35 & AT&SF RR, Temple	Stl	Stl	Yes	None	30 mph	"
4.	I-45 & FM 517, Galveston County	Stl	Stl	Yes	None	None	"
5.	I-45 & FM 518, Galveston County	Stl	Stl	Yes	None	None	"
6.	I-45 & FM 519, Galveston County	PC	PC	Yes	None	None	"
7.	SH 7 & Big Sandy Creek	Stl	Stl	Yes	None	45 mph	"
8.	SH 7 & Little Sandy Creek	Slab	Slab	Yes	None	45 mph	"
9.	SH 7 & Keechi Creek	Slab	Slab	Yes	None	45 mph	"
10.	FM 1860 & TP&L Spillway	RC	RC	Yes	None	No records	"
11.	US 84 & Coryell Creek	RC	RC	Yes	None	40 mph	"
12.	US 84 & Greenbriar Creek	RC	RC	Yes	None	40 mph	"
13.	SH 36 & Leon River	(Steel	and	Yes	None	35 mph	"
14.	US 84 & Leon River	Reinf.	Conc.)	Yes	None	No record, but city traffic	"
15.	US 84 & Dodds Creek	Stl	Stl	Yes	None	40 mph	"
16.	US 84 & Cowhouse Creek	Stl	Stl	Yes	None	40 mph	"
17.	US 84 & Langford Creek	RC	RC	Yes	None	40 mph	"
18.	US 84 & Lampasas River	RC	RC	Yes	None	40 mph	"
19.	US 281 & Partridge Creek	Stl	PC	Yes	None	40 mph	"
20.	US 281 & Cowhouse Creek	Stl	PC	Yes	None	40 mph	"
21.	SH 36 & Bear Creek	RC	RC	Yes	None	45 mph	"
22.	SH 36 & Little Bear Creek	Stl	Stl	Yes	None	45 mph	"
23.	SH 36 & Warring Creek	Stl	Stl	Yes	None	45 mph	"
24.	US 281 & Mesquite Creek	RC	RC	Yes	None	40 mph	"
25.	US 281 & Honey Creek	Stl	PC	Yes	None	40 mph	"
26.	US 79 & GC&SF RR at Milano	Stl	Stl	Yes	None	None	"
27.	US 183 & S. San Gabriel River	Stl	PC	Yes	None	35 mph	"
28.	US 183 & S. San Gabriel River	Stl	PC	Yes	None	35 mph	"
29.	SH 22 & N. Rocky Creek	Stl	Stl	------------------	No Information		--------------------
30.	SH 22 & S. Rocky Creek	Stl	Stl	------------------	No Information		--------------------

The following procedure was used in inspecting all bridges:

1. Walk over the deck from end to end to determine its overall condition.
2. Walk the deck to completely cover it observing closely for defects, with particular attention paid to cracking and spalling in the region where the new concrete joins the old. Look for differences between old and new portions.
3. Inspect the underside of the deck for cracks, marks of leakage. Look for differences between old and new portions. Note type, span, and spacing of beams.

Analysis of Cores from Bridge Decks

Collection of Cores: Cores were cut from the decks for purposes of determining the condition of the concrete, and to detect any differences that might be evident in concrete from different areas of the deck. A total of 109 nominal 4 in. diameter cores were taken from the bridges listed in Table 2. Except for the bridges that were in service at the time of coring, all of the cores were taken a month or so after casting.

Portions of the decks at midspan and nearest the traffic-carrying lanes underwent much greater deflections and higher amplitude vibrations under traffic loads than those areas at or near piers. The cores that were taken from the former areas were designated as disturbed, those from the latter as undisturbed. Cores from all bridges except those from Route 183-Elm Fork bridge were divided between disturbed and undisturbed. Each core from the Route 183 bridge was taken from a disturbed area, and each contained either top or bottom bar steel, or both. In these, the last bridge cores that were collected, emphasis was

TABLE 2. SCHEDULE OF BRIDGE DECK CORES

Bridge	Number of Cores Disturbed Area	Number of Cores Undisturbed Area	Beam Type* Old	Beam Type* New	Core Marks
I-35 & Ave. D Temple	7	5	C-Stl	C-Stl	ADTD1-7 ADTU1-5
I-35 & AT&SF RR, Temple	7	5	C-Stl	C-Stl	RRTD1-7 RRTU1-5
I-45 & FM 517, Dickenson (Houston)	7	5	C-Stl	C-Stl	F7HD1-7 F7HU1-5
I-45 & FM 519, League City	7	5	S-PC	S-PC	F9HU1-7 F9HU1-5
I-10 & Dell Dale Ave., Houston	8	4	S-PC	S-PC	DDHD1-8 DDHU1-4
US 75 & White Rock Creek, Dallas					
Southbound	10	3	S-PC	C-Stl	WRDD1-10 WRDU1-3
Northbound	7	3	S-PC	C-Stl	WRDLD1-7 WRDLU1-3
US 84 & Leon River, Gates-ville	10	6	OH-Stl	OH-Stl	LRGD1-40' LRGU1-56'
Texas Hwy. 183 & Elm Fork Trinity River Irving (Dallas)	10	0	C-Stl	S-PC	EFDD1-10

* C - Continuous; S - Simple; OH - Overhanging; PC - Prestressed concrete; Stl - Steel.

placed on the condition of the concrete immediately surrounding the steel. Locations of cores removed from bridges are shown in Appendix B, Figures B1-B9.

A few cores were unintentionally broken during drilling, and most of these broke at either top or bottom level of steel. Some cores were intentionally broken off below the level of top steel to keep from drilling completely through the slab. Others were drilled the full depth. All cores were marked for identification, then placed in plastic bags awaiting the time that detailed inspection could be made.

Inspection, test, and treatment: Each core was subjected to a careful inspection classed broadly as visual inspection. After the visual inspection, certain cores were selected for soundness tests using ultrasonic pulse velocity measurements, some were investigated for the condition of bond between the steel and concrete by dye tests, and some were tested for compressive strength. Each of these four tests is described below.

1. Visual inspection: Each core was measured, visually inspected, and some were photographed. The first inspection was a search for any obvious distress that could be detected with the naked eye. Several cracks were found in this way, and one core was found to have problems of bond between the steel and concrete. It was also noted in this inspection that the concretes from the different bridges varied in color and density depending on the aggregate type and compaction -- or lack of it -- that could be seen through voids of air bubble size and larger.

Following the initial inspection described above, each core was examined through a two-power hand-held magnifying glass. This examination was possibly the most revealing of all the visual inspections. The glass, being easily and quickly manipulated, made it possible to make a quick assessment of the overall condition of each core. Cracks and voids not seen before were discovered, and cores that needed further study were easily identified through the use of the glass.

Some of the cores that were selected for further study were set aside for an inspection under the microscope. These were sawed to remove about 1/2 inch from the ends, and some were sawed through again at levels below the top steel and above the bottom steel. The sawed surfaces were then ground smooth and polished on a lapping wheel. The polished surfaces were searched with a 20-power stereomicroscope for defects that had not been found earlier, or to further study those that had been found.

After the microscopic examination of the polished surfaces, photographs of the surfaces of selected ones were made. The surfaces were then treated with a fluorescent crack detector. The detector (marketed by Magnaflux Corporation under the trade name PARTEX) contained fluorescent particles suspended in a volatile liquid. It was applied with a squeegee bottle and then brushed smooth. The liquid carrier quickly penetrated cracks, but the particles accumulated on the top of the crack and was not able to enter because of crowding or because of size. After the liquid evaporated, a matter of 10 minutes or so, the surfaces were viewed under a black light that made the particles give off a fluorescent glow. This made the detection of very small cracks quite easy. Figure 4 shows a photograph of a polished core end on which the cracks revealed in this study were marked in white ink by a hand-held pen.

2. Ultrasonic pulse velocity: A number of the specimens that had been trimmed and polished were investigated for soundness in these tests. A 50 kHz pulse was introduced at one polished surface through a piezoelectric transducer and picked up at the opposite polished surface with a similar element. The

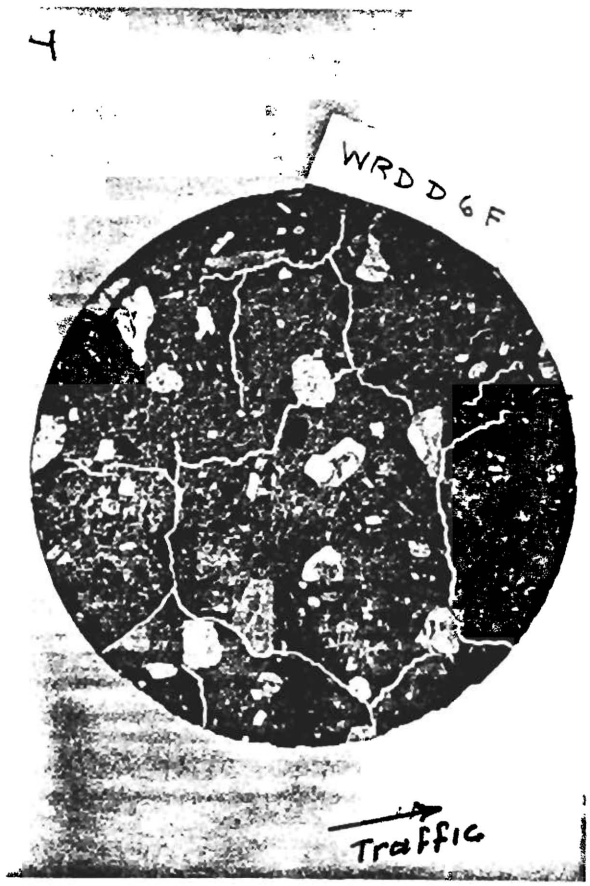


Figure 4. Trace of Cracks on a Polished Surface of a Four-inch Diameter Concrete Core.

piezo mounts were coupled to the concrete through a thin coating of silocone grease. The velocity of the pulses was calculated by the time of travel, determined by cathode ray oscilloscope, and the measured distance between the two faces. All measurements were made along the core axes.

3. Dye tests for bond: Twelve cores, listed in Table 3, were selected for this test. Three bridges provided one pair each, one from an undisturbed area, and the remaining six came from disturbed areas of the two US-75 and White Rock Creek bridges. The former six cores were selected to see if differences occurred in the two areas of the deck, the latter six were chosen to determine the condition at or very close to the joint between the old and new concretes.

Each core was chosen to have a #5 transverse bar -- either a reinforcing bar from the old concrete that extended across the joint to the new concrete, or a new-concrete reinforcing bar that terminated at the joint. The core was cut transversely to contain that #5 bar; if other steel was at the immediate level of the #5 bar, it, too, was contained in the cut. The cuts varied in length from 2 to 5 inches, approximately, so that the steel had about 1 inch of cover.

After cutting, the cores air dried at least one day, then they were set into a vat of red ink in a standard laboratory desiccator, see Figure 5. They were subjected to a negative pressure of about 33 inches of Hg for at least 4 hours. They were then removed and air dried. After they dried they were grooved with a diamond blade for breaking as shown in Figure 6.

The condition of the concrete, the steel, and the imprint area of the steel in the concrete were studied for information on bond between the steel and the concrete. The absence, or near absence, of red ink color in the

TABLE 3. DYE TEST BRIDGE CORES.

Core Mark*	Bridge	Core Location Shown in Figure Listed Below
ADTU4	I-35-Ave. D	B-1
ADTD7	I-35-Ave. D	B-1
F9HU2	I-45 & FM 519	B-4
F9HD2	I-45 & FM 519	B-4
LRGU1	US 84 & Leon River	B-8
LRGD10F	US 84 & Leon River	B-8
WRDD4	US 75 & White Rock Creek, South Traffic	B-6
WRDD7	US 75 & White Rock Creek, South Traffic	B-6
WRDD3F	US 75 & White Rock Creek, South Traffic	B-6
WRDLD2F	US 75 & White Rock Creek, North Traffic	B-7
WRDLD4	US 75 & White Rock Creek, North Traffic	B-7
WRDLD5F	US 75 & White Rock Creek, North Traffic	B-7

*The first 3 or 4 letters represent the bridge. The next letter, U or D, indicates undisturbed area (U) or disturbed area (D). The number represents the core number shown in the figure, in the right-hand column, in Appendix B. The letter F indicates that the core was full depth; the absence of F indicates less than full depth.

Figure 5. Dye Chamber for Core Dye Test.

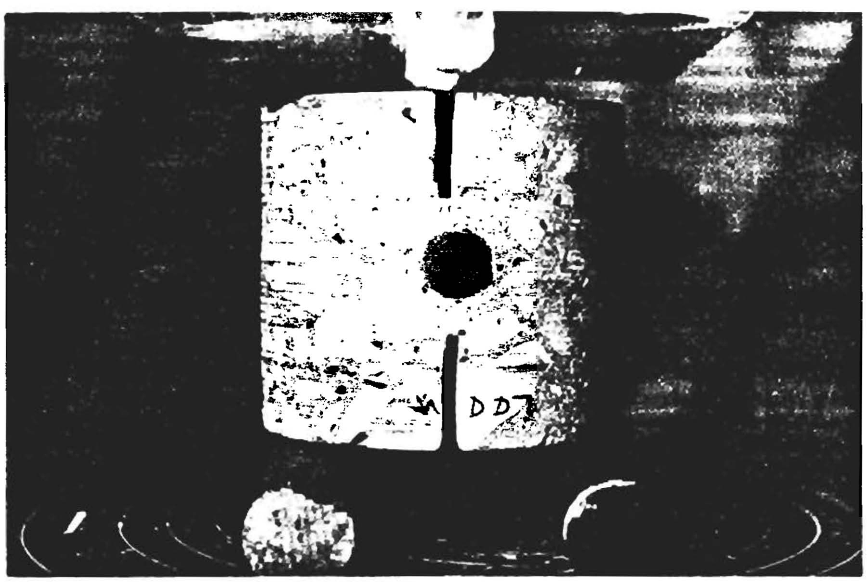


Figure 6. A Grooved Core Prepared for Inspection of Dye Penetration.

imprint area was considered to indicate good bond; the absence, as less than good bond. A well defined, sharp imprint of steel in the concrete indicated good bond; a poorly defined imprint, as poor bond.

4. Strength tests: The specimens that were used in ultrasonic pulse velocity tests were used in tests to determine compressive strength. Since the cores were usually in the order of 7 in. long, all cores had a length to diameter ratio less than 2. The cores, having been previously trimmed and polished at each end, were not capped for the test.

Field Measurements

Vibrations and deflections caused by traffic were measured and recorded for nine bridges. Figures B-1 through B-9 give general information on the bridges, and cross section details are shown in Figures 22.b through 29.b. The break line, the connection between the old concrete deck and the new portion, is indicated in each of the sections.

The structures chosen for measurements were representative of the beam and span types in the SDHPT highway system. Four of them had continuous steel beams, two had simple prestressed concrete beams, and two had a combination of continuous steel and simple prestressed concrete beams. Span lengths varied from 40 to 110 ft.

One representative span of each bridge was selected for instrumentation and measurements. Accessibility of a working area underneath the span and a manageable height of structure above ground were two major factors in the selection of the particular span to be studied.

Figures B-1 through B-9 give the profiles of the test bridges, and point out the instrumented span in each bridge. Deflection gages for beams were attached at the middle of the spans.

Three of the nine bridges were instrumented before the concrete was added to widen the decks. Deflections were recorded before, during, and after concrete placement on those three. In this way it was determined if there was differential movement between steel and concrete which might lead to bonding problems.

Linear potentiometers were used to measure the vertical movements at midspan, and these movements were recorded by Visicorder oscillograms. The potentiometers were attached to weights resting on the ground directly beneath the gaged point. A taut flexible steel cable attached to the deck element mechanically, see Figure 7, transferred the deflection to the potentiometer. The potentiometer signal was then carried to the recording oscilloscope where it was recorded on light-sensitive paper. The resolution of the system was continuous and in the range of ± 0.01 in. A photograph of the system set up on a field site is shown in Figure 8.

The events that were selected for recording were crossings of trucks, usually tractor-trailer units. These trucks were a part of the normal daily traffic, and no details of their weights nor dimensions are available. In only one case were these details known. The truck used in some measurements on the I-35 and Avenue D bridge in Temple was a single axle dump truck loaded with sand. Its gross weight, axle spacings, and speeds were known, but deflection readings under its weight were small when compared to those caused by flowing traffic.

In order to determine if relative movement occurred between the reinforcing steel and the fresh concrete, some deflection gages were attached to reinforcing steel and some to the forms on US 75 and White Rock and Texas 183 and Elm Fork Trinity River bridges. Some dowel bars -- reinforcing steel extending across the break line into the fresh concrete and lapped with the new rebars -- and some reinforcing bars in the fresh concrete were instrumented. The dowels were #5 reinforcing steel spaced approximately 6 in. apart, extending some 18 in. beyond the break line either straight or bent in the horizontal plane.

Laboratory Beam Tests

Laboratory beams were tested to provide further information on conditions that were revealed in the field studies, and to observe the behavior of very early age concrete in repeated flexure. The beams, flexed continuously from

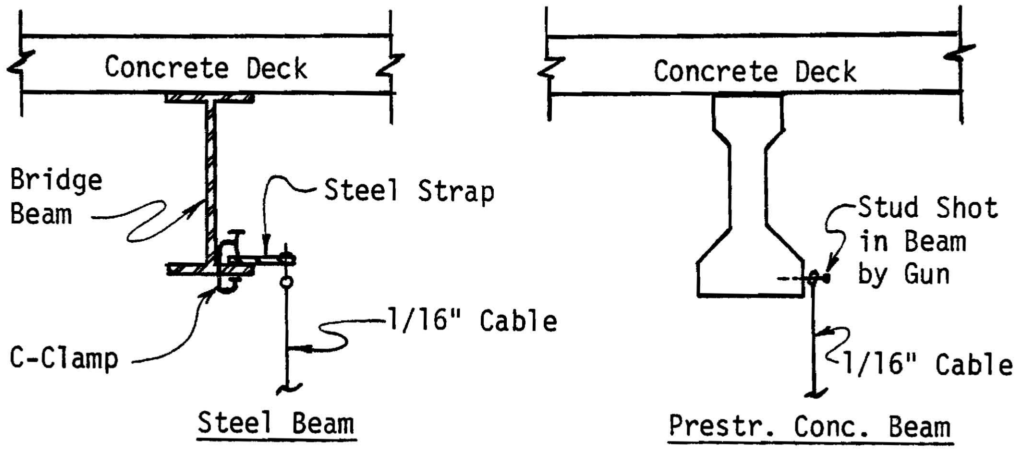


Figure 7. Detail of Attachment of Cable to Bridge Beam.

Figure 8. Linear Potentiometers Set Up for Deflection Measurements Under I-35 & Ave. D (Temple) Bridge.

time of casting, simulated a transverse portion of the concrete deck subjected to traffic loads.

In the study of cores from bridge decks, reported in another section of this report, it was found that there were problems with bond between the concrete and the reinforcing bars (dowels) crossing from the old concrete into the new. One of the objectives of the laboratory beam study was to develop further information on the bond problem. Another objective was to determine if the fresh concrete beams would crack during flexing, and if so, at what curvature and age.

A total of five beams were tested. One of these was not reinforced, and the others were reinforced to simulate the bridge deck. The beams were periodically deflected from time of casting to an age of 24 hours. Cracking behavior was observed during the flexing, and cores were taken and studied after the loading program was terminated.

Details of the beam core study are given in the next section of this report.

The concrete mix used Type 1 cement, maximum size silicious aggregate of 1 in., and was designed for a water-to-cement ratio of 0.41. The 1 cu yd batch was delivered by a local ready-mix company in a 7 cu yd truck and water was added at the fabrication site for workability. Slumps varied between 3 and 6 in. Proportions are shown in Table 4.

The forms were constructed of 3/4 in. plywood fastened with steel angles and bolts. The 7 in. vertical sides were slit at 6 in. intervals to permit flexibility of the form, and the side boards were lined on the inside with plastic sheeting to prevent leakage through the slits. Shear connectors, made of 5/8 in. diameter threaded rod, were fastened to the supports and to

TABLE 4 . CONCRETE MIX PROPORTIONS*

TEST NO.	COARSE AGGREGATE (lbs)	FINE AGGREGATE (lbs)	CEMENT (lbs)	WATER (lbs)
1	1950	1290	545	225
2	1950	1320	545	213
3	1950	1320	555	243
4	1920	1320	555	221
5	1920	1350	560	228

*Quantities given are per cubic yard of concrete.

the bottom of the form. The end of the form at the loaded end was made of 1/2 in. steel plate which fastened to the form by steel angles. All details are shown in Figures 9 through 12.

Each beam was 10 ft 8½ in. long, 12 in. wide, and 7 in. deep. The four that were reinforced had two #5 bars for top steel, two #4 bars for bottom steel, and #4 transverse bars spaced at 12 in. at the level of the top steel. Two #5 reinforcing bar dowels, embedded 21 in. in the beams at the loaded end, passed through the steel end plate of the form and welded to the loading frame. These dowels simulated the reinforcing from the old concrete, on the bridge deck, which extended into the new part of the deck (dowel bars). A photograph of the loaded end is shown in Figure 10.

The concrete was compacted in the form with an internal vibrator, and the top surface was smoothed with a steel trowel. As soon as the top surface was firm enough for covering, wet mats were applied and kept wet throughout the test.

The layout of the beam reinforcement is shown in Figure 9.

Six 6 in. x 12 in. cylinders were cast in cardboard molds and six 6 in. x 6 in. x 20 in. beams were cast in steel molds during the time that each test beam was cast. The cylinders were compacted by rodding; the beams by internal vibrator. These specimens were cured under wet mats by the side of the test beam for 24 hours. At an age of 24 hours, five cylinders and three beams were tested for strength, and the others were placed in the moist room for further curing and to be tested after 28 days.

The test beams, simulating a one foot strip of bridge deck, were supported on flexible supports. They were loaded at one end, and deflections were measured at several points along their lengths. The details of the setup are

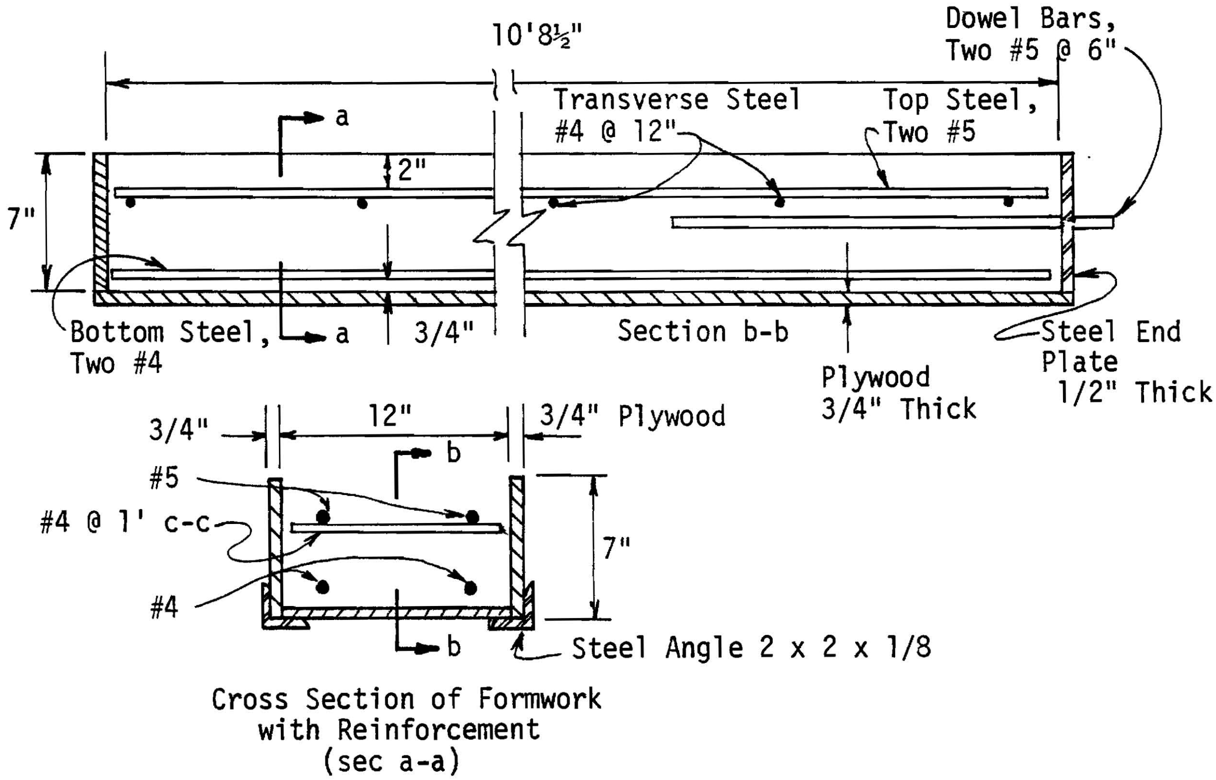
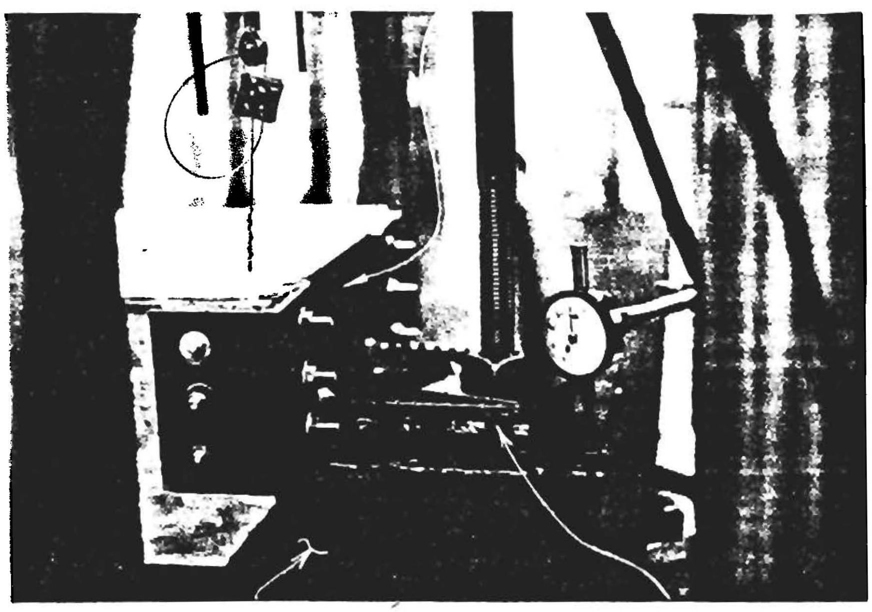


Figure 9 . Layout of Reinforcement and Formwork.

End Steel Plate



Steel Channel

Dowel Bar #5

Figure 10. Detail of Loaded End of Beam.

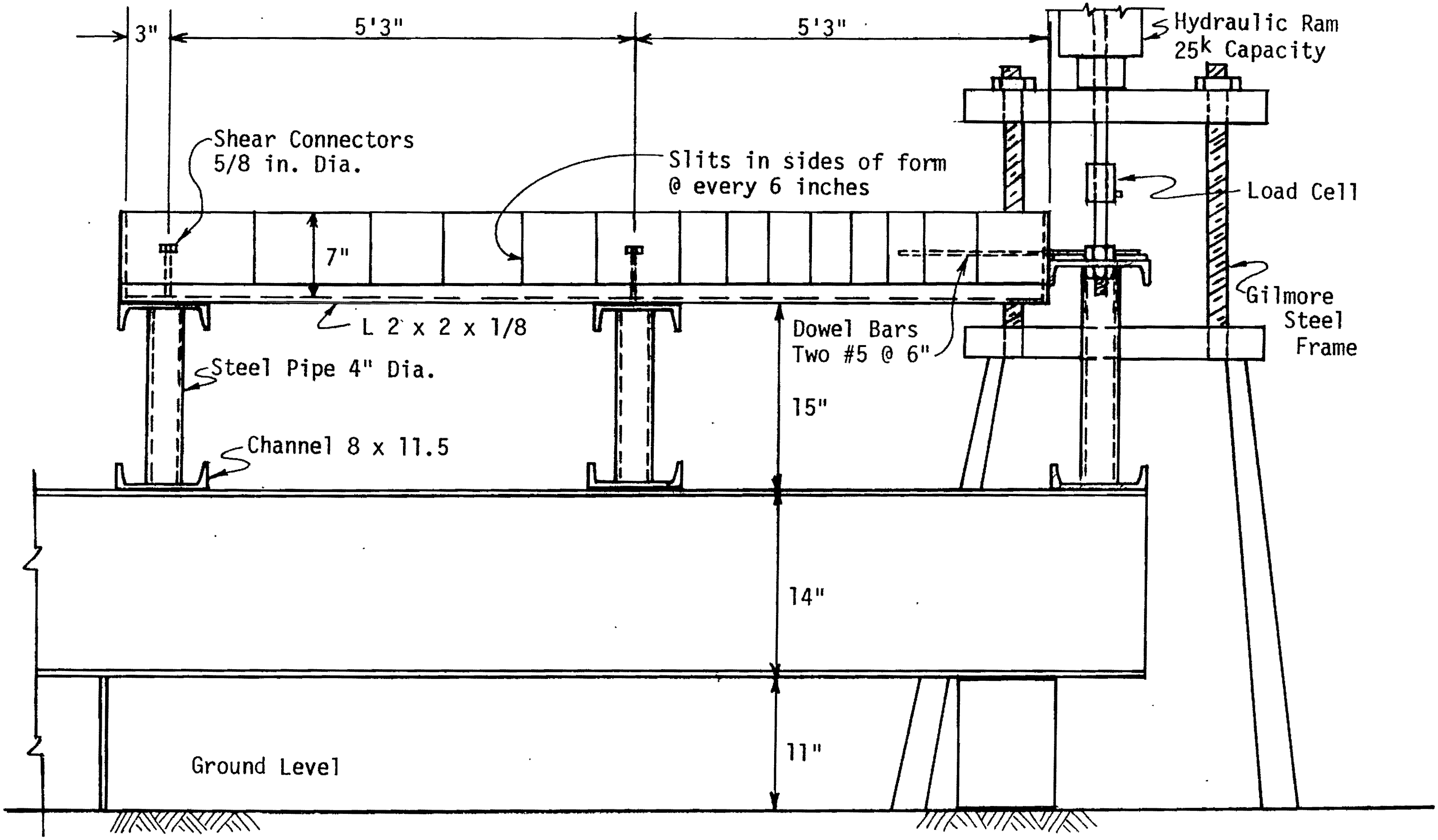


Figure 11. Side Elevation of Loading Frame and Test Specimen Details.

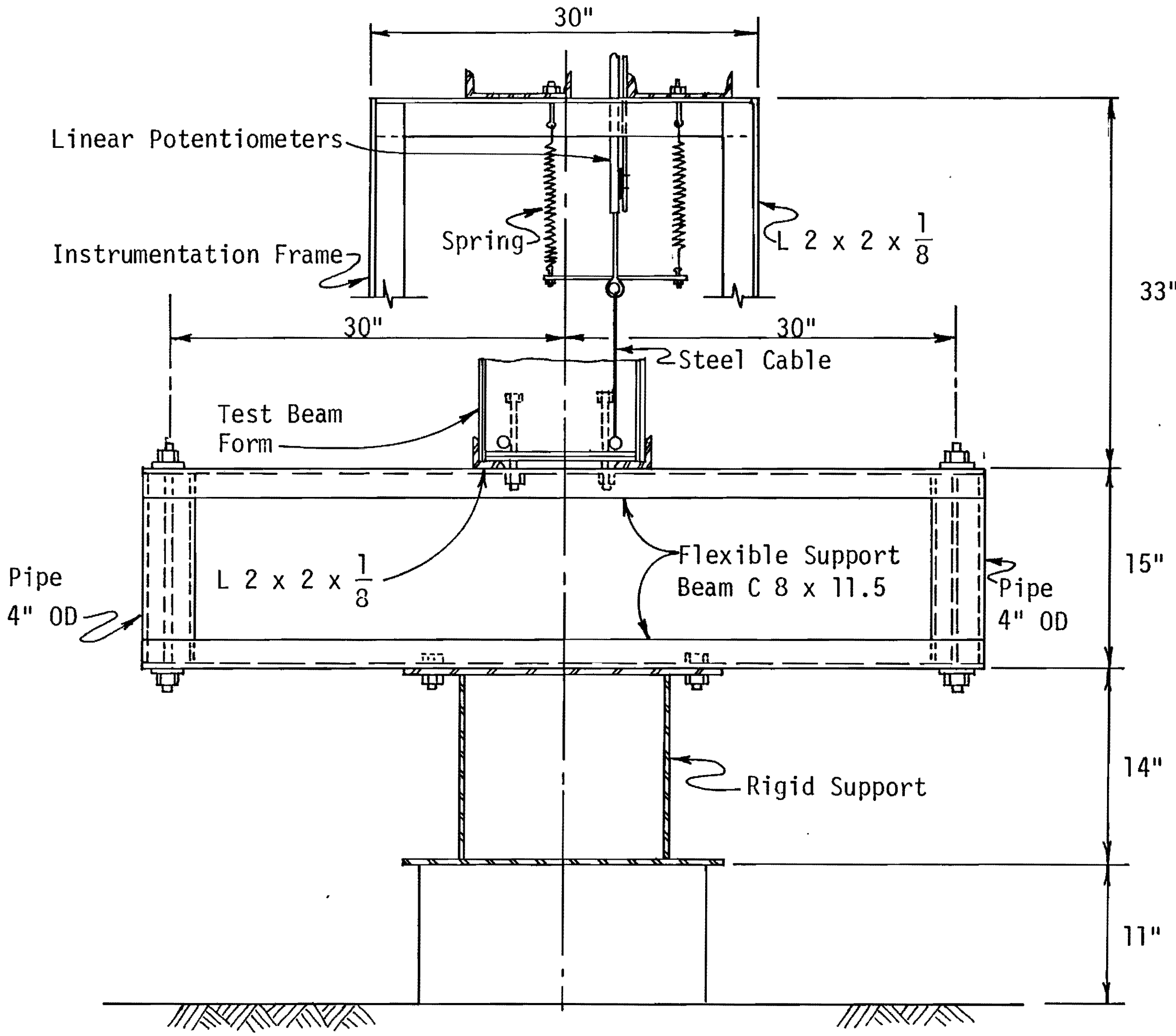


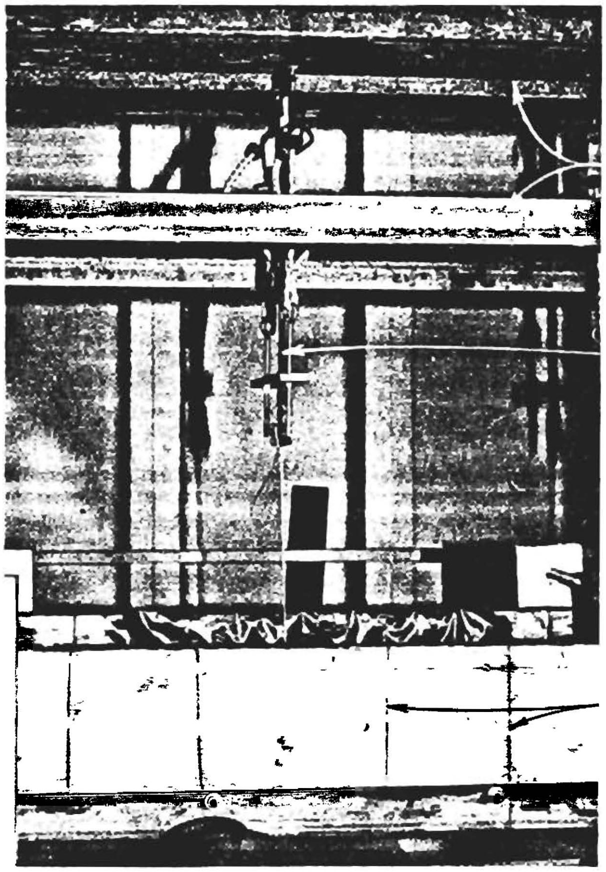
Figure 12. Cross-Sectional Elevation of Test Specimen and Details of Instrumentation.

shown in Figures 11 through 13. The flexible supports, simulating the bridge beams that support the slab, were designed to provide approximately the same curvature in the test as was encountered in bridge deflections measured in the field.

Loading was provided by a hydraulic ram, a Gilmore system, that was programmed to provide a given deflection under the load at five-minute intervals. The deflection and the loading interval were chosen to simulate the passage of heavy trucks on the bridge. The ram forced the flexible supported beam down, which, in turn, transmitted the deflection to the steel end plate attached to the concrete form at the end. The deflection was also transmitted to two #5 reinforcing steel bars welded to the flexible support beam and passing through the steel end plate to extend into the forms for a distance of 21 in., Figures 10 and 11.

Deflections of reinforcing steel in the beam, the reinforcing bar dowels at the loaded end of the beam, the beam form, and the concrete were measured at different points by linear potentiometers. Two measurements were sought in this instrumentation: first, differential movement between steel, concrete, and form; and second, the deflections of the test beam from which curvature could be calculated. The deflections from each of ten linear potentiometers were recorded at selected time intervals by a recording oscillograph. Figures 12 and 13 show the linear potentiometers, used for measuring deflections, in place.

After the test beam was cast, the Gilmore system, programmed to produce a downward deflection cycle every 5 minutes, was activated. Hence, in every cycle the steel channel was deflected downward, carrying with it the formwork and the test beam. The deflection cycle was in the form of a one-half sine



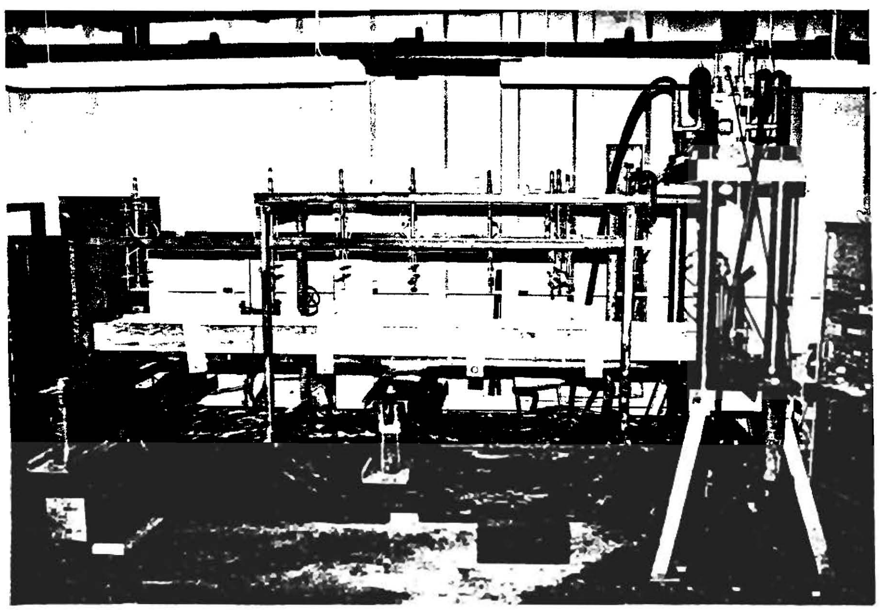


Figure 13. Instrumented Test Specimen during a Test.

wave and had a time duration of one second. The magnitude of the deflection at the loaded end was 0.25 in. for the first four beam tests, and 0.15 in. for the fifth beam test.

In the fourth beam test a ±0.020 in. amplitude vertical deflection at a constant frequency 6 Hz was superimposed on the programmed downward deflection. Table 5 shows a schedule of the loading condition of each beam.

The test beam was subjected to the five-minute deflection cycles for 24 hours, at which time the loading was terminated. During the first eight hours of the test, almost every deflection cycle was recorded. Later they were recorded every half hour only.

The test beam was under close supervision and inspection throughout the entire test. The time, measured from casting, at which any cracks were detected was recorded.

At a station about 2½ feet from the loaded end of the beam, one potentiometer was attached to the formwork, another was attached to the hardened concrete, and a third was attached to the #5 top reinforcing bar. Simultaneous deflection records of the three points were made.

After the test was terminated, all instrumentation was dismantled. The test beam was removed from the form and the test frame and was then thoroughly inspected again. The width of cracks which had formed, and depths that showed on its vertical sides, were measured and recorded.

The test beams were then removed from the laboratory. Later, 3 in. diameter cores were cut from them for a study of concrete condition and the condition of bond between the steel and the concrete.

TABLE 5. BEAM END DEFLECTIONS.

BEAM NO.	END DEFLECTION (in.)	VIBRATION Amplitude (in.)	Frequency (Hz)
1	0.25	None	None
2	0.25	None	None
3	0.25	None	None
4	0.25	±0.020	6
5	0.15	None	None

Laboratory Beam Core Study

A total of 16 three-inch nominal diameter cores were taken from the five beams which were tested. All cores were 7 in. long except for two which were broken at the level of the top steel. Table 6 lists the different cores, and the steel bars contained in each. Figures 14 and 15 show the location of the cores taken along the tested beams.

The aim of this part of the study was two-fold. The first was to determine the depth of penetration of cracks at the locations where flexural cracks have occurred. The second was to investigate the condition of bond between the steel and concrete at the portion of the beam where the dowels were located and where some slight movement between the steel and concrete would be more likely to occur than at other locations. Such movement could be detrimental to bond between the steel and concrete.

All cores were first visually inspected for distress that could be detected with the naked eye. After the visual inspection certain cores were selected for dye tests to determine depth of cracks and to detect any debonding of steel that might have developed.

Ten cores forming two sets were selected for dye tests. The first set of five cores was obtained from the location on the beam where transverse flexural cracks had occurred. These generally occurred in the neighborhood of the center support, about five feet from the loaded end. The second set of five cores came from the portion of the beam which had a #5 dowel bar. This portion of the beam was located between the loaded end and a section at a distance 21 in. from that end, and was considered critical concerning bonding of the reinforcing steel to the concrete. Some slight movement, which might be detrimental to bond, was expected between the steel dowel or top bar and the concrete.

TABLE 6. TEST BEAM CORES

Beam No.	Core Designation	Comments
1	1-1	Beam was not reinforced.
2	2-1*	Contains dowel, top bar & bottom bar.
	2-2*	Contains dowel, top bar & bottom bar.
	2-3*	Has a transverse** crack showing at its top end. No steel.
	2-4*	Has a transverse crack showing at its top end. One top transverse bar runs through the core.
3	3-1*	Contains dowel, top bar & bottom bar.
	3-2	Contains dowel, top bar & bottom bar.
	3-3*	Has a transverse crack showing at its top end. One top transverse bar runs through the core.
4	4-1*	Contains dowel, top bar & bottom bar.
	4-2	Core is 3 in. long only. It broke after cutting through the top steel.
	4-3*	Contains dowel, top bar & bottom bar.
	4-4	Contains dowel. Parts of the top and bottom bars are chipped.
	4-5*	Has a transverse crack. Core is cracked on a horizontal plane @ 4 in. from top. Probably this is due to coring operation.
	4-6	Has a transverse crack. It contains top & bottom bars and a vertical all threaded rod (shear connector).
5	5-1	Contains dowel, top bar & bottom bar.
	5-2*	Has a transverse crack.

*Cores which were dye tested.

**Cracks which run across the width of the beam are termed transverse cracks.

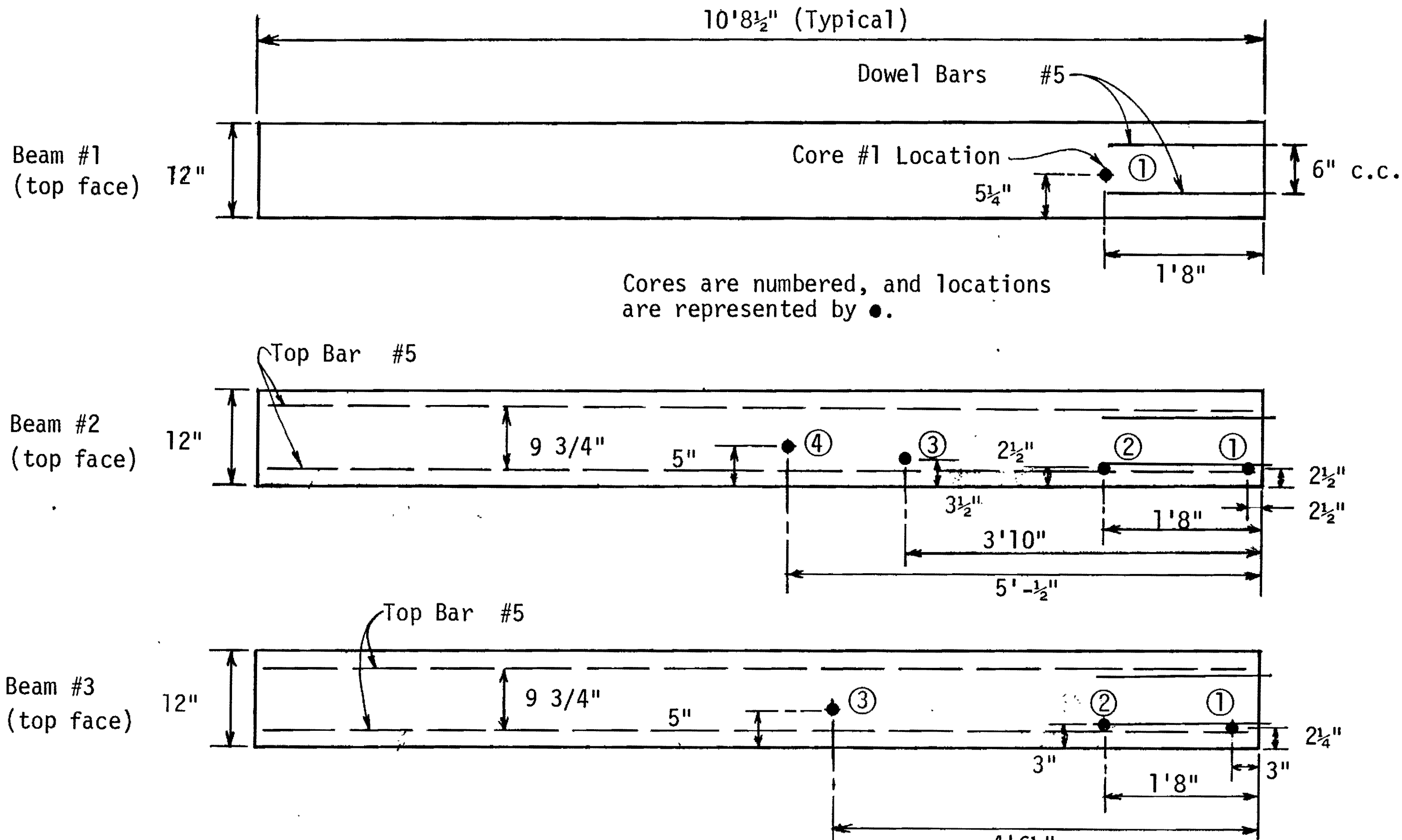


Figure 14. Location of Drilled Cores along Test Beams #1, #2, & #3.

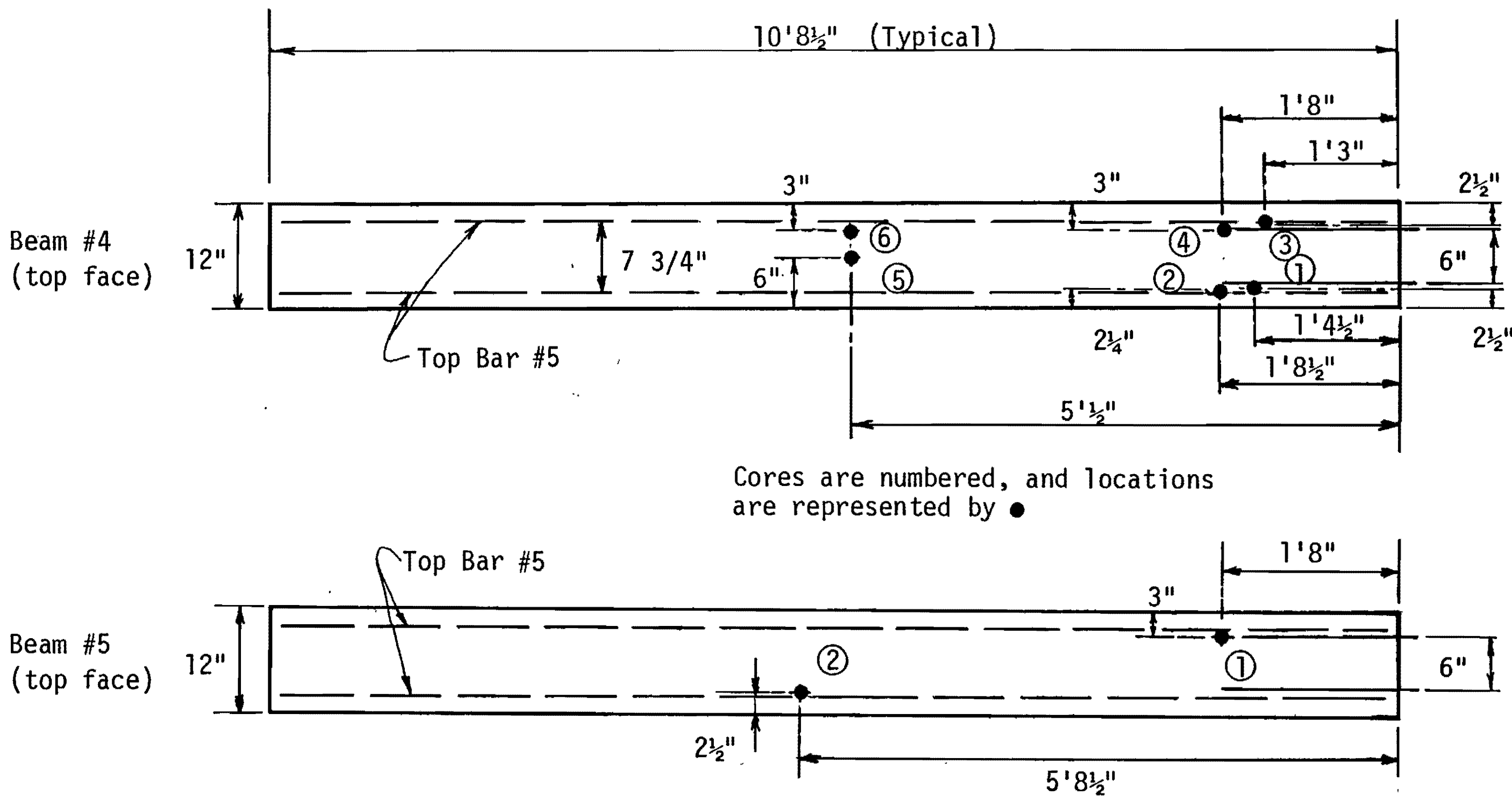


Figure 15. Locations of Drilled Cores along Test Beam #4 and #5.

The procedure of a dye test may be summarized in the following steps:

1. Cores were sawed normal to their axis into lengths that varied from 4 to 5 inches.
2. Each of the 5 in. parts was immersed in a red dye in a dessicator. The dye was diluted to a water-thin consistency.
3. The dessicator was connected to a vacuum pump which was operated to pull 33 in. Hg. negative pressure; thus displacing air in the concrete with dye.
4. The cores were left under negative pressure for at least 3 hours; then they were taken out and dried with paper towels.
5. Cores were then sliced normal to a transverse crack or split open along a steel bar depending upon whether the depth of crack or condition of bond, respectively, was to be investigated.

The ten cores, which were selected for dye tests, are designated by an asterisk in Table 6. These cores were from reinforced beams. The five cores -- 2-3, 2-4, 3-3, 4-5 and 5-2 -- were studied to determine how deep flexural cracks penetrated into the beams. The cores were sawed longitudinally (parallel to their axes) and normal to the transverse crack which showed at the top of each of them. The depth of penetration of the dye, measured from the top of the core, was considered as a measure of the crack depth.

The other five cores -- 2-1, 2-2, 3-1, 4-1, 4-3 -- and 5-2, which was previously studied for crack penetration, were selected for the purpose of studying the condition of bond between the reinforcing steel and the concrete. After treatment with the dye, the cores were split open along the reinforcing bars. Absence, or near absence, of dye color in the bar-cavity and sharp imprints of re-bars were interpreted as a condition of good bond. Presence of dye color and ill-defined rebar imprints in the bar-cavity were considered as poor bond.

RESULTS

Visual Inspection of Bridges

Four methods of joining new deck to old were encountered in the inspection. The first, illustrated in Figure 1(a), shows the construction joint between two supporting longitudinal bridge beams. It was made when the bridge on US 183 over the North San Gabriel River in Williamson County was widened, and it is now overlaid with asphaltic material. No steel crosses the joint to form continuity between the existing and added materials, and a crack, spalled by differences in deflection between these two parts, runs the length of the joint. This is the only joint of its kind encountered in the inspection, and it is no longer used in bridge widening in the state of Texas.

The second type of joint, shown in Figure 1(b), was encountered only in a few short-span, reinforced concrete beam bridges. There appeared to be no deck problems associated with this type of joint.

The third type, shown in Figure 1(c), was used in most of the bridges. No evidence of problems in the deck that could be associated with this type of joint, nor that shown in Figure 1(d), was found.

The inspection revealed very few instances of distress, and none that could definitely be attributed to traffic during construction. In a few bridges a thin crack along the longitudinal construction joint was visible, but all of them were in good condition. Except for these longitudinal cracks, the cracking patterns in the newer material were about the same as those in the older parts.

As discussed earlier, the negative bending is probably greatest over the most interior new beam -- beam 1' in Figure 3, and it is in that region that longitudinal cracking from flexure was anticipated. That area was inspected closely to see if cracking was more prevalent than in other areas. Among all of the bridges inspected, only the bridge across the Leon River at Gatesville shows any evidence of such cracking. A few longitudinal cracks in that area were found, but they did not give a consistent pattern over the entire deck. Cores were taken later from that deck to find out more about it, and the cores are described in another section of this report.

On the basis of the on-site visual inspection, it is concluded that there is no clear evidence that the concrete was damaged by placement and curing while traffic continued to use the bridge.

Bridge Deck Core Analysis

Visual inspection: The bridges that were cored are listed in Table 2, and details of core locations are given in Appendix B, Figures B-1 to B-9. The overall evaluation of the conditions of the cores is given below, and the details of the condition of each appears in Appendix B, Table B-1.

A total of 109 cores were taken from the 9 bridges studied. The ends of 62 of these, 56 percent of the total number, were sawed, polished, studied through a 2X glass, a 20X microscope, and treated with the fluorescent crack detector. All of the cores that were cut and polished displayed random cracking on one or both ends. These cracks were very narrow and could be located only by using the crack detector on the polished surfaces. They appeared to be of the nature of microcracks that have been reported by others (8,9), but they are listed as hairline cracks in Table B-1 because they were so clearly defined with the fluorescent particles under black light. The crowding of the particles along these cracks probably made them appear much wider than they actually were, although no crack widths were measured.

Some of the random, narrow cracks intersected the vertical sides of the cores, but it was impossible to follow them along the rough, unpolished sides. In order to get an idea of their depth, 1/4 in. thick slices from the ends of eleven cores were polished on both sides and treated with the detector. It was found that the crack pattern on one side was partially duplicated on the other side, indicating that some of the cracks extended through the slices. If these cracks began on the rough top or bottom surfaces of the cores, these slices would indicate that some of the cracks were at least approximately 3/4 in. deep.

Twenty-one of the 55 randomly cracked cores were taken from undisturbed areas of the deck, and 34 came from disturbed areas. The 21 make up 58 percent of all undisturbed area cores, and the 34 make up 47 percent of the disturbed area cores. From this it can be concluded that the random cracking was not caused by traffic disturbance during placement and curing of the deck concrete.

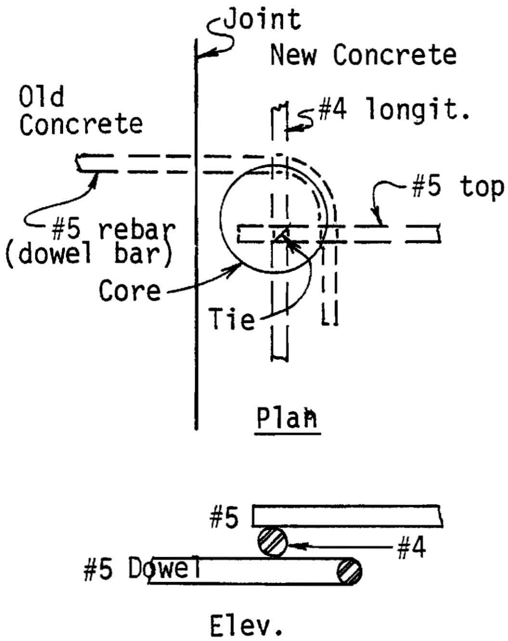
The wider cracks extending across the top or bottom surfaces could sometimes, but not always, be traced along the sides of cores to the interior of the concrete. Of the 109 cores inspected, 20 had these wider type cracks. One of these cores, WRDD10F, had three cracks, and another, RRTU3, had two cracks, for a total of 23 cracks. Five of the 23 were parallel to traffic, 14 perpendicular, and four diagonal. Since most of them were perpendicular to traffic -- transverse deck cracks -- one is led to think of them as shrinkage cracks, although very few of them extend the full depth of the slab. The cracks parallel to traffic -- longitudinal deck cracks -- are oriented as one would expect flexural cracks between the bridge beams to be, but four of these five cracks were found in undisturbed regions. There was no indication that transverse bending causing longitudinal cracking was a problem.

The core WRDD10F, from the joint between old and new concrete at the middle of the central span of a 70-90-70 ft continuous unit, southbound traffic bridge, provided the clearest evidence of disturbance found in the study. The top surface was cracked in the general direction of traffic, the concrete above the top mat of steel was badly damaged, and the interface of steel-to-concrete had the appearance of puddled plastic concrete, and all traces of bar deformation marks were lost.

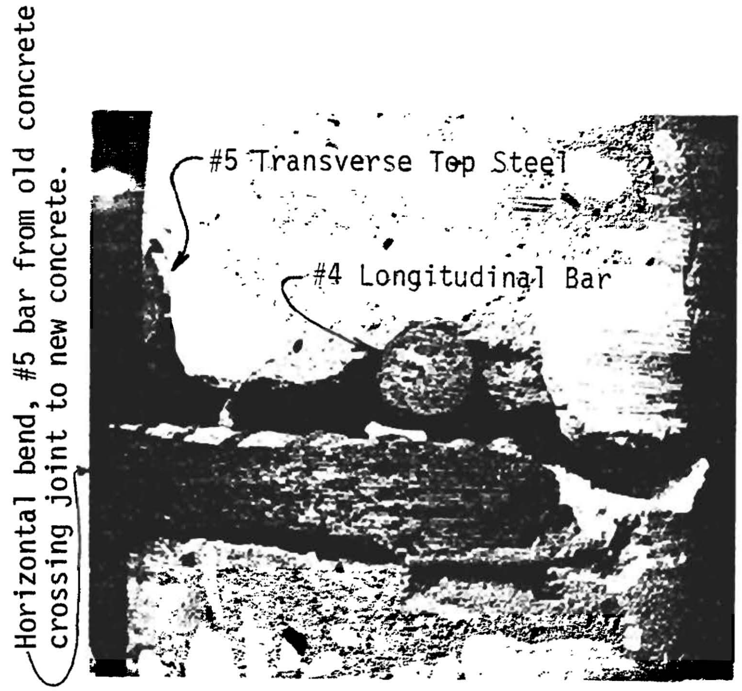
Figure 16 shows details of the core. The location of the core with respect to the joint, and the relative position of the steel in the core, are shown in (a). The photograph in (b) shows the two pieces, top and bottom, fit together as closely as possible after being broken apart in coring. The end of the top #5 bar can be seen at the left side face of the photograph. The longitudinal #4 bar end is seen just below the #5 rebar to which it is tied. The #5 rebar from the old concrete, called the dowel bar here, was split by the drill at its 90° bend, and it is seen just below the #4 bar. Light shows through the horizontal crack, between the #4 and the bent dowel. The puddled condition of the concrete is seen in Figure (c), especially in the #4 bar imprint. Figure (d) shows the interface of the top portion of the core. In this view the end of the #5 rebar in the new concrete can be seen in the core, and the tie between the #4 and the #5 bar is clear.

When the photographs were made, the loose curved piece of dowel was laid in the position seen in the photographs. However, it was most likely in contact at the top of the bar, having the 1/4 in. crack below it, when the core was cut. The imprint of this bar on the bottom interface looks the same as that of the #4 bar.

It is believed that the intermittent deflections caused by traffic on the old deck caused the fresh concrete to puddle at the level of top steel. This



(a) Layout



(b) Side View

(c) Bottom Part

(d) Top Part

Figure 16. Core WRDD10F from Southbound Traffic Bridge, US 75 & White Rock Creek.

puddled material had no strength, and it formed a layer of unconsolidated particles that were flushed out by the coring water. This left the void seen in the photographs.

In this phase of the investigation only core WRDD10F provided clear evidence of damage caused by traffic. Further evidence from other cores was found in the dye test, which is discussed in another section.

Ultrasonic pulse velocities: The data from these tests are given in Table 7. All measurements were carefully made and reproducibility was good. With the exceptions of cores ADT, RR, and F9H, values are reasonable.

The very low velocities from cores F9HU5F and F9HD4F, from bridge number 6 in Appendix A, cannot be explained from the information collected in the tests. The concrete was made of a siliceous aggregate, it was a hard material and it appeared to be sound, without any particular defects.

The inconsistencies in pulse velocities of the ADT and RR cores, taken from bridges numbered 2 and 3 respectively in Appendix A, cannot be explained by information collected in the tests. Both were made of a mixture of siliceous and limestone coarse aggregate. The ADT cores were hard and sound, and they contained a few fossilized shells. The RR cores had many fossilized shells, and the concrete was considerably softer than the ADT material, but the unit weights were about the same. The lower pulse velocity of the two from ADT cores came from the disturbed area of the deck, whereas the lower velocity of the RR cores came from the undisturbed area.

In comparing the pulse velocities with the condition of cores in Appendix B, and with compressive strengths of the cores, nothing was found in this test to point out damage of the concrete from passing traffic.

TABLE 7. PULSE VELOCITIES AND COMPRESSIVE STRENGTHS OF BRIDGE DECK CORES.

CORE	LENGTH (in.)	DIAMETER (in.)	STEEL (T=Top; B=Bottom)	AVG. PULSE TIME (sec)	PULSE VELOCITY (ft/sec)	P_{max} (lb)	COMPRESSIVE STRENGTH (psi)	REMARKS
ADTD6F	7 7/8	4	1T & 1B	100×10^{-6}	6,562	52,200	4,154	RHC*, top & bottom.
ADTU5F	6 15/16	4	1T & 1B	51.33×10^{-6}	11,263	55,950	4,452	RHC, top & bottom.
RRD1	5 5/8	4	1B	45.33×10^{-6}	10,340	35,050	2,789	RHC, top & bottom. Crack .01 x 2" dp. on bottom.
RRU5	6	4	1T & 1B	63.66×10^{-6}	7,854	35,850	2,853	RHC, top & bottom.
LRGD3F	6 1/8	4	1T & 1B	38×10^{-6}	13,432	42,150	3,354	RHC, top & bottom. Thin diagonal crack on top.
LRGD4F	6 3/16	4	1T & 2B	35.66×10^{-6}	14,459	44,250	3,521	RHC, top & bottom. Bottom steel had voids around it.
DDHD4F	6 3/16	4	1T & 2B	45.00×10^{-6}	11,458	39,050	3,106	RHC, top & bottom.
DDHU1F	6 5/16	3 15/16	None	47.00×10^{-6}	11,192	44,800	3,679	RHC, top & bottom.
F7HD5F	7 3/16	4	2T & 1B	49.33×10^{-6}	12,142	57,500	4,576	RHC, top.
F7HD3F	6 3/4	4	2T	43.33×10^{-6}	12,982	78,000	6,207	RHC, top & bottom.
F9HD4F	6 9/16	4	2T & 2B	81.66×10^{-6}	6,697	87,815	6,988	RHC, top & bottom.
F9HU5F	6 5/8	4	1T & 1B	81.33×10^{-6}	6,788	59,925	4,769	RHC, top & bottom. Hairline crack on top perpendicular to traffic.
WRDD5	6 7/16	3 15/16	1B	44.33×10^{-6}	12,101	59,650	4,899	No visible defects.
WRDD2F	6 7/8	3 15/16	1T & 1B	48.00×10^{-6}	11,936	44,800	3,679	RHC top & bottom. Numerous air bubbles.
WRDLU3F	7 1/2	4	2B	56.33×10^{-6}	11,095	42,400	3,374	RHC, top & bottom.

*Random hairline cra

Results of dye tests: The 12 field specimens used in these tests were prepared as explained under Tests and Inspection Procedures. The cores are listed in Table 3, and the conditions determined from this test are given in Table 8.

The concrete from the two I-35 bridge decks in Temple was noticeably softer than that from other locations. The dye penetrated completely through some of the one-inch aggregates in that concrete, and it colored the mortar to a depth of 1/2 in. in some places. Core ADTU4 from an undisturbed area of the I-35 and Avenue D bridge in Temple permitted the dye to color the full length of embedment of the #5 bar in the core. The bar imprint was distinct and sharp, giving no indication that bond was deficient; and, being from an undisturbed area, no reason for debonding could be given. It was concluded that the relatively soft nature of the material, and not debonding, accounted for the dye penetration seen around the bar in this core.

The disturbed specimen ADTD7, from the I-35 and Avenue D bridge, displayed some of the penetration characteristics as its mate, ADTU4, but penetration around the steel was not complete. A photograph of core ADTD7 is shown in Figure 17. A portion, but not all, of the embedded area of the #5 bar was colored. Bits of mill scale from the bar pulled away and stuck to the concrete when the bar was removed. Even though the core was taken from a disturbed area of the deck, there was no apparent disturbance of bond between the steel and concrete.

A set of cores, one from disturbed area and the other undisturbed, from the I-45 overpass of FM 519 in Houston were tested. The concrete in this deck was a very hard, dense material, considerably different than that of the Temple bridge from which the cores discussed above were taken. Dye penetration into the mortar seldom reached 1/4 in., being more on the order of half that distance for the most part. In the undisturbed core F9HU2, the dye penetrated into the

TABLE 8. BRIDGE CORE CONDITIONS.

Core	Condition
ADTU4	One #5 top bar, parallel to traffic in the core. Dye penetrated the mortar at this soft concrete about 1/2 in. and it penetrated completely through 1 in. C.A. The full imprint of this bar was colored with dye in this relatively soft material.
ADTD7	This specimen had a #5 bar with 3/8 in. clear cover on one side and 3/4 in. clear cover on the other side. The ends of the bar were exposed as cut on the surface of the core. Dye penetrated 3/4 in. from each end, and through the 3/8 in. clear cover. No penetration developed through the 3/4 in. clear cover. Bar imprint was colored 3/4 in. at each end and on the side with 3/8 in. cover. A portion of the imprint with 3/4 in. cover was not colored. Mill scale broke away in all areas of the bar when the specimen was split open, and bond was apparently good. The penetration of dye was primarily through concrete, not through a void between steel and concrete.
F9HU2	Two #5 bars lying side to side, perpendicular to traffic, and one #4 bar parallel to traffic, were embedded in this core. The dye penetrated the mortar 1/8 to 1/4 in.; it penetrated 1/2 in. into the core from the ends of each bar. The interior regions of bars and bar imprints were free of dye color.
F9HD2	One #5 bar perpendicular to traffic, tied to two #4 bars lying side by side running parallel to traffic, were embedded in this core. The dye penetrated the mortar 1/16 to 1/8 in. and a like distance from each end of the #4 bars. It did not penetrate around the ends of the #5. The interior regions of bars and bar imprints were free of dye color.
LRGU1	This core contains a #5 top bar perpendicular to traffic tied to a #4 bar parallel to traffic. The dye penetrated the mortar about 1/4 in., but the interior of the bars and their imprints were free of dye color.
LRGD1OF	A #5 top bar was cored out with the concrete at the bend in the bar. A #4 parallel bar was tied to the bent #5. Dye penetrated the mortar about 1/4 inch; it penetrated one inch along the bottom of the incline of the #5; penetration at the horizontal end at the #5 and at each end of the #4 was about 1/4 in. The interior regions of bars and bar imprints were free of dye color.
WRDD4	This core was cut at the edge of the joint where the new concrete was doweled to the old. The #5 dowel was taken in the core at the bend of the bar, along with a 2 in. length of #5 transverse bar, and a #4 longitudinal bar which was directly

Core	Condition
	below and in contact with the dowel. Mortar was penetrated 1/4 in; the interfaces of several coarse aggregates were colored; penetration along the #5 dowel and the longitudinal #4 was complete. The imprint of the dowel was indistinct.
WRDD7	This specimen contained a #5 top bar, perpendicular to traffic. Dye penetrated the mortar approximately 1/4 in. About one-fourth of the bar imprint was completely colored with dye -- from one rib located at about 7 o'clock upward to about 10 o'clock. The remainder was clear of color except for about 1/4 in. at each end.
WRDD3F	The #5 top bar, perpendicular to traffic, permitted incomplete penetration of dye to about one inch from each end. The imprint of the bar is sharp and distinct, indicating good bond. The #5 bar parallel to traffic, tied to the other #5 bar and to the #4 bar, allowed penetration to 1-1/4 in. at one end and 1/4 in. at the other. The remainder of the imprint was not colored. This bar left a sharp, well defined imprint, indicating good bond. The #4 bar permitted penetration only for about 1/4 in. at each end. Its imprint was sharp and distinct, indicating good bond.
WRDLD2F	The embedment surface of the #5 dowel (from old concrete to new concrete) was completely colored with dye; a part of it was coated with dry paste; voids line its surface; and the imprint was not well defined. The dye penetrated about 1/16 in. deep around the imprint surface, indicating soft porous material there. The imprint of the #4 bar, which was tied to the #5 bar, was very poorly defined, and it was coated with dye. Dowel movement with respect to the plastic concrete would account for the condition.
WRDLD4	Dye penetrated the mortar to a depth of about 1/4 in. The #5 top bar, perpendicular to traffic, and centered 2-1/2 in. below top surface, had very poorly defined embedment imprint. The imprint surface was completely colored with dye; it was full of voids, dry paste, and had numerous uncoated fine aggregate particles on its surface. Clearly, this bar moved with respect to the plastic concrete.
WRDLD5F	Dye penetrated the mortar to a depth of about 1/4 in. The #5 top bar, parallel to traffic, was very close to the vertical face of the core. It had only 1/8 in. cover on the outside, and there was a void space visible at the top of this bar. Dye around this bar was complete, coloring the entire embedded area. When the bar was lifted from its embedment it was coated with a film of colored paste. The imprint was not sharply defined and there were numerous small voids and air pockets in that imprint. This condition was probably caused by movement of the bar with respect to the plastic concrete.

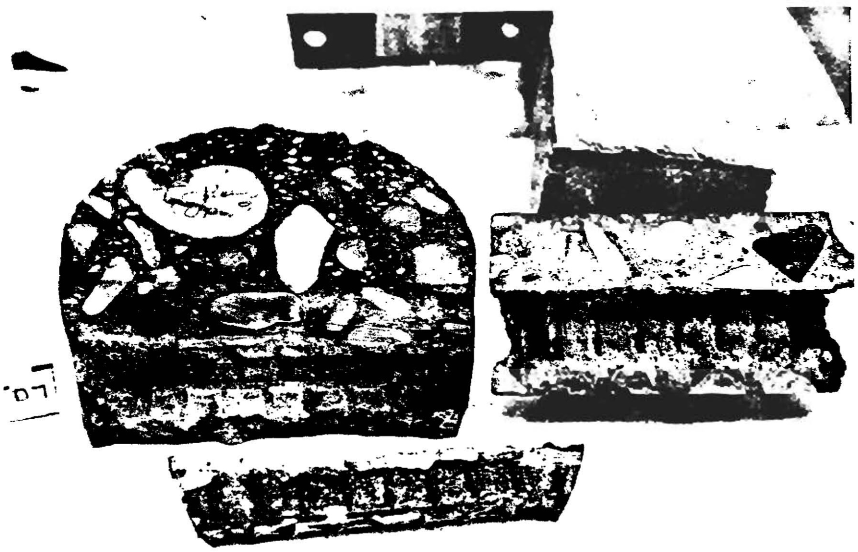


Figure 17. Core ADTD7 after Dye Treatment.
(Light areas in the bar imprint are not colored;
dark areas are colored.)

concrete 1/2 in. from the end of the two #5 bars lying side by side. This core is shown in Figure 18. The core F9HD2 contained two #4 bars side by side, and one #5 bar. The dye penetrated 1/4 in., at the most, from the ends of these bars. There was no evidence of debonding of steel in these cores -- neither from dye penetration nor from the condition of the material in the bar imprint area.

The other set of disturbed-undisturbed cores from the same bridge came from the Leon River bridge on US 84 at Gatesville. The imprints of reinforcement were sharp in each of these cores, and the penetration of dye from ends of all straight bars was about 1/4 in. Penetration along a #5 inclined bar in LRGD10F extended along the incline about one inch from the face of the core, but gave no evidence of debonding.

Three cores taken from the disturbed region of the southbound traffic bridge on US 75 over White Rock Creek in Dallas were tested. Penetration into the mortar was approximately 1/4 in. in each of the cores.

Core WRDD4, taken at the joint between old and new concretes, of the 50 ft span contained a #5 dowel bar, a #5 transverse bar, and a #4 longitudinal bar. The dowel and the #4 bar were in direct contact with each other, see Figure 19. Dye penetration was complete over the imprints of these two bars, and the surface imprint of the dowel was blurred and the deformations were not sharply defined. The mortar on the imprint surface of the dowel was porous, but that of the #4 bar was well compacted and its outline was clear. This test showed that there was relative movement of the dowel, which extended from the older slab to the fresh concrete, with respect to the new concrete.

Cores WRDD7 and WRDD3F were taken from the 50 ft span southbound US 75-White Rock Creek bridge. There was some dye penetration along some of the steel in these cores, but the full bar imprint was never completely colored.

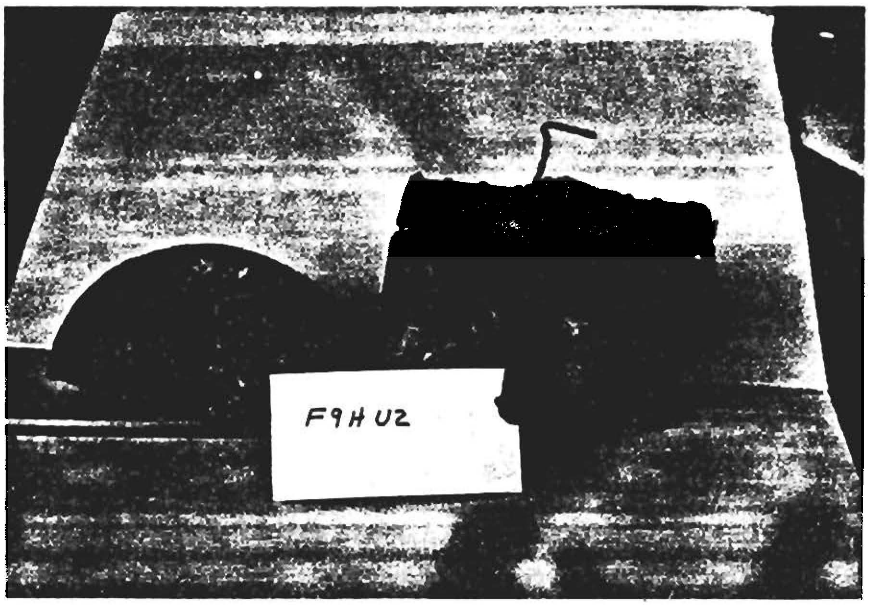
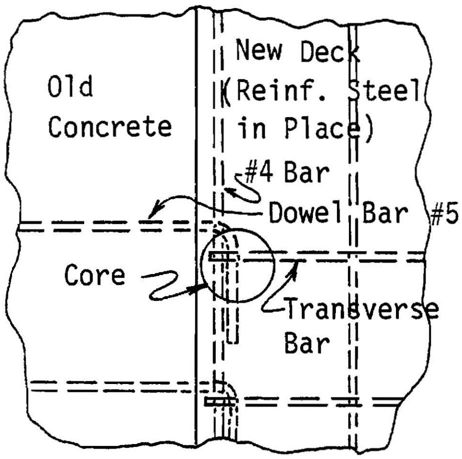
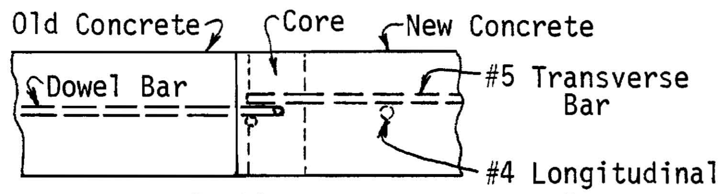


Figure 18. Core F9HU2 After Dye Treatment.



Plan



Section

The dowel bar is a reinforcing bar in the old concrete.

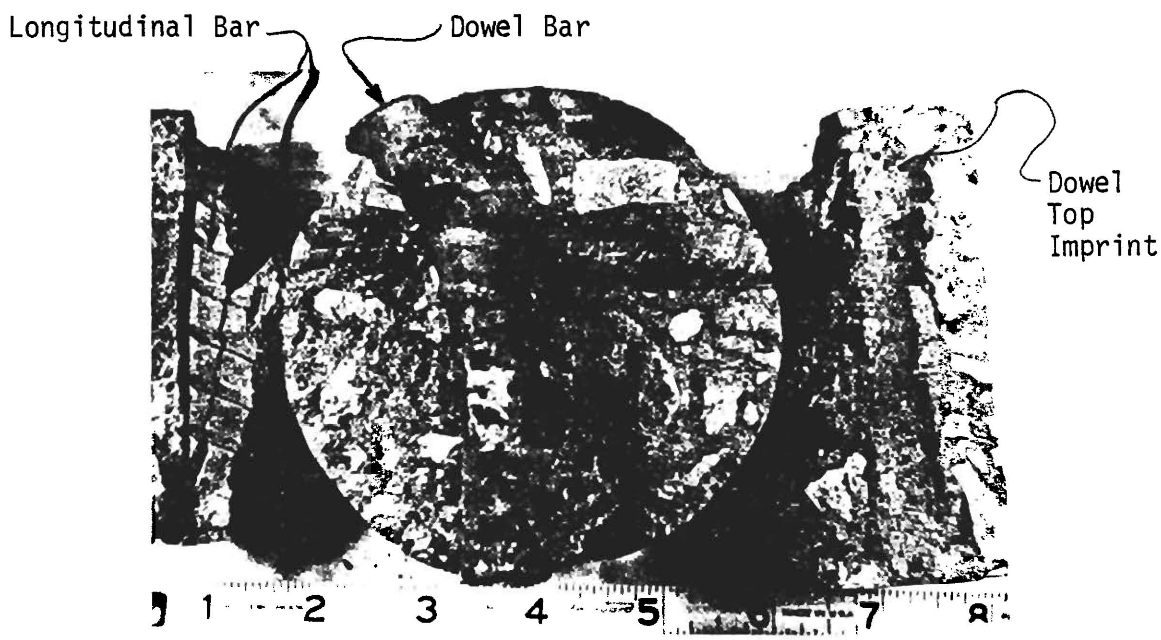


Figure 19. Core WRDD4; Location and Photograph.

The imprint of the bars in the mortar was sharp and clear in both of these cores, and there appears no good evidence that there was debonding in either case. Core WRDD7 is shown in Figure 20.

The remaining three cores of this test series came from the 90-ft span of northbound US 75-White Rock Creek bridge. Each of these cores was taken at or adjacent to the joint of old to new deck. One of them, WRDLD2F, contained the #5 dowel that crossed the joint; the other two contained steel that was in contact with dowels in the vicinities of the cores.

Each of these three cores, WRDLD2F, WRDLD4, and WRDLD5F, bore unmistakable evidence that reinforcing steel had moved with respect to the fresh concrete. Bar imprints in the concrete were colored with dye, concrete adjacent to the bars was porous and poorly compacted, dried colored paste coated the imprint areas, and in some areas the concrete appeared as if it had been puddled, bringing water to the interfaces, then dried. These three cores are shown in Figure 21.

Cores WRDLD1F and WRDLD2F were taken at the joint of the 90-ft span, northbound traffic of the US 75-White Rock Creek bridge, Figure B-7. Each of these cores contained a part of the old concrete and a part of the new. These two parts fell apart upon removal from the core bit, showing no bond whatsoever of the new to the old. The interface between the parts was smooth and it offered little or no resistance to vertical movement. If this lack of bond permitted even slight vertical movement of old concrete with respect to new, it could contribute to the absence of bond around the steel discussed above.

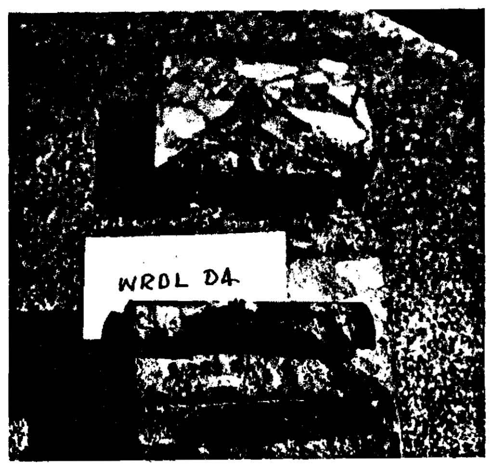
The dye tests showed that there does not appear to be any problem of debonding due to vibrations of steel in regions a few feet away from the joint between the old and new deck. But, there is clearly differential movement of

Before Treatment

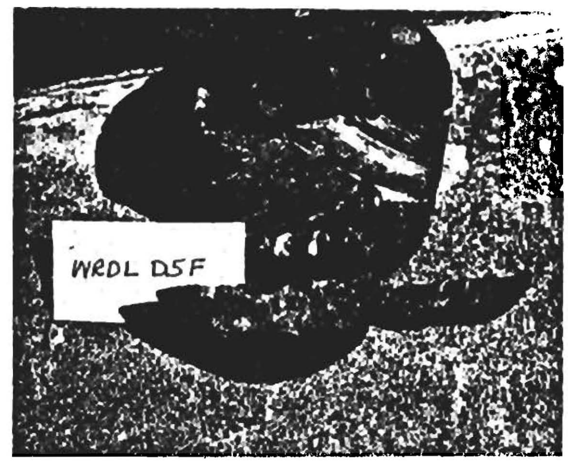
After Treatment

Figure 20. Core WRDD7 Before and After Dye Treatment.

Core WRDLD2F



Core WRDLD4



Core WRDLD5F

Figures 21. Cores After Dye Treatment.

steel with respect to fresh concrete at the joint, and this movement causes void areas around the steel in that immediate area.

Strength tests: Compressive strengths of the cores are given in Table 7. The effects of various reinforcing bar steel in the cores, and different length-to-diameter ratios probably influence the strengths given in the table. Each value represents one break. Cores from disturbed and undisturbed areas from four bridges show differences in strengths, but in only one set of these cores, F9HD4F and F9HU5F, is the strength of the disturbed area core of appreciable difference than that of its undisturbed area mate. There was nothing in the appearance of these two cores that can explain the difference in their strengths.

The data from these tests do not indicate a consistent difference between strengths of disturbed and undisturbed cores.

Deflections and Vibrations of Bridges

The bridges and their measurements procedure described in the previous sections resulted in several hundred feet of 10 channel oscillograph records. Typical oscillograph traces, for the different bridges, are shown in Figures 22(a) through 29(a). In some of these, the frequency of free vibration of the bridge can be easily identified. The general shape of the trace is that for an influence line of the deflection being measured at midspan.

The transverse distribution of beam deflections at midspan are plotted along the bridge cross section as shown in Figures 22(b) through 29(b). These deflections are the maximum (peak) deflections for a certain traversing vehicle. Hence, the profile of the deflected cross section is displayed. These deflections were used to compute the transverse curvatures of the new concrete deck due to differential deflections between the beams. A simple method, which assumes that an arc of a circle passes through each three successive points, was used in computing the curvatures. This method is outlined in Appendix C.

Transverse surface curvatures of the widened concrete decks computed for the different bridges are listed in Table 9. This table also lists the maximum differential deflections and the natural frequencies for every bridge. The term differential deflection refers to the relative deflection between the existing edge beam of the old concrete deck and the first new beam of the new deck. The natural frequencies represent a full deck of hardened concrete in some cases, and a deck of part hard and part plastic concrete in other cases.

The largest measured curvature in fresh concrete was about 0.114×10^{-4} in.^{-1}. This occurred in the 7 3/4 in. concrete deck of Texas Highway 183 and Elm Fork Trinity River bridge (see Table 9). Hilsdorf and Lott (4), in an

Table 9. Natural Frequencies of Bridges and Maximum Transverse Bending Curvature.

Bridge	Beam Type* Old	Beam Type* New	Span (ft)	Nat. Freq. (Hz)	Max.** Diff. Defl. (in.)	$R_{min.}$*** (in.)	$\frac{1}{R_{min.}}$ $\frac{1}{10^5 \text{ in.}}$
I-35 & Ave. D	C-Stl	C-Stl	60	5	0.032	465,333	0.215
I-35 & AT&SF	C-Stl	C-Stl	70	5	.041	300,976	.332
I-45 & FM 517	C-Stl	C-Stl	54	5	.120	405,140	.247
I-45 & FM 519	S-PC	S-PC	110	3.5	No traffic on old deck.		
I-10 & Dell Dale Ave.	S-PC	S-PC	87	4	.060	118,580	.843
US 75 & White Rock Creek							
Southbound	S-PC	C-Stl	50	10	.032	429,684	.233
Northbound	S-PC	C-Stl	90	5	.058	120,869	.827
US 84 & Leon River	OH-Stl	OH-Stl	67.5	4.5	.058	145,455	.688
Tex 183 & Elm Fork Trinity River	C-Stl	S-PC	50	6	.040	87,803	1.139

* C - Continuous; S - Simple; OH - Overhanging.

** Maximum differential deflection between the outside existing beam and the adjacent new beam under new deck.

*** R - Radius of curvature of the deck in transverse bending.

experimental investigation, statically deflected reinforced concrete slabs 6 in. thick between 2 and 4½ hr after mixing. They found that appreciable cracking did occur when deflections caused a concrete surface curvature of approximately 5×10^{-4} in.$^{-1}$. It is known that for an elastic model, concrete curvature is inversely proportional with the distance from the neutral axis to the top fibers of the slab, which is a function of the slab thickness. Thus the thickness of the slab has some bearing on the curvature in the fresh concrete. In order to have a fair comparison between the curvature suggested by Hilsdorf and Lott and that measured for the bridge, the former curvature must be modified somewhat to take into consideration the effect of slab thickness. Assuming the same strain at cracking, Hilsdorf and Lott's curvature can be related to bridge curvature by multiplying the former by a factor of 3/4. Comparing both curvatures, it is found that the curvature suggested by Hilsdorf and Lott and classified as being critical for cracking, after modification, is about 33 times the maximum found in the bridge deck. Thus it can be said that cracking, due to transverse bending, is unlikely unless much larger curvatures would occur due to some unusually heavy vehicle. Results of visual inspection and core studies of the test bridges, reported earlier in this report, support the above argument as flexural cracks due to transverse bending were not detected, or could not be associated with curvature, in any of the bridges.

Beam tests with traffic-simulated loading were conducted as part of this research and reported later in this report. A concrete surface curvature of approximately 0.36×10^{-4} in.$^{-1}$ was required to develop cracking in the beam which was 7 in. thick. This curvature was compared with the maximum curvature which occurred in the fresh concrete of the widened bridge deck of about the same thickness as that of the beams. It was found that the smallest curvature causing cracks in the beams was about three times the maximum curvature in the widened bridge deck. This is further evidence as to why cracking did not occur in the widened decks.

The measured natural frequencies of the test bridges ranged between 3.5 and 10 Hz. Bridges for which measurements were being taken during placement of the concrete deck showed no appreciable change in natural frequency after placement. In only one bridge, US 75 and White Rock Creek (90 ft span) in Dallas, did the frequencies change with the added width. Before widening, this bridge had a natural frequency of 5 Hz, and it reduced to 3.5 Hz, directly after placing concrete on the widened part. About 12 hours later, after casting, the frequency of the bridge increased to 5 Hz, which can be explained by noting that the natural frequency is proportional to a bridge inertia and inversely proportional to its mass. When concrete is first placed and is still plastic, the dominant parameter is the mass and thus a reduction occurs in frequency. As the concrete hardens and stiffens, the inertia of the bridge increases and consequently its frequency, too, increases.

In US 75 and White Rock Creek (90 ft span) bridge, a #5 dowel bar and a #5 top transverse bar were instrumented prior to concrete placement. Both bars had the same natural frequency as that of the bridge beams, even when the frequency changed after casting. Since reinforcing steel had the same vibration frequency as that of the bridge system, relative movement between steel bars and the plastic concrete would be unlikely. In such a situation, bonding problems between steel and concrete would not be expected.

In Texas Highway 183 and Elm Fork bridge four steel bars were instrumented at seven locations as shown in Figure 30. The 1/16 in. thick corrugated steel form, the existing steel edge beam of the old deck, and the neighboring prestressed concrete beam of the widened deck were also instrumented. Deflections and vibrations of these points were recorded before, during, and after the placing of the fresh concrete on the widened part. The data indicated that the vibration frequency for the different instrumented points was the same at all times. Hence, the reinforcing steel mat, the form, and the bridge beams are all vibrating at the same frequency. This shows that relative movement between the reinforcing steel and the plastic concrete did not occur, in this case.

Measurements were recorded for I-45 and FM 519, Houston bridge after it was widened and brought back into service. During the time that measurements were taken, the traffic was rerouted, and was active on the widened deck only. Thus, transverse bending curvature due to traffic on the old deck could not be computed.

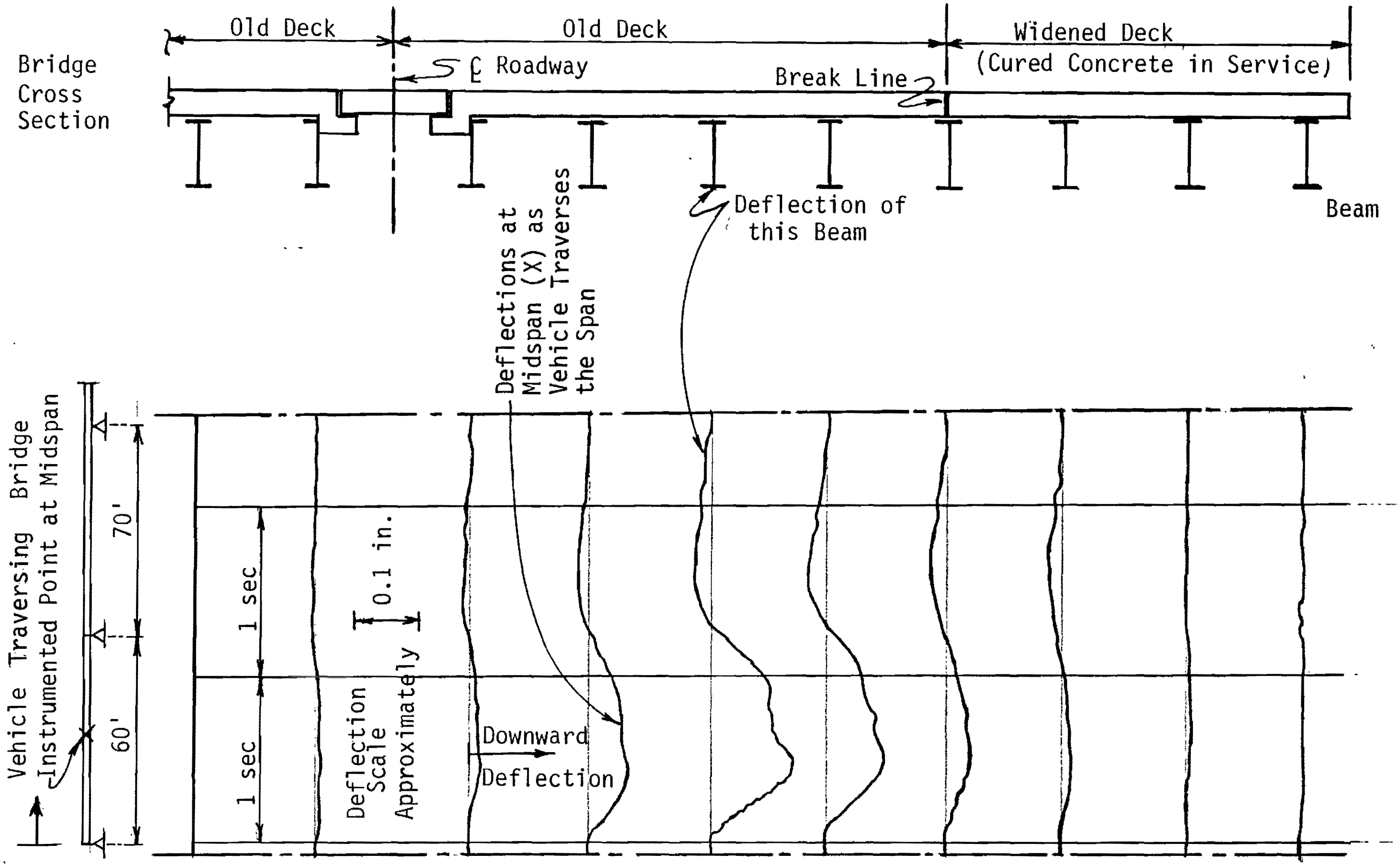


Figure 22.a. I-35 and Avenue D, Temple, Midspan Deflections of Beams due to One Crossing

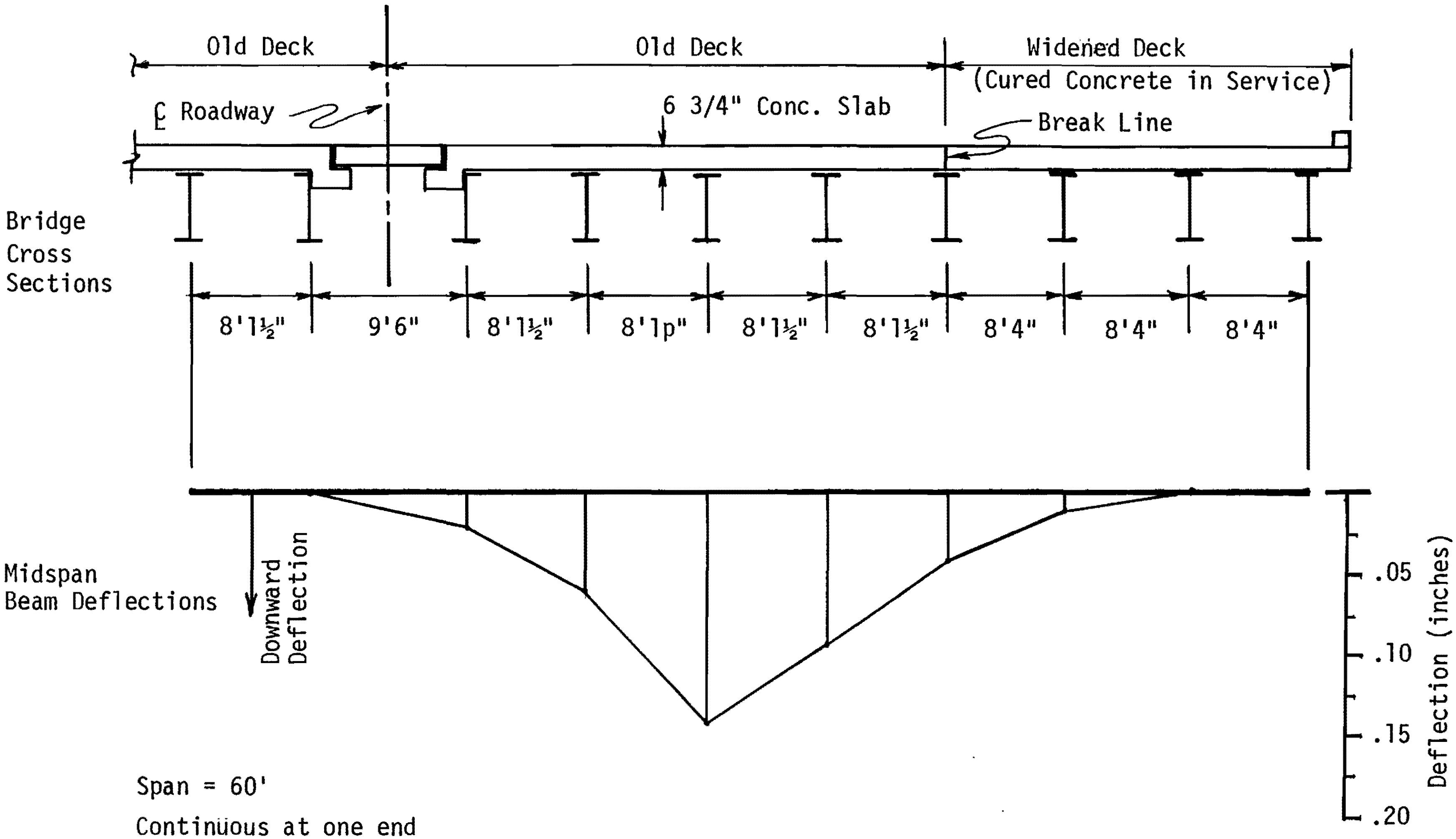


Figure 22.b. I-35 & Ave. D, Temple, Maximum Midspan Beam Deflection for One Crossing.

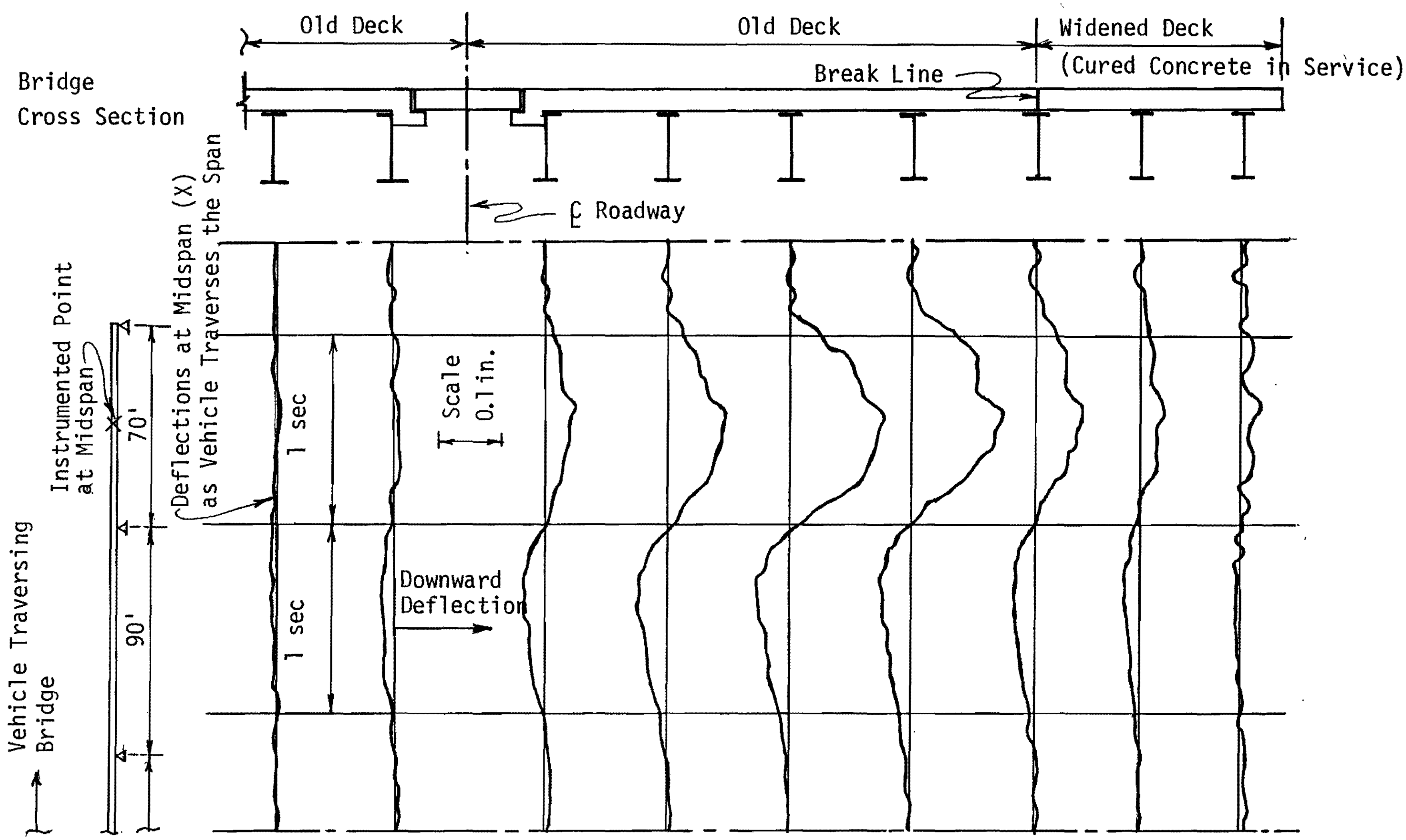


Figure 23.a. I-35 & AT&SF RR, Temple, Midspan Deflections of Beams due to One Crossing.

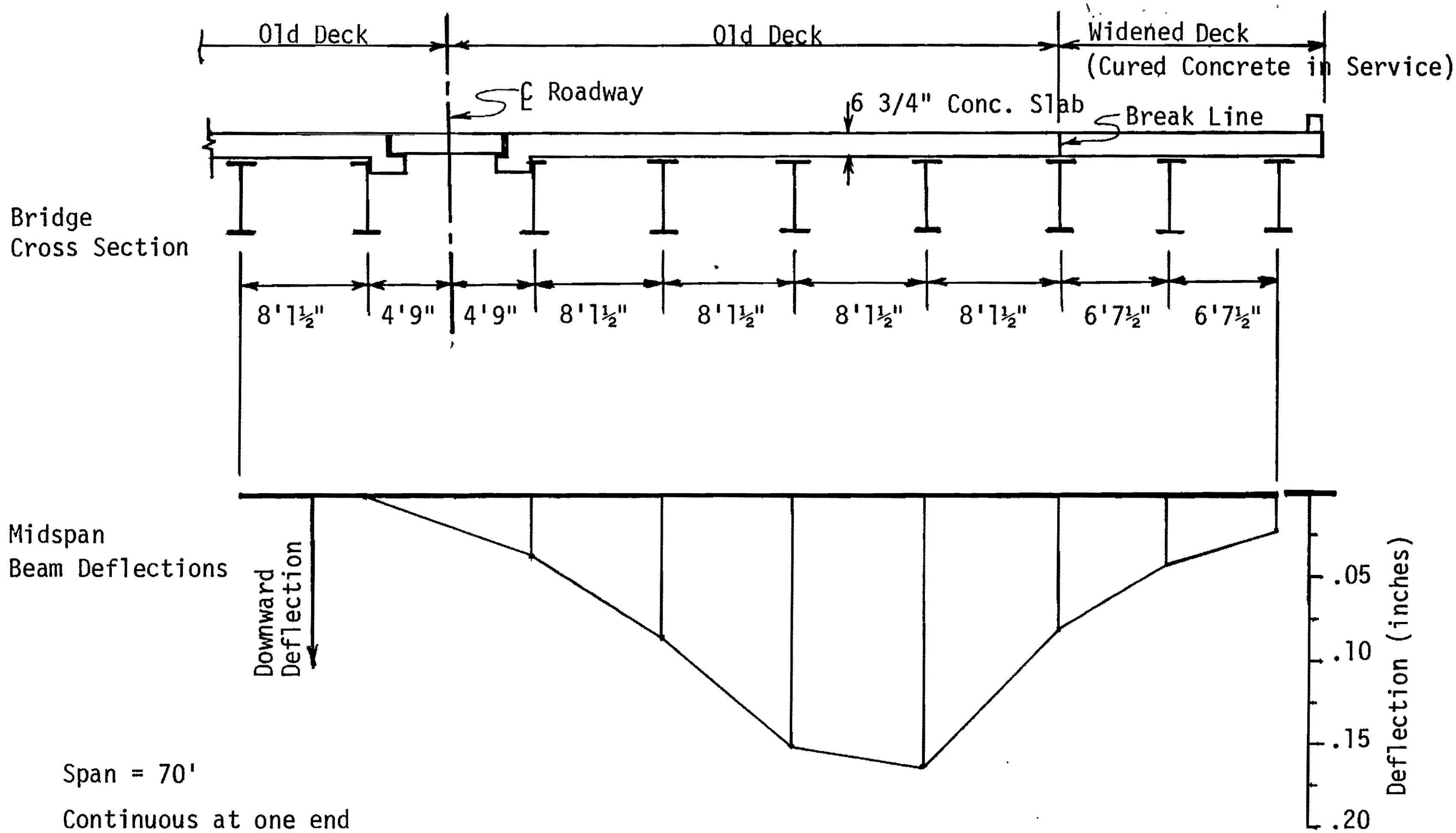


Figure 23.b. I-35 & AT&SF RR, Temple, Maximum Midspan Beam Deflection for One Crossing.

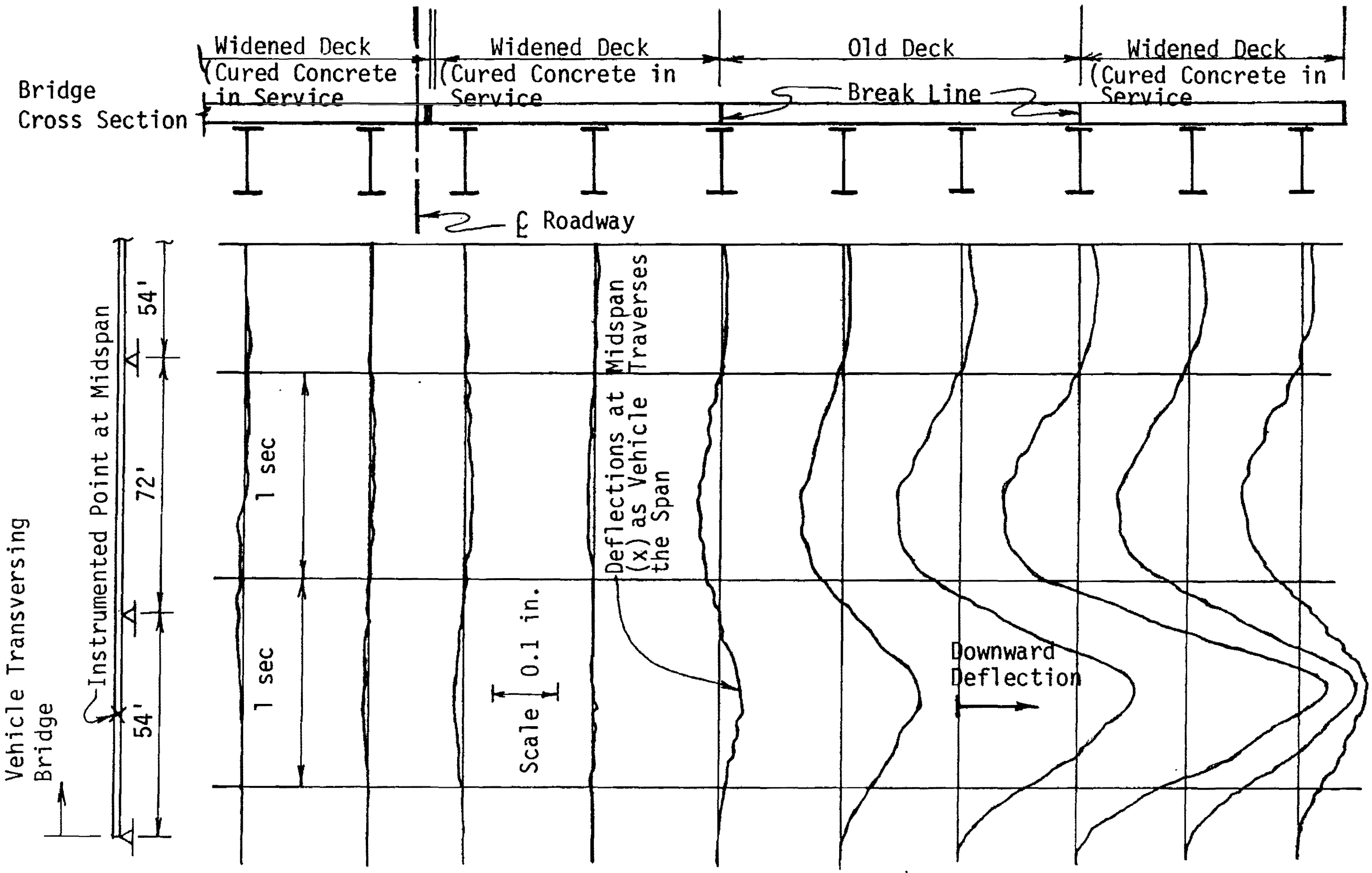


Figure 24.a. I-45 & FM 517, Dickenson (Houston), Midspan Deflections of Beams due to One Crossing.

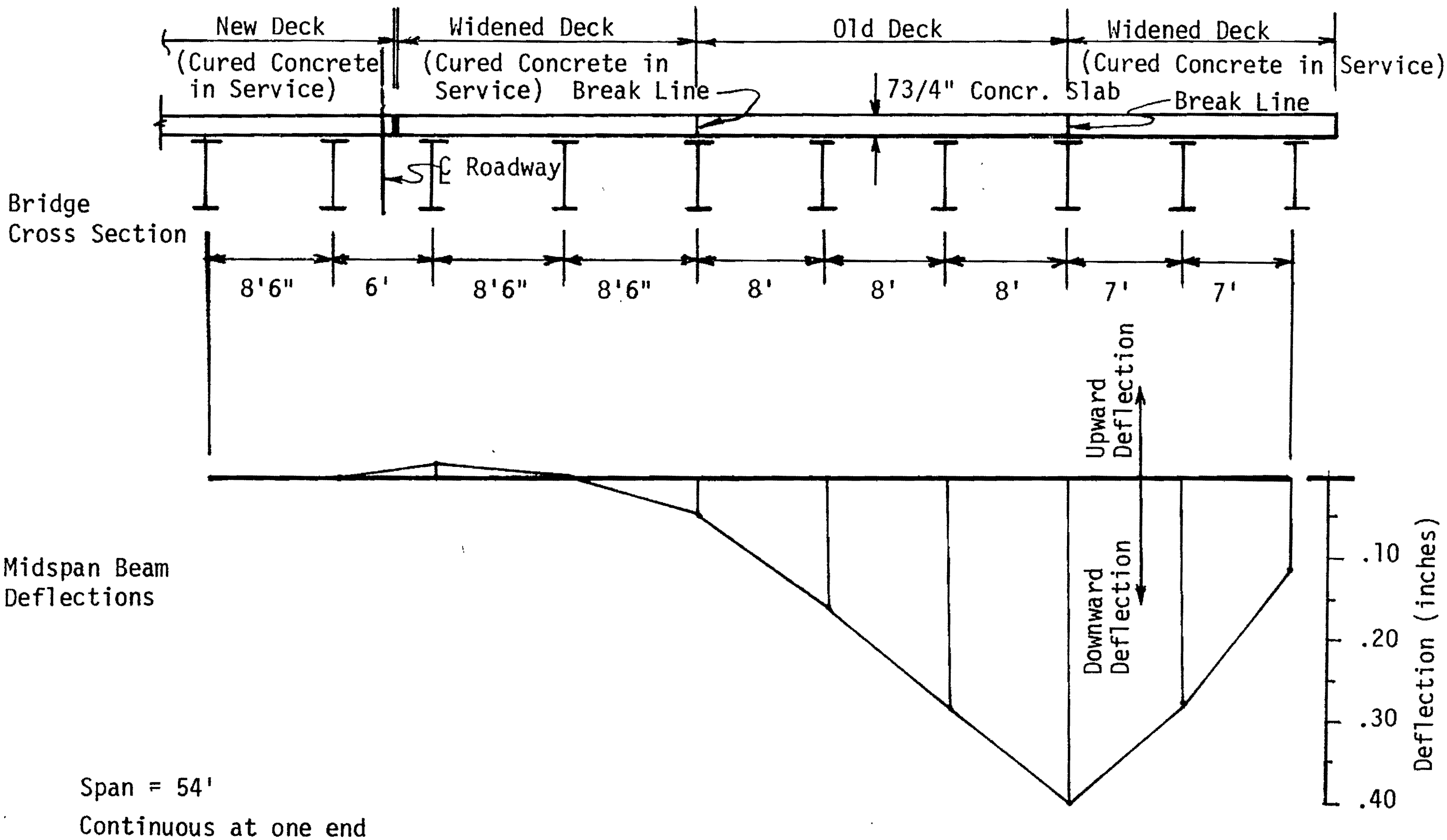


Figure 24.b. I-45 & FM 517, Dickenson (Houston), Maximum Midspan Beam Deflections for One Crossing.

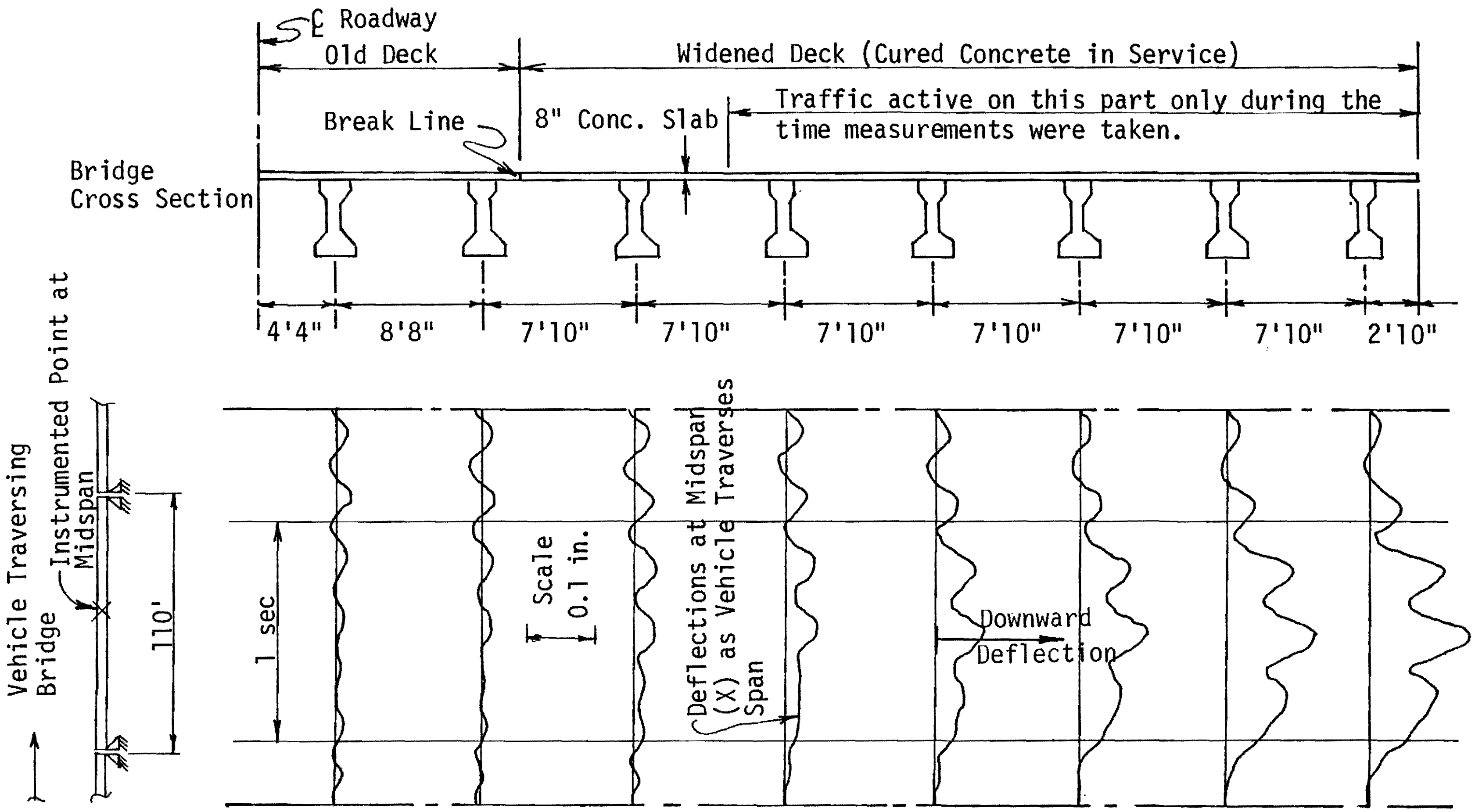


Figure 25. I-45 & FM 519, League City (Houston), Midspan Deflections of Beams due to One Crossing.

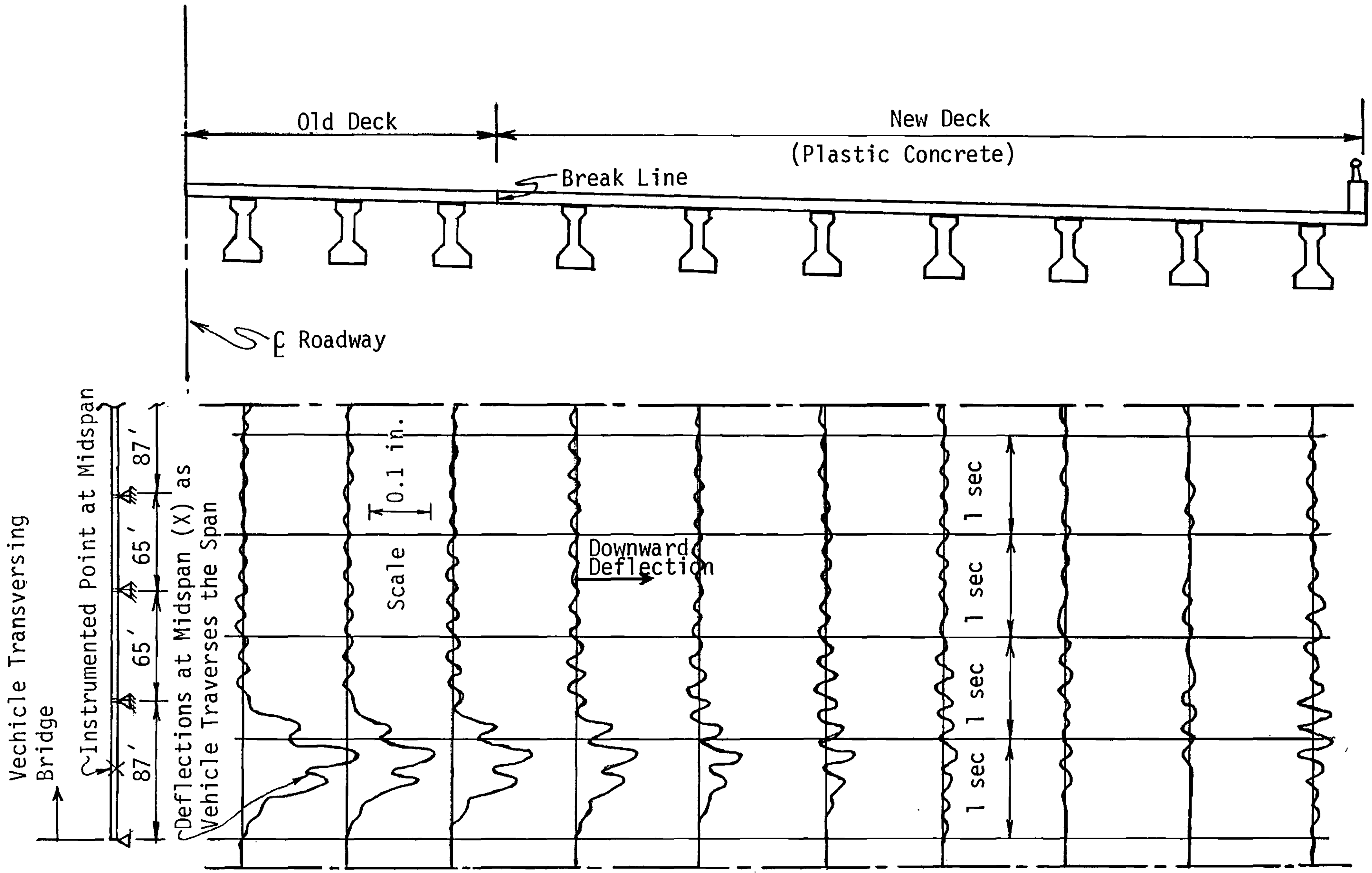


Figure 26.a. I-10 & Dell Dale Ave., Houston, Midspan Deflections of Beams due to One Crossing.

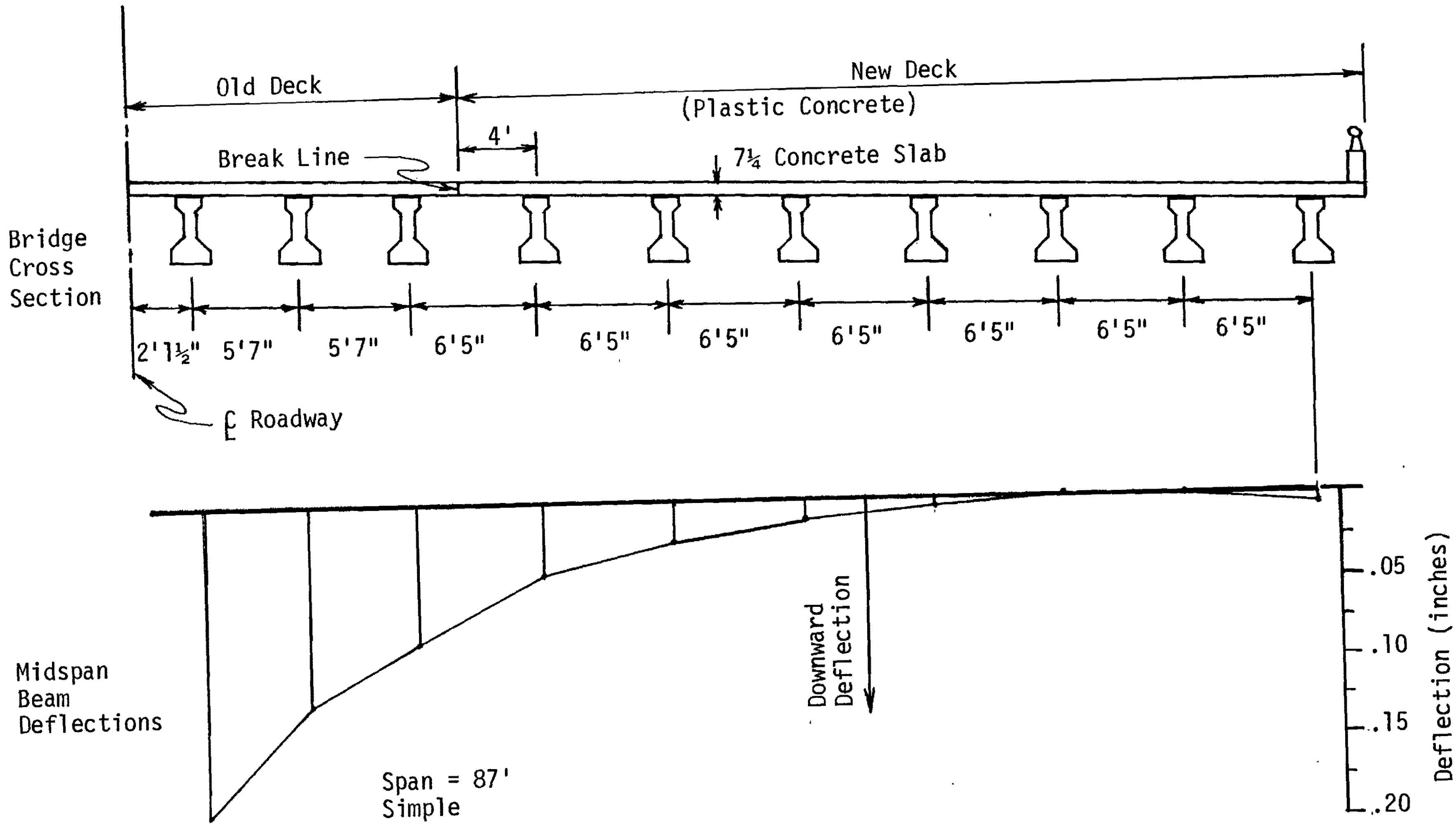


Figure 26.b. I-10 & Dell Dale Ave., Houston, Maximum Midspan Beam Deflection for One Crossing.

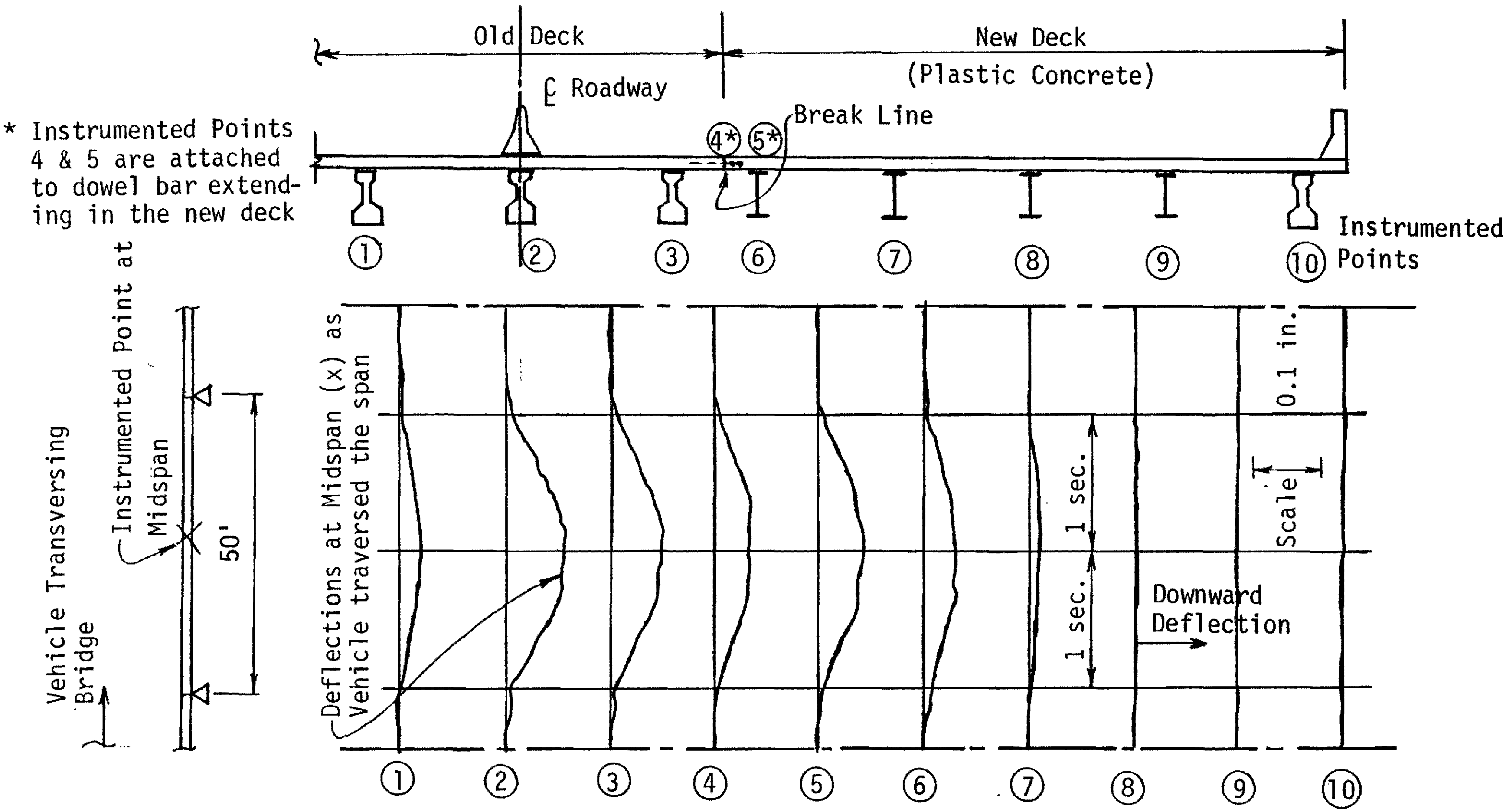


Figure 27.a. US 75 & White Rock Creek, Dallas, Southbound (50' span) Midspan Deflections of Beams and Rebars due to One Crossing.

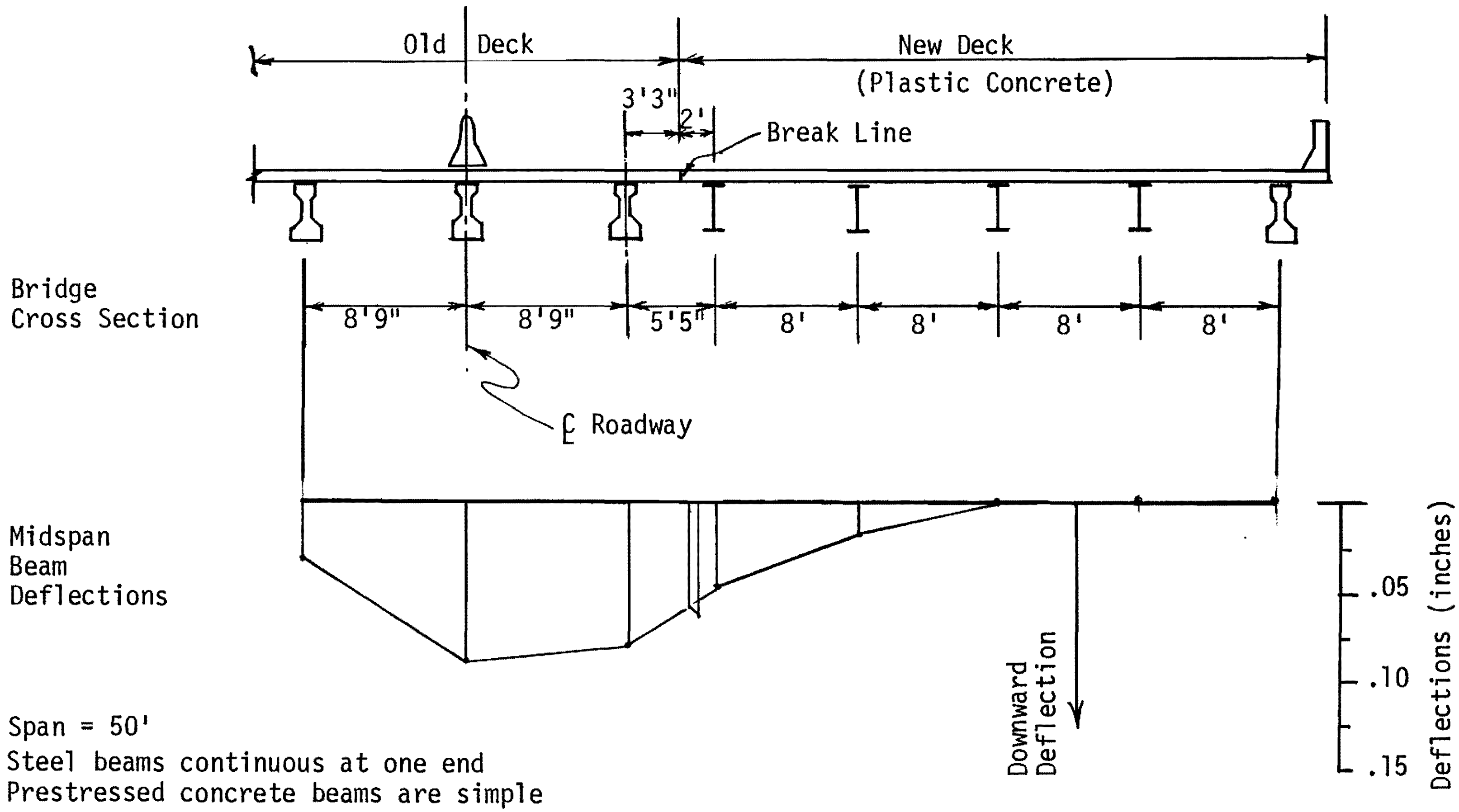


Figure 27.b. US 75 & White Rock Creek, Dallas, Southbound (50' span)
Maximum Midspan Beam Deflections for one Crossing.

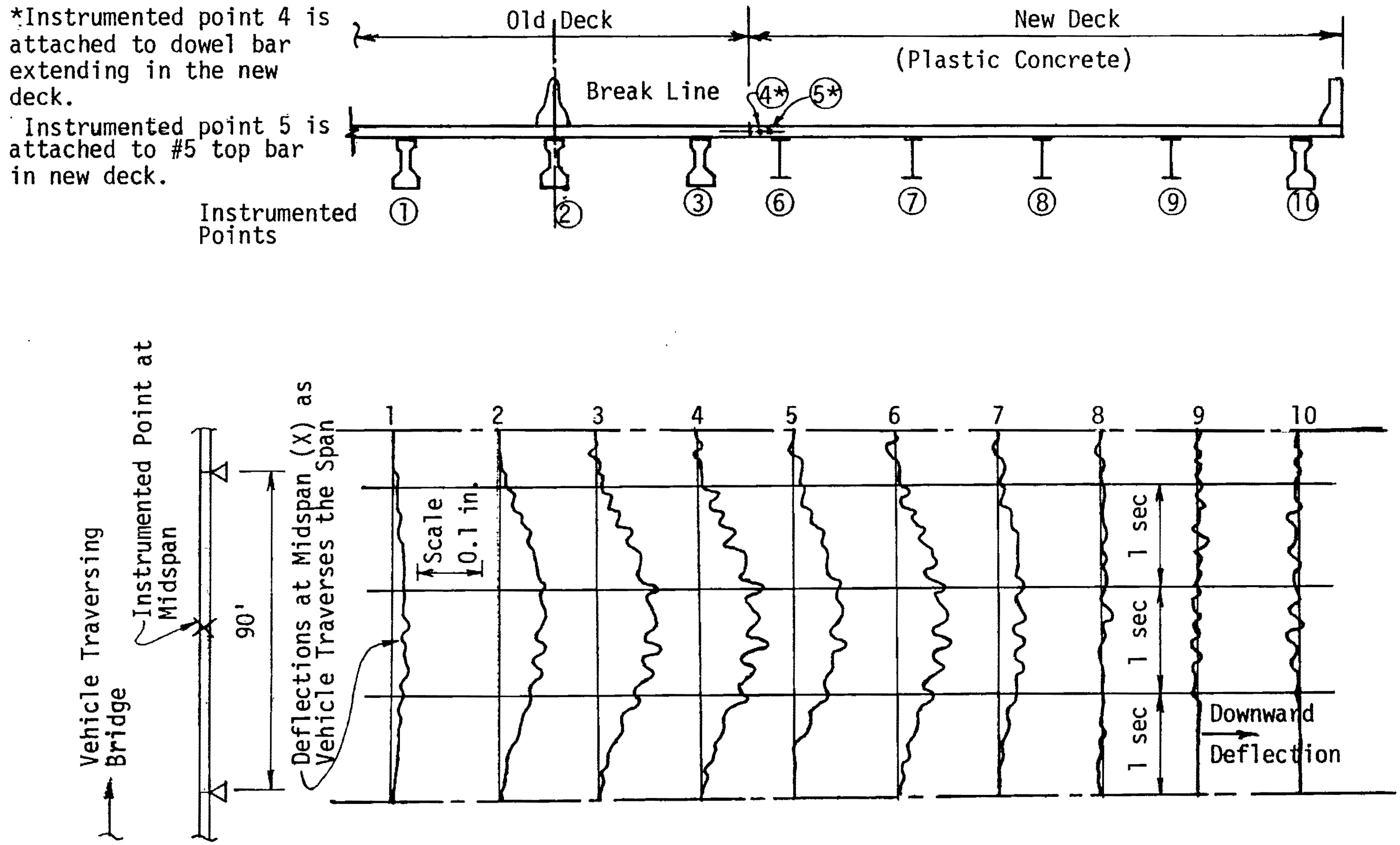
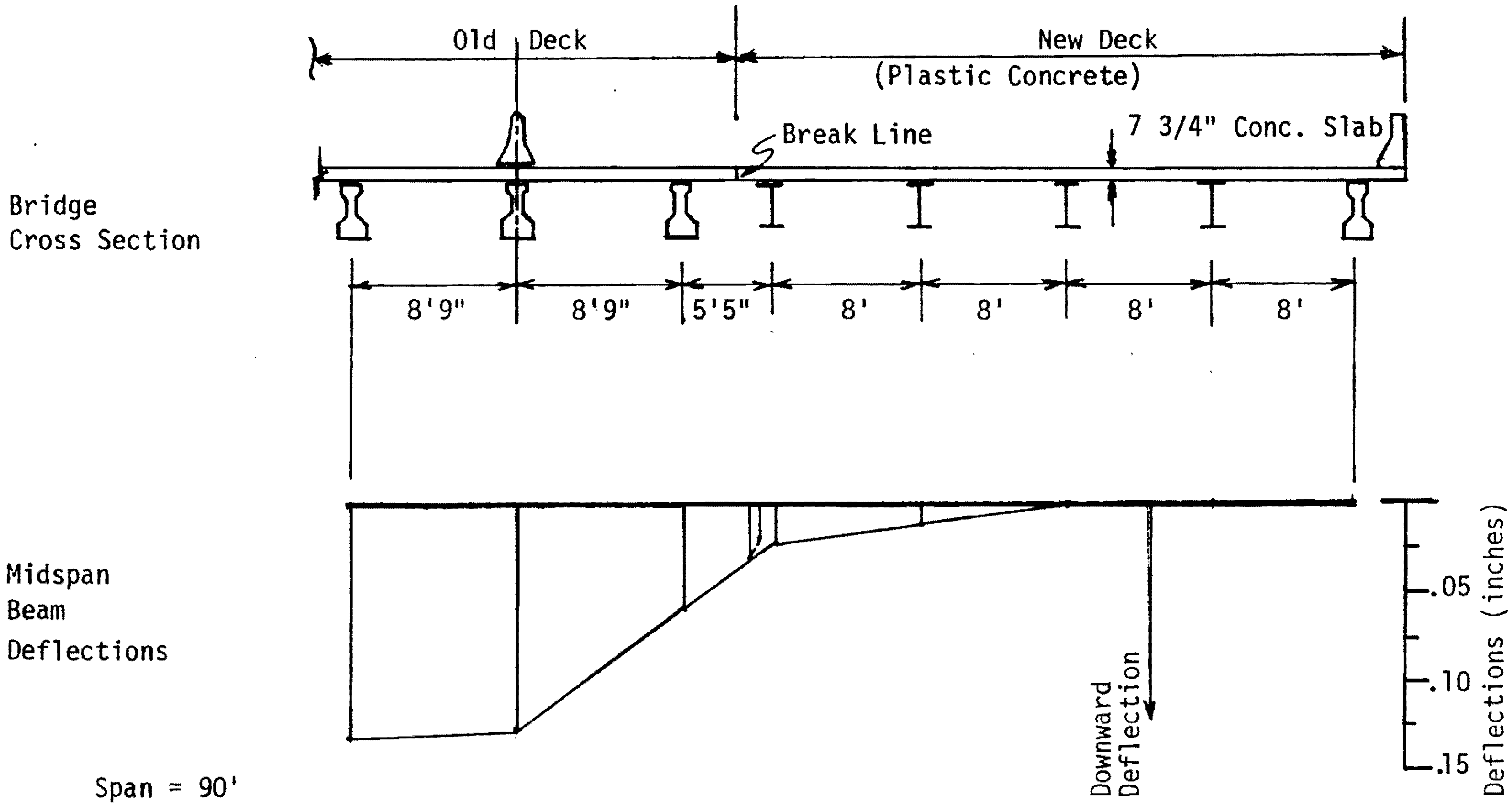


Figure 28.a. US 75 & White Rock Creek, Dallas, Northbound (90' Span), Midspan Deflections of Beams and Rebars due to One Crossing.



Span = 90'
Steel beams continuous at both ends
Prestr. conc. beam, simple

Figure 28.b. US 75 & White Rock Creek, Dallas, Northbound (90' Span), Maximum Midspan Beam Deflections for One Crossing.

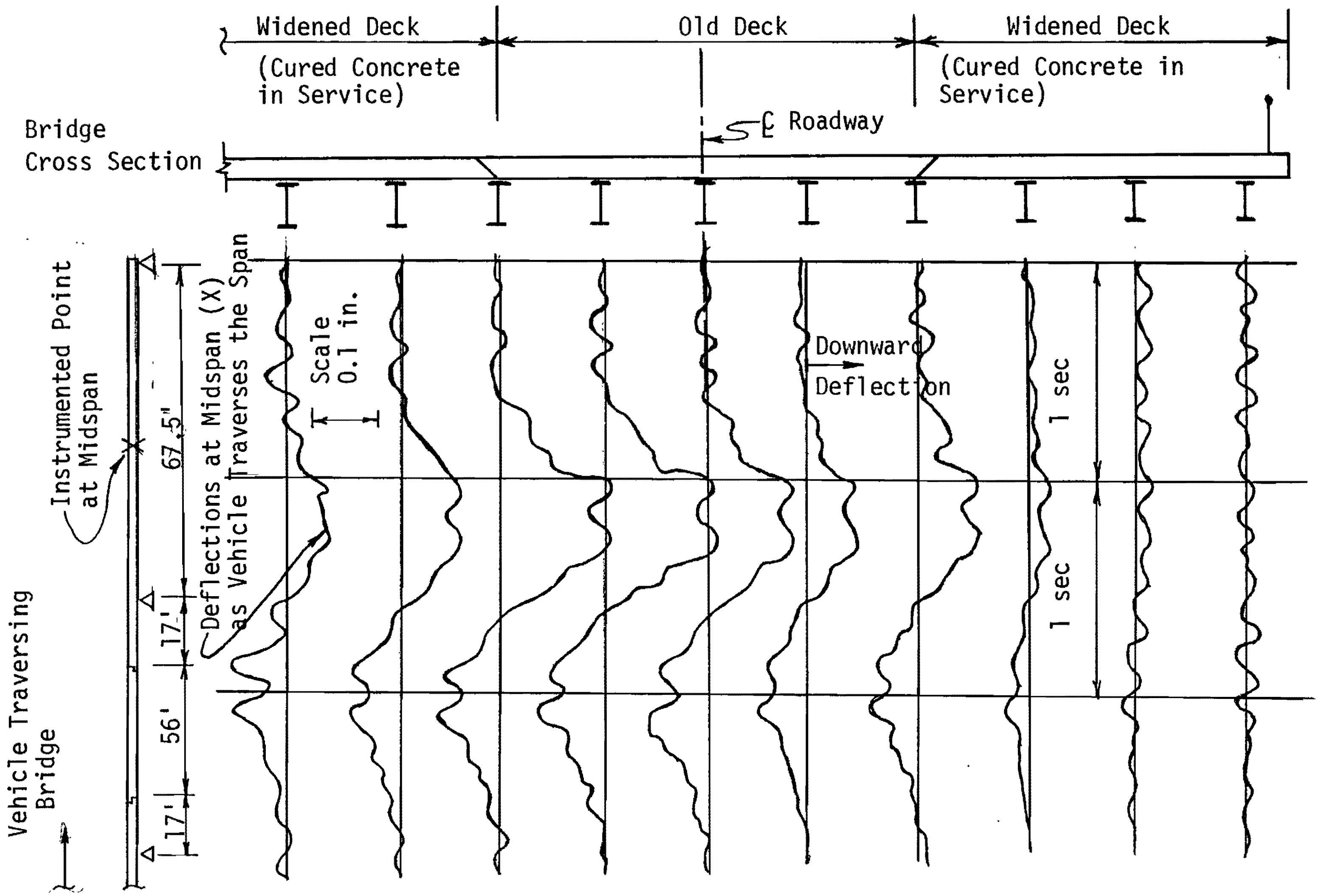


Figure 29.a. US 84 & Leon River, Gatesville, Midspan Deflections of Beams due to One Crossing.

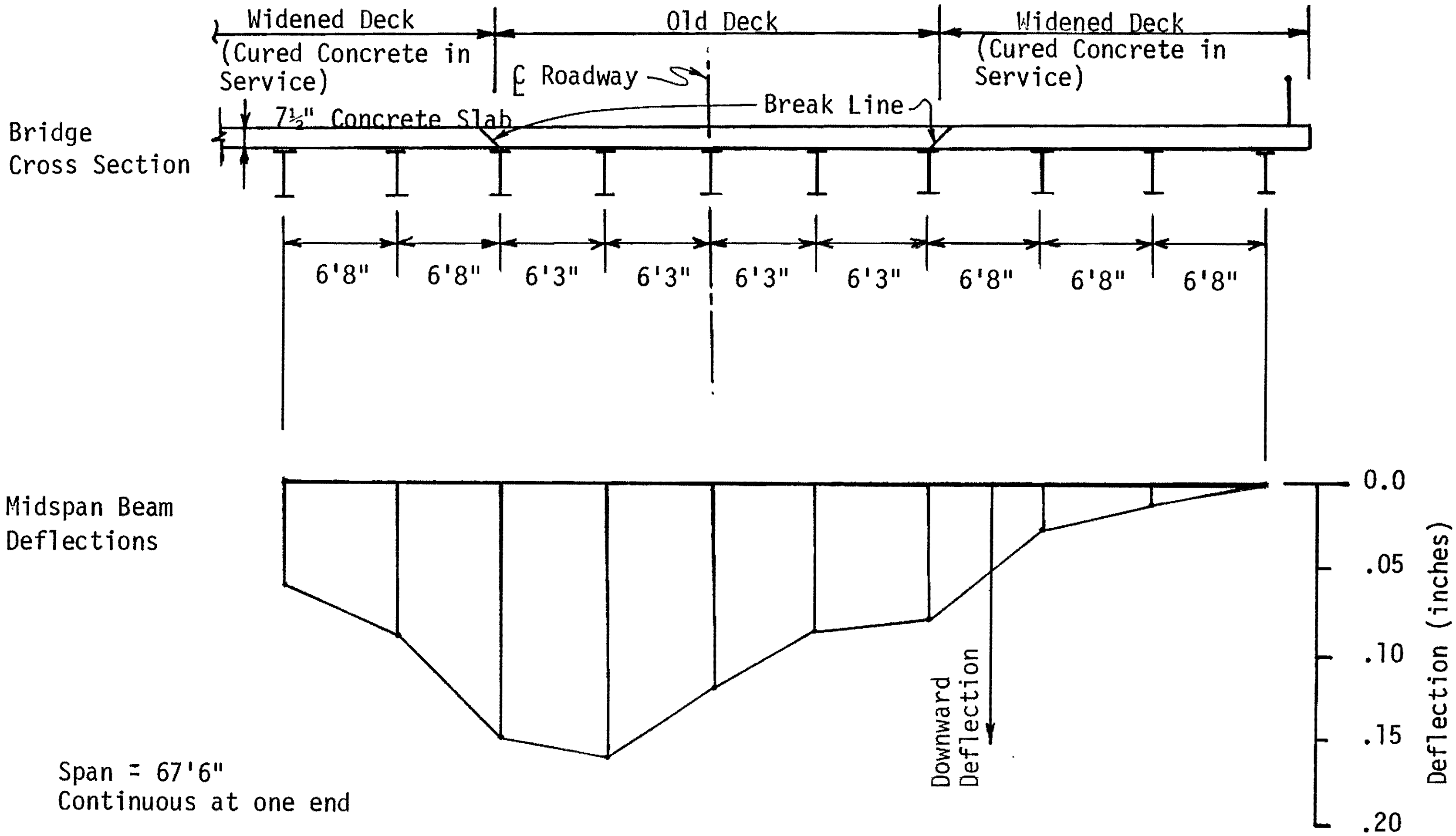
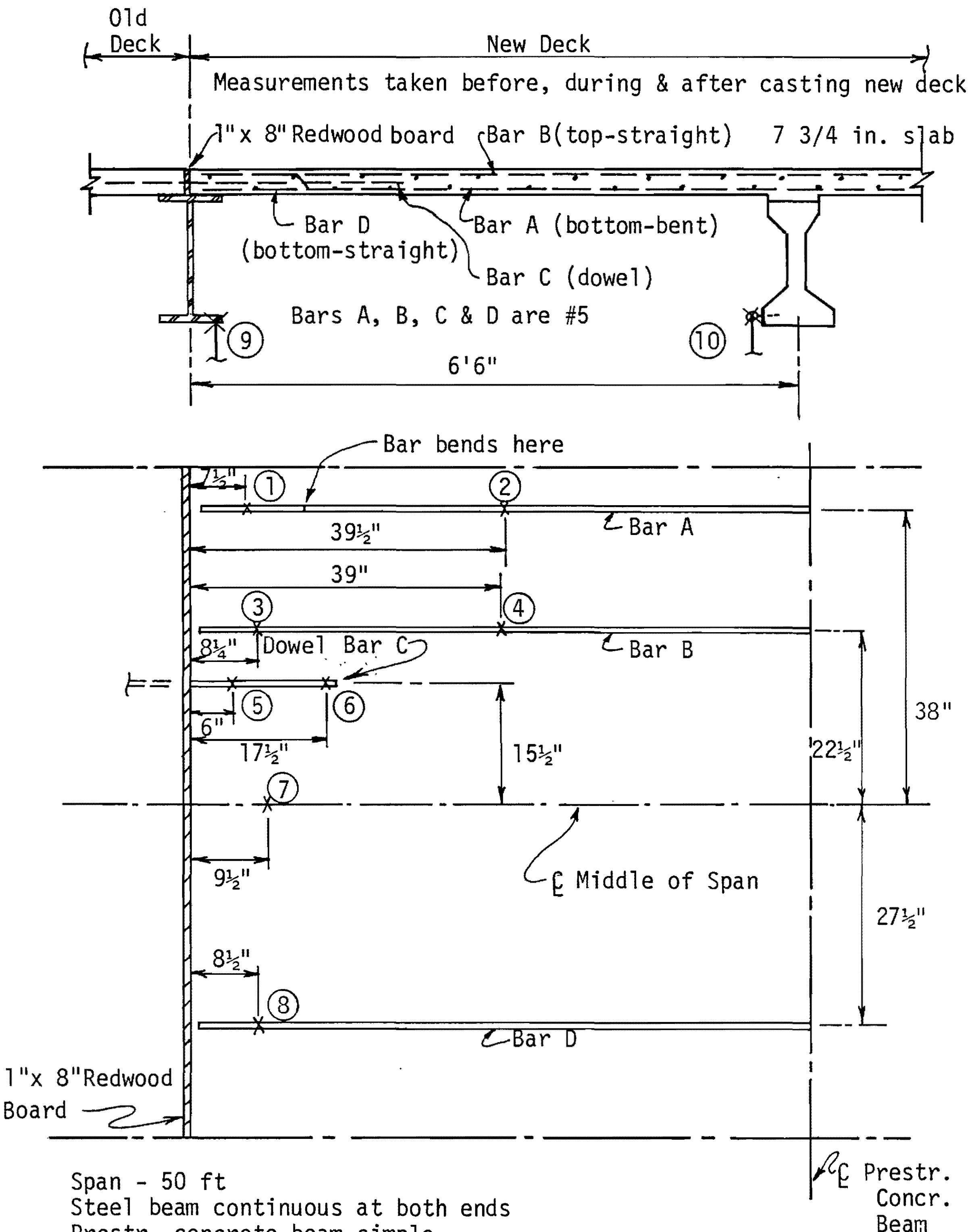


Figure 29.b. US 84 & Leon River, Gatesville, Maximum Midspan Deflections for one Crossing.



Span - 50 ft
Steel beam continuous at both ends
Prestr. concrete beam simple

Instrumented Points are Numbered and Designated by X.

Figure 30. SH 183 & Elm Fork Trinity River, Irving (Dallas), Instrumented Reinforcing Bars & Beams.

Laboratory Beams

As previously mentioned, all beams were reinforced except for the first one which had no reinforcement. All beams had a cyclic end deflection of 0.25 in. except beam #5 which had an end deflection of 0.15 in. Beam #4 had a superimposed vibration of 6 Hz frequency and $\pm$0.020 in. amplitude at the loaded end in addition to the cyclic deflection.

Flexural cracks were generated as a result of deflecting the formwork supporting the fresh concrete. Figures 31 through 35 show the deflected configuration of the test beams at the peak of a cycle of deflection. Two cases of deflection are presented for each beam. The first case is for a cycle at an early age after casting, when concrete is still plastic. The second case of deflection is for a cycle at a later age, about 5 hours from casting, when concrete had at least gained its initial set. Figures 36 and 37 show the cracking patterns on the top surface of each of the five test beams after the test was terminated. The figures provide also the maximum and average width of cracks measured in mils. Table 10 lists the curvatures computed for the deflection cases shown in Figures 31 through 35. Table 10 also lists the approximate time after casting when a crack was first detected. Table 11 gives the maximum depth of vertical penetration of the cracks as seen on the sides of the beams.

The curvatures in Table 10 were computed for that portion of the beam in the vicinity of the center support, where most of the cracks occurred. Computation of curvatures was made by assuming an arc of a circle passing through three deflected points, and as explained earlier in the report.

Test beam #5, with 0.15 in. end deflection as compared to 0.25 in. for the other test beams, had the smallest surface curvature as shown in Table 10. Only one crack formed in the vicinity of the center support as compared to

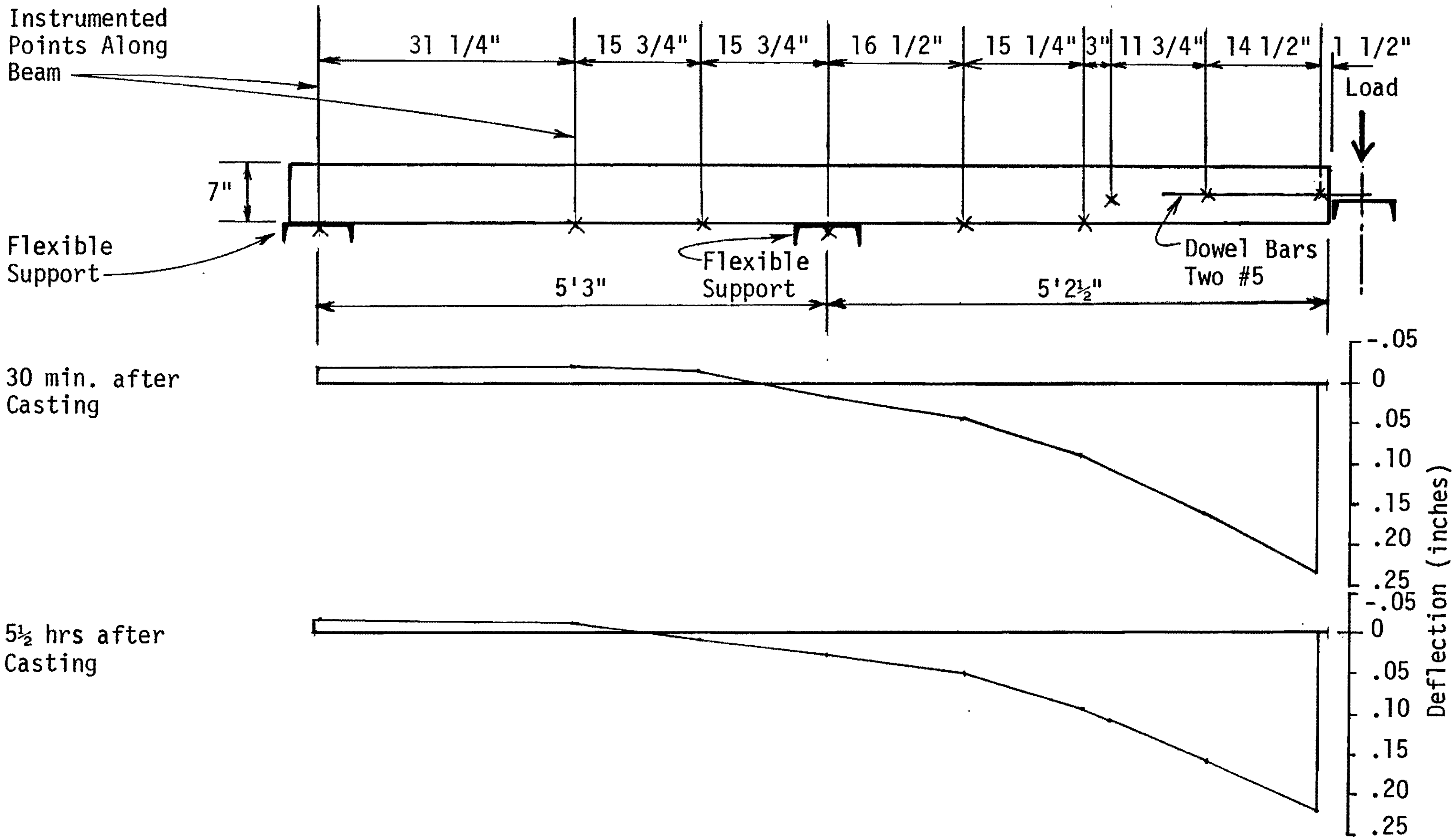


Figure 31. Beam #1, Deflections at Two Different Ages.

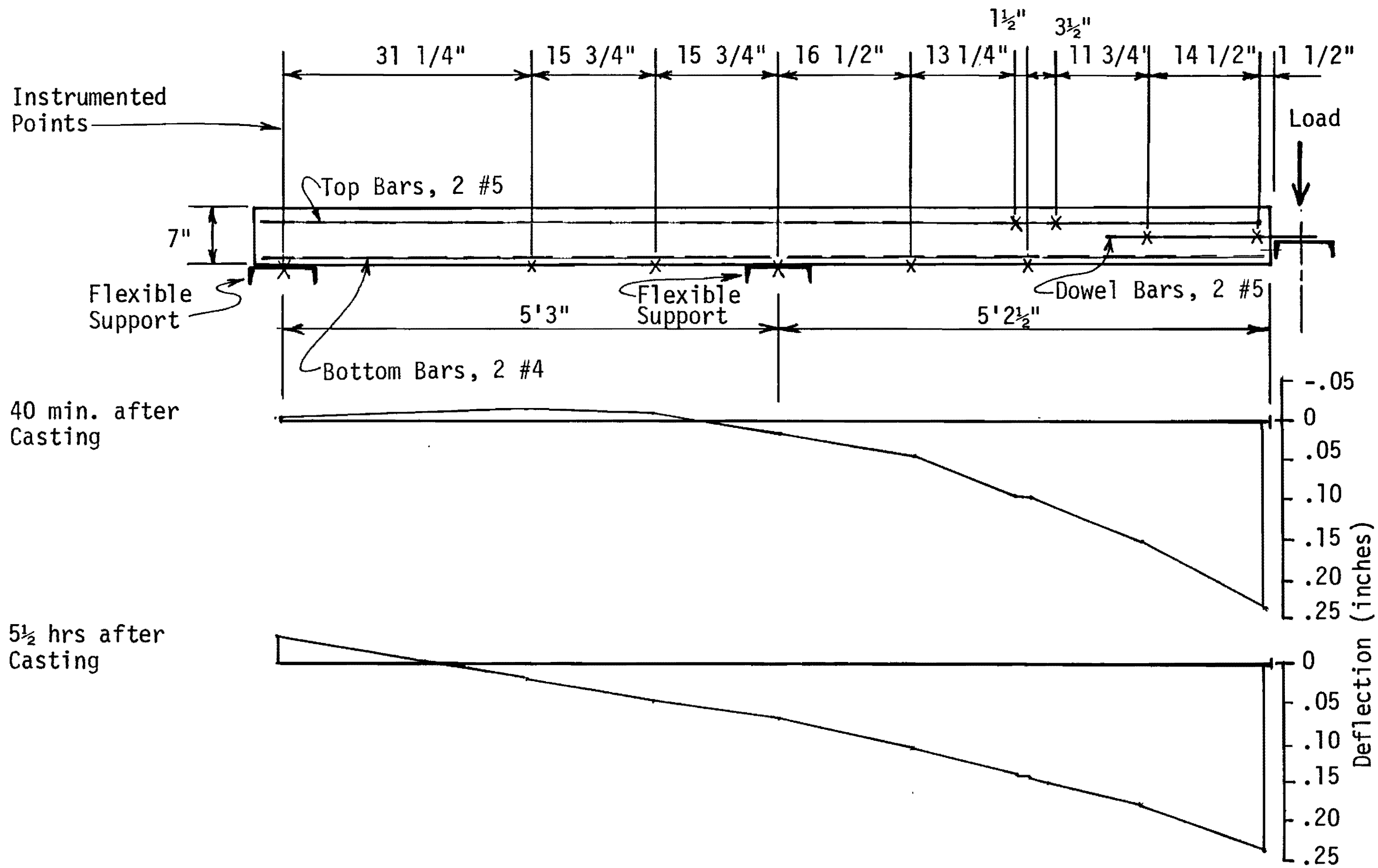


Figure 32. Beam #2, Deflections at Two Different Ages.

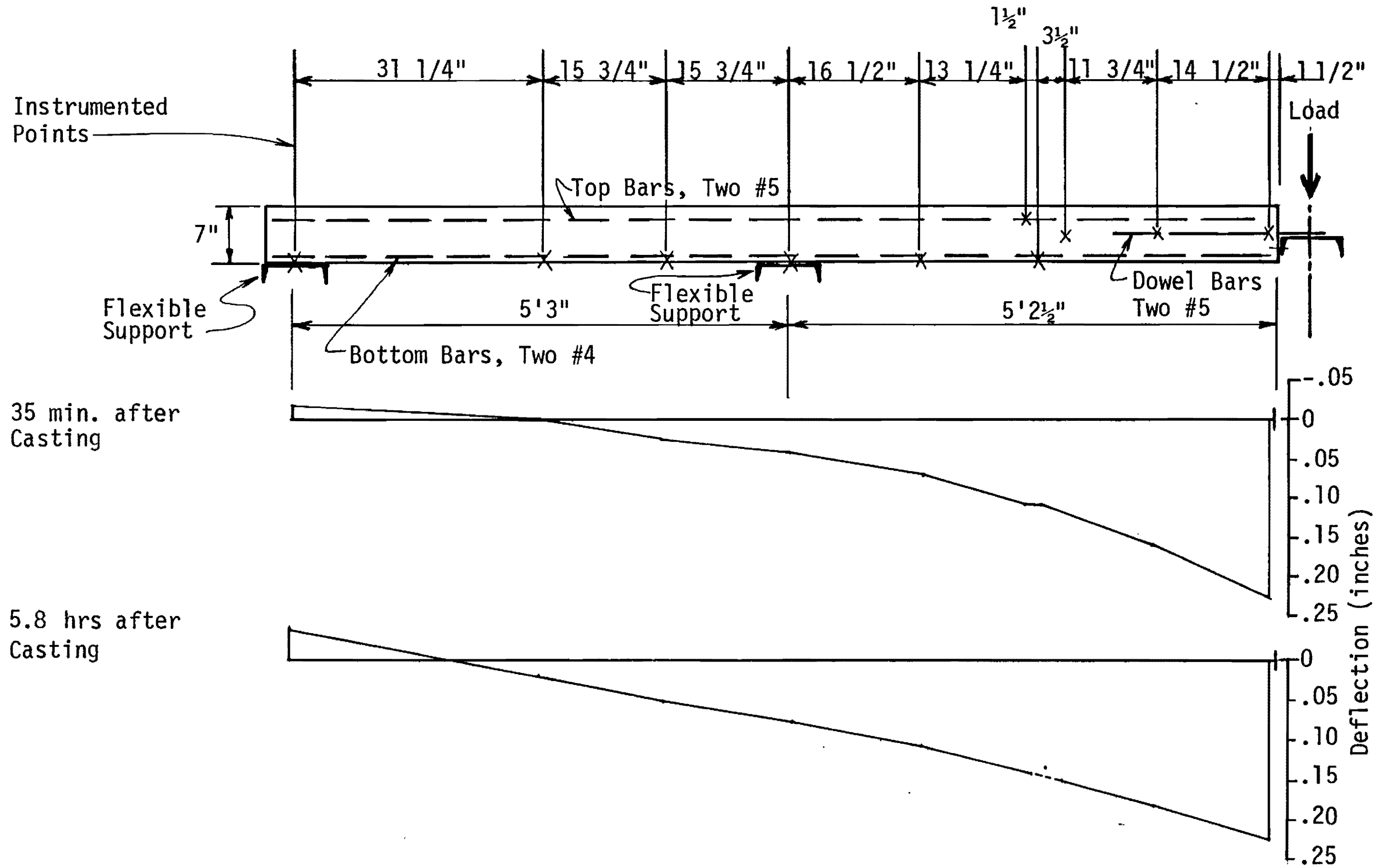


Figure 33. Beam #3, Deflections at Two Different Ages.

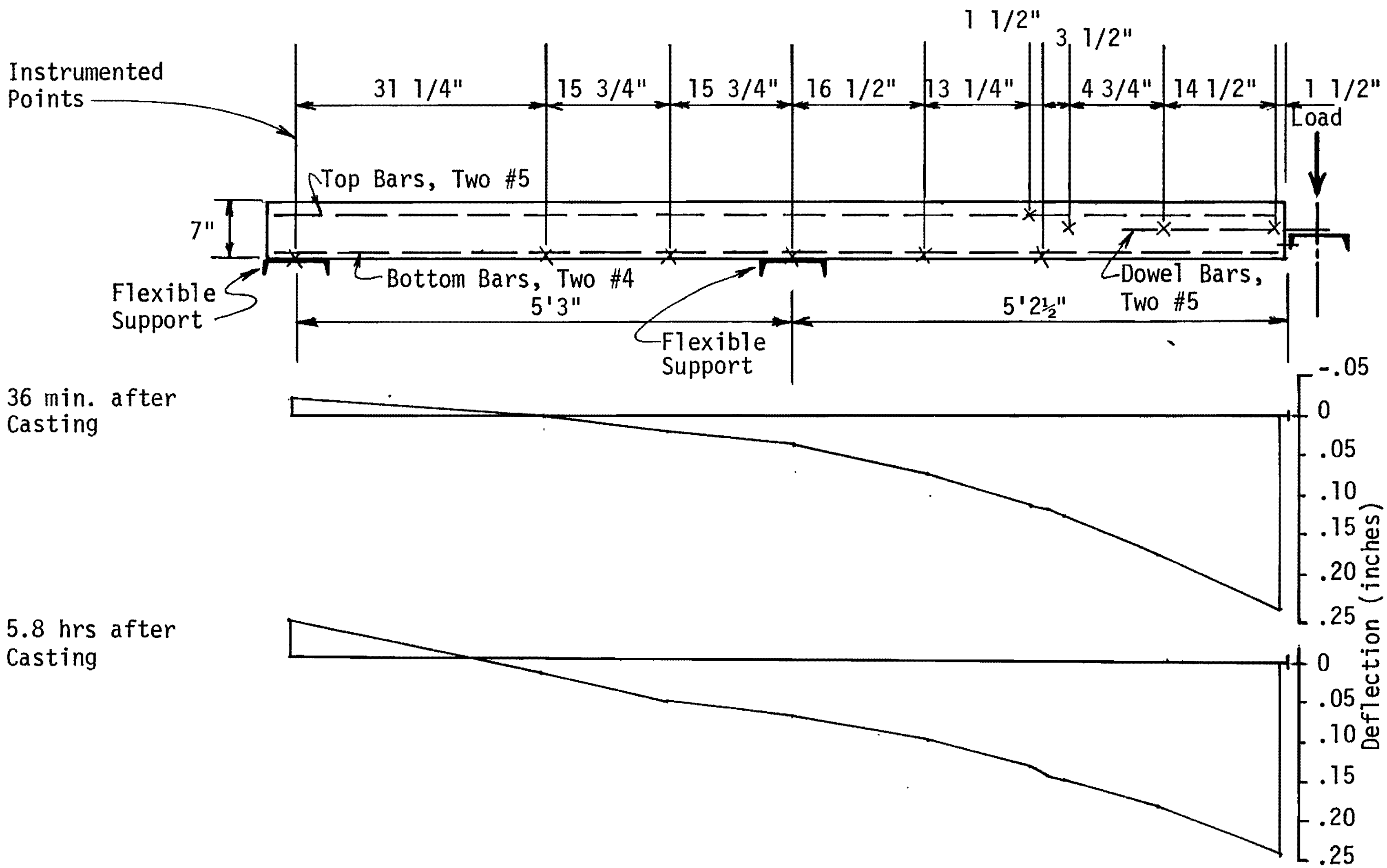


Figure 34. Beam #4, Deflections at Two Different Ages.

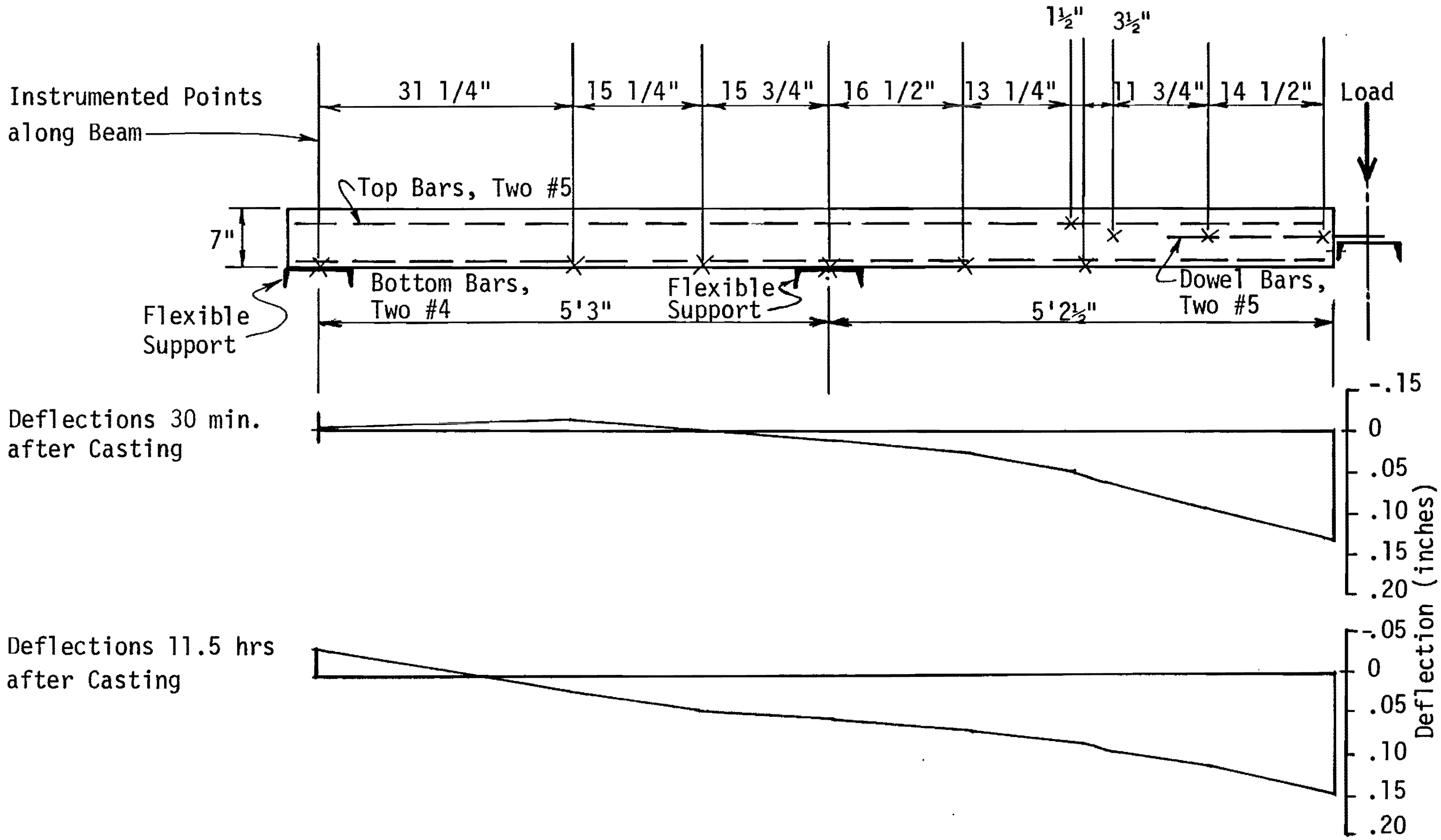
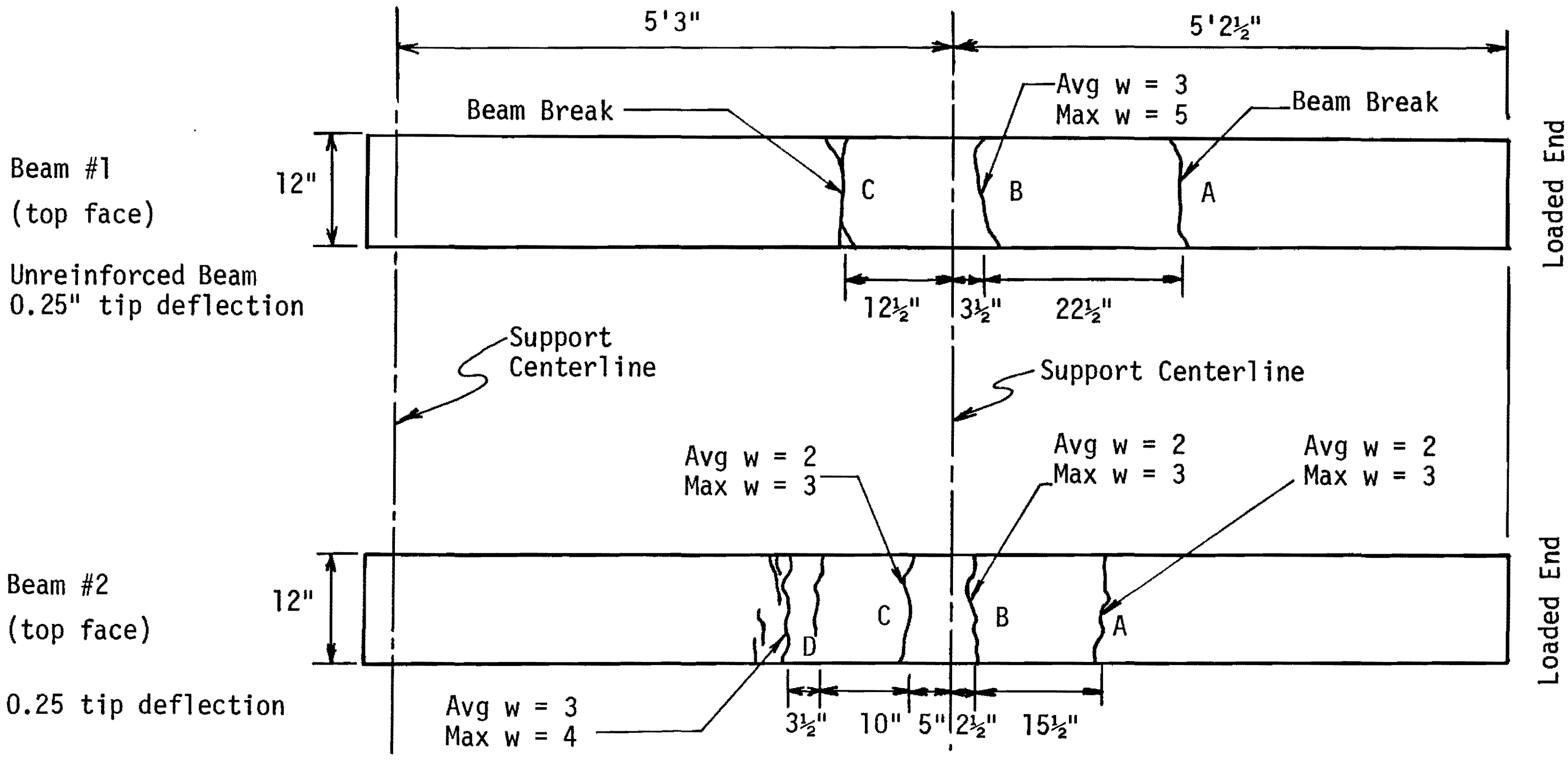


Figure 35. Beam #5, Deflections at Two Different Ages.



w = crack width in $\frac{1}{1000}$ in.

Figure 36. Beam Crack Pattern at End of Test. Beams #1 & #2

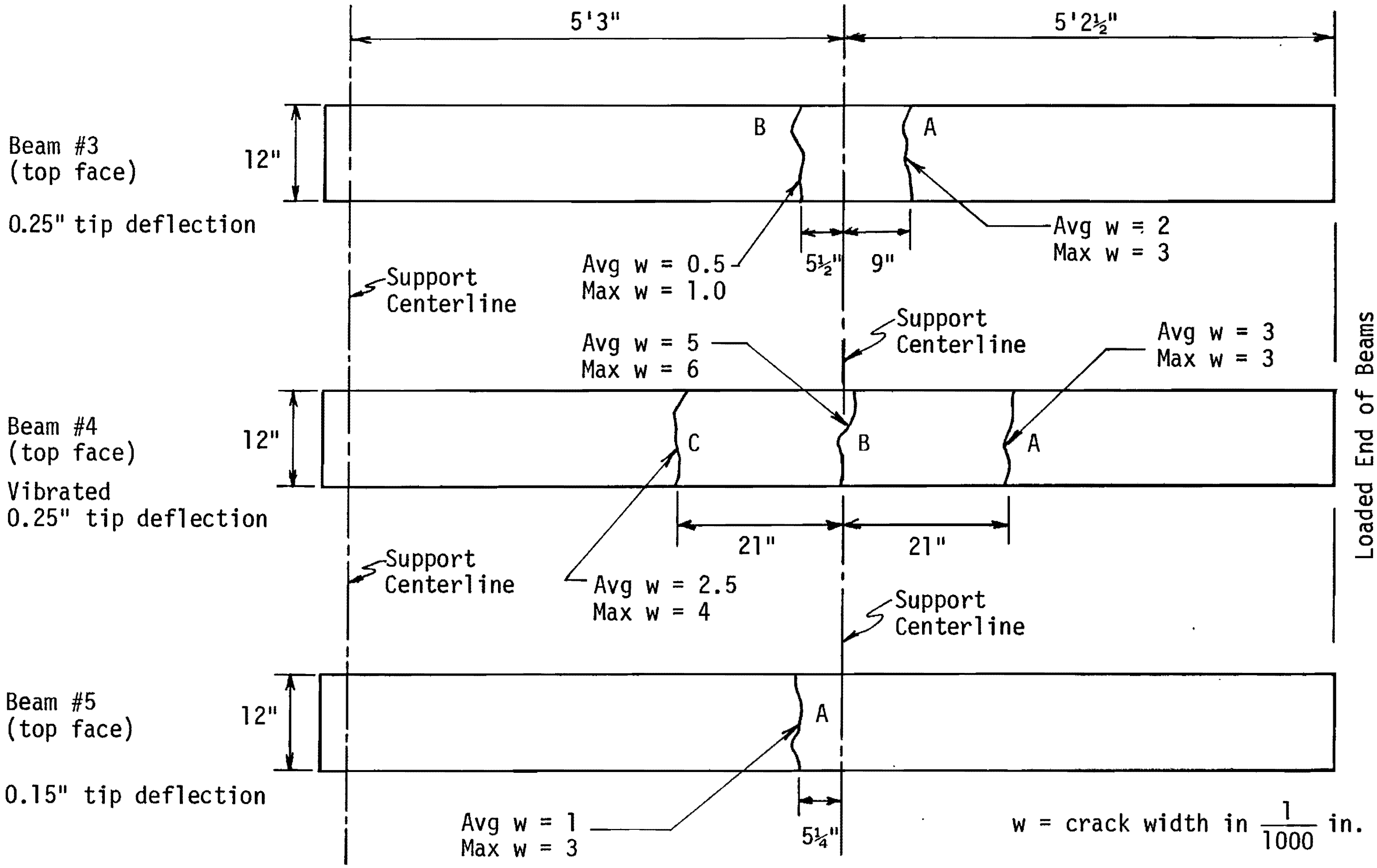


Figure 37. Beam Crack Pattern at End of Test
Beams #3, #4, & #5

TABLE 10. BENDING CURVATURE OF TEST BEAMS AT TWO DIFFERENT AGES AND THE APPROXIMATE TIME TO FORMATION OF FIRST CRACK ON SURFACE.

BEAM NO.	AGE AFTER CASTING (hr-min.)		RADIUS OF CURVATURE R (in.)	CURVATURE 1/R $\times 10^{-5}$ (in.$^{-1}$)	APPROXIMATE TIME TO FIRST CRACKING (hrs)
1*	00	30	15,280	6.54	4.5
2	00	40	18,434	5.42	4.4
	5	30	23,074	4.33	
3	00	35	17,957	5.57	4.0
	5	48	39,016	2.56	
4	00	36	16,354	6.11	3.75
	5	47	22,148	4.52	
5	00	30	27,936	3.58	10.50
	11	34	22,690	4.41	

*Curvature of unreinforced beam #1 is not given because all rotation occurred at cracks which formed hinges in the beam.

TABLE 11. MAXIMUM CRACK PENETRATION AS VIEWED FROM SIDE OF BEAM.

	CRACK DEPTH (in.)			
BEAM NO.	A	B	C	D
1	7[a]	b	7[a]	--
2	6	5.5	3.5	5.5
3	4.5	1.5	--	--
4	5	7	4.5	--
5	2.25	--	--	--

[a]Full separation of specimen occurred.

[b]Depth not determined.

more than one crack forming in the other test beams. In beams #2, #3 and #4, the cracks penetrated beyond the top steel, as indicated in Table 11, and to the level of the top steel in beam #5. The crack width averaged 1 mil in beam #5 as compared to a minimum average of 2 mils in the other test beams. This shows that the extent and severity of cracking is related to the surface curvature of the plastic concrete.

Concrete surface curvature decreased with age for the reinforced concrete beams #2, #3 and #4 as shown in Table 10. This can be explained by referring to the elastic beam model. For the elastic beam, curvature is inversely proportional to the modulus of elasticity of the material, E, and the moment of inertia of the cross section, I. If this is valid for the plastic concrete as well, which is most likely, then it can be said that the surface curvature of the concrete beam has decreased at the later age because the concrete has become stiffer. As the concrete gets older, E increases and hence the curvature decreases. This agrees with Hilsdorf and Lott's (4) findings that for the same curvature, cracks in 4-hour age slabs were deeper and wider than those in 2-hour age. This indicates that concrete when plastic can tolerate more curvature without cracking than the older concrete.

Deflection data of the reinforced beam tests showed that when concrete was first placed in the form, the center flexible support deflected about 0.015 in. when the end load was applied. As the concrete got older, and hence stiffer, the deflection of the center support increased. When the concrete was about 10 hours old, the center support reached its maximum deflection, which was about six times the initial deflection. In beam #1, unreinforced, the maximum center support deflection was about 2.33 times the initial 0.015 in. deflection.

Summarizing, it can be said that when the concrete is plastic its particles have freedom to move with respect to one another, and thus it can tolerate a large curvature. Later, as the material gets older and the concrete begins transforming from the plastic state to solid, the hydration product (gel particles) begin to bind particle to particle, and the freedom of movement of the plastic state becomes restricted. At this stage the concrete tolerates less curvature, begins to develop tensile and compressive capabilities, and there will be flexural action. By this reasoning it appears that a deck might go uncracked for a few hours after casting as vehicles move across the bridge, but it might crack later, possibly at or about initial set, under the same traffic conditions as before.

The effect of the superimposed vibrations in test beam #4 proved to be insignificant with respect to bonding of the reinforcing steel. Deflection measurements showed that the concrete form, the concrete beam, and the reinforcing steel were all vibrating at the same applied frequency (6 Hz) and amplitude ($\pm$.020 in.). Hence, relative movement between the reinforcing steel and the plastic concrete did not occur. If such a movement had taken place, it might have been detrimental to the bond between the reinforcing steel and the concrete.

Beam #4 showed significant differences in flexural cracking to that of beams #2 and #3. The latter two beams had the same applied end deflection as beam #4 but were not subjected to the superimposed vibration. Beam #4 had a larger surface curvature at the early age, about 13% greater than the curvature of beam #2 and about 10% greater than the curvature of beam #3, see Table 10. Consequently, this increase in curvature had an effect on flexural cracking condition of the beam. Beam #4 had wider and deeper cracks when compared to beams #2 and #3, as shown in Table 11 and Figures 36 and 37. The larger surface curvature of beam #4 could possibly be attributed to the larger deflections, about 8% higher than beams #2 and #3, occurring in this beam test due to the superimposed vibrations.

At about 2½ feet from the loaded end, three points were instrumented and their deflection measurements were taken as mentioned previously. The hard concrete, the concrete form and the top steel were instrumented at points relatively very close to each other as shown in Figures 31 through 35. Within the accuracy of the deflection measuring system, the deflections of these three points were so nearly the same that differences could not be read. Hence it can be said that the form did not separate from the concrete beam and both deflected the same. This same thing was found in one bridge, Texas Hwy. 183 & Elm Fork Trinity River bridge. The form deflection was measured and it was found to be identical to that of the concrete. This indicates good agreement, in this respect, between the test beam and the bridge.

The three instrumented points also showed the same deflection for the #5 top bar and the hard concrete. Thus no relative movement did occur between the reinforcing bar and the concrete.

Laboratory Beam Cores

The ten core specimens considered for study are listed in Table 12, where their conditions after the dye tests are given.

The first set of five cores indicated that cracks penetrated below the level of the top steel when the beam tip deflection was 0.25 in. The end deflection of beam #5 was 0.15 in., and the transverse flexural crack extended only to the level of the top steel. Thus, it can be said that crack depth is dependent on the magnitude of the differential deflection and consequently the magnitude of the surface curvature of the concrete.

Cores 2-4 and 3-3 each contained a transverse #4 bar. In both cases, the flexural transverse crack allowed the dye to penetrate to the steel bar level completely soaking the bar cavity. Transverse bars in the beams correspond to longitudinal reinforcement in a bridge deck. Although this reinforcement is not the main reinforcement of the bridge deck, yet, if water can reach the longitudinal deck steel, deterioration of the concrete deck would be inevitable under normal conditions.

The study of the second set of five cores indicated that bonding of the dowel bars, connecting between the old and new decks of a bridge, or bars tied to them, may be somewhat deficient or imperfect. Cores 2-1, 3-1 and 4-1 were obtained from that portion of the beam which had a dowel bar as well as top and bottom steel. These cores were compared to core 5-2 which was obtained from the middle portion of the beam where relative movement between the steel and concrete is highly improbable. In all three cores, the dye penetrated through the ends of the dowel bar and the #5 top bar, completely soaking the whole embedment surface of the bar. Also, bar imprints were not sharp and well defined for at least one bar in each of the three cores. The undisturbed core, 5-2, did not display any bond problems, and dye penetration at the ends of the #5 top bar was 1/4 in. at most. These results indicate that some slight movement does in fact occur between the dowel bar and the fresh plastic concrete, and unfavorable bond might result from such action.

In cores 2-1 and 4-1, the condition of bonding of the dowel bar and #5 top bar were compared to that of the bottom #4 bar. The latter bar had very sharp and well defined imprints, and dye penetration was minimal. Figure 38 shows photographs of cores 2-1 and 4-1. The darker bar cavity of the dowel bars indicate that dye has penetrated through end of bar.

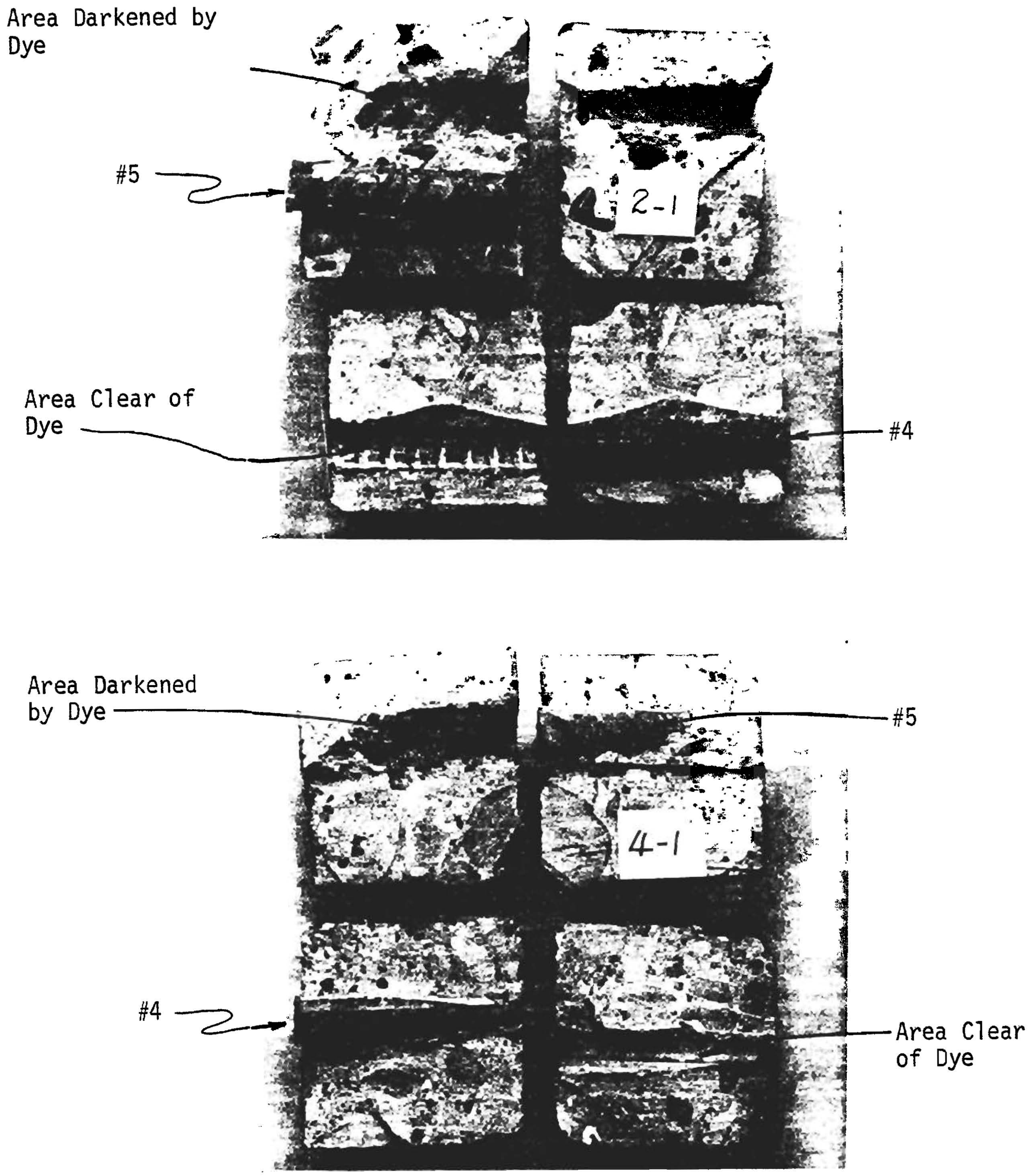


Figure 38. Cores Split after Dye Test.

TABLE 12. CONDITION OF DYE-TESTED BEAM CORES

Core	Condition
2-3	A transverse flexural vertical crack existed. Core had no steel. The dye penetrated the mortar to a depth of 6 1/8 in. The penetrated dye showed a crack profile that ran through the mortar and around aggregate interfaces only.
2-4	Core had a #4 transverse top bar with a clear cover of 2 5/8 in. The dye showed that the transverse flexural vertical crack penetrated to the level of the #4 top bar and had extended below it. Depth of penetration of the crack was about 4 3/4 in. The #4 bar cavity was all soaked in dye; however, the bar imprints were well defined and the bar pulled out mortar particles indicating good bonding of the steel and the concrete.
3-3	Core had a #4 transverse top bar with a clear cover of 2 5/8 in. The transverse flexural vertical crack penetrated to the level of the #4 top bar and extended below it. The dye color showed that the full depth of penetration of the crack through the mortar was 5 1/4 in. The crack propagated through mortar and aggregate interfaces only. The #4 top bar cavity was all soaked with dye. Imprints of this bar were well defined, indicating good bond.
4-5	Core had no steel. A transverse flexural vertical crack existed at top of this core. When core was sliced, normal to this crack, the dye showed that the crack had penetrated to the full depth of the core, i.e., 7 in. It should be pointed out that beam #4 had been deflected extensively (up to 0.8 in.) after it was tested, to study the width of cracking. This explains why the vertical crack penetrated through the core's full depth.
5-2	Core piece had a #5 longitudinal top bar with a 2 in. clear concrete cover. A transverse flexural vertical crack existed at top of the core. The dye showed that the crack penetrated through the mortar and aggregate interfaces extending to the level of the #5 top bar. The depth of penetration of the crack was 2½ in. The dye also penetrated the mortar at the ends of the bar only for about 1/8 in. from one end and 1/4 in. from the other. The dye penetrated the mortar about 1/16 in. The bar cavity had an area of about 1/16 in. wide and 1 in. long around the circumference colored with dye. This occurred at the location where the transverse vertical crack intersected with #5 longitudinal bar. The rest of the bar cavity was free of dye color, and the bar imprints were sharp and well defined, indicating good bond. This core was obtained from the middle portion of the beam, at a flexural crack, and where reinforcing bars were supposed to have least disturbance.
2-1	This core was sawed, normal to its axis, into two pieces of lengths 4½ in. and 2½ in., respectively. The top 4½ in. length had a #5 dowel bar and a #5 top bar. The bottom 2½ in. length had a #4 bottom bar. Both pieces were dye tested. The dye penetrated the mortar (from outside surface of core) only for about 1/16 in. It penetrated through

TABLE 12 - Continued

Core	Condition
	the ends of the #5 top bar and soaked all the bar cavity. Bar imprints were well defined. Similarly, the dye penetrated the mortar through the ends of the #5 dowel bar, soaking all the bar cavity. However, the bar imprints were not very sharp. Some bar imprints were wider than the actual deformations on the bar, indicating a possibility of some relative movement between the dowel bar and the concrete. The bottom #4 bar had very sharp and well defined imprints. The dye penetrated the mortar from the ends of the #4 bar, 1/16 in. from one end and 1/8 in. from the other.
2-2	The core specimen had a #5 dowel bar and a #5 top bar. Both bars were tied together with a tie wire at the location of the core. The dye penetrated the mortar through one end of the dowel bar about 3/8 in. The rest of the bar cavity was free of dye and the bar imprints were sharp and well defined. The dye penetrated the mortar from one end of the #5 top bar about 1/2 in. and about 1 in. from the other end. It penetrated the mortar through outside surface of core about 1/16 to 1/8 in. Some of the bar imprints were not very sharp nor well defined, indicating an uncertain condition of bond. The poor imprint may be attributed to some slight movement of the top #5 bar which was tied to the dowel bar.
3-1	The core specimen had a #5 dowel bar and a #5 top bar. The dye penetrated the mortar at the ends of the dowel bar soaking all the bar cavity. Some of the bar imprints were not very sharp nor distinct, indicating imperfect bond. The dye penetrated the mortar at the ends of the #5 top bar about 1/2 in. from one end and 1 in. from the other. Bar imprints were sharp and distinct.
4-1	This core was sawed normal to its axis into two pieces of lengths 4½ and 2½ in., respectively. The top 4½ in. core length had a #5 dowel bar and a #5 top bar tied to the dowel bar, whereas the bottom 2½ in. core length had a #4 bottom bar. Both pieces were dye tested. The dye penetrated the mortar at the ends of the #5 top bar and soaked all bar cavity. Bar imprints were well defined. Dye penetrated mortar from outside surface of core about 1/16 in. The dowel bar had dye color penetrating through one of its ends about 3/8 in. and about 1/2 in. from the other end. The bar imprints were well defined. However, the dowel bar did not pull out any mortar particles, as was normally the case, nor did mill scale break when the specimen was split open which indicates that a condition of poor bond may have existed. The bottom #4 bar appeared to have excellent bond as compared to the above two bars. Bar imprints were very sharp, distinct and well defined. The dye penetrated the mortar at one end of the bar about 1/4 in. and about 3/8 in. from the other end.
4-3	This core specimen had two bars embedded in it, a #5 dowel bar and a #5 top bar. Bars were not tied together. The dye penetrated the mortar at the dowel ends about 1/4 in. from one end and 1/2 in. from the other; while penetrated the mortar to the inside of the core 1/16 to 1/8 in. Dowel imprints were sharp and well defined. Similarly, the bar imprints of the #5 top bar were well defined. The dye penetrated the mortar at one end of this bar about 3/8 in. and about 1 in. at the other end.

Summary of Results

Thirty bridges that had been in service during placement of widening concrete and for a period thereafter were visually inspected. All of widening and replacement construction on the interstate routes as well as on rural routes appeared to be in good-to-excellent condition. Various degrees of cracking intensity were encountered, and the cracks were oriented from random to transverse to longitudinal. Relatively few longitudinal cracks were found, compared to random and transverse. The search for deterioration at the joint between old and new concrete, and in the region of negative bending adjacent to the joint, failed to locate any fault in the deck that could be charged to traffic during the very early age of concrete.

The 109 cores were taken from the new concrete on decks that had been widened for traffic. Two-thirds of these were from areas of the deck disturbed by passing traffic; one-third from areas not disturbed. About half of the cores showed that microcracking had developed, but these cracks can be expected from any normal concrete in service. Possibly more of the cores would have showed this kind of cracking if they had been sawed and polished. In a small sampling of the 109 cores it was found that some of the narrow cracks extended at least 3/4 in. from the top and bottom surfaces. The investigation did not attempt to carry the depth-of-cracking study to the point where the full extent and depth of such cracks could be established. It was determined that about the same percentage of cracked cores came from the undisturbed region as from the disturbed.

Most of the wider cracks, approximately 10 to 30 mils in width, found in cores were oriented perpendicular to traffic, such as would be expected of shrinkage cracking. Among the 23 wider cracks, 61% were perpendicular to traffic, flexural tension caused by differences in deflections of the bridge beams. They cannot be attributed to that cause, however, since they occurred, for the most part, in undisturbed deck areas. Neither these wider cracks nor the hairline, microscopic cracks provide evidence that they were caused by traffic-induced disturbance during or after placement.

Compression tests and pulse velocity tests were made on cores taken from nine bridges. Some of the cores were from regions of the deck that were essentially undisturbed by traffic, and others from midspan areas that were subjected to movements due to traffic. The strengths ranging from almost 2800 psi to 7000 psi showed that traffic disturbance caused no reduction in strength. Pulse velocities were essentially the same in disturbed as in undisturbed cores from the same bridge.

Five of the 109 cores, all from disturbed areas at or near the joint between old and new concrete, showed unmistakable evidence of relative movement between reinforcing steel and concrete. All five came from disturbed deck areas of US 75 bridge over White Rock Creek in Dallas, which had a joint of the type shown in Figure 1(c). The reinforcing bar dowel was bent 90^{o} in the horizontal plane as shown in Figure 19. Two, WRDD4, shown in Figure 19, and WRDD10F, shown in Figure 16, displayed void areas around bars which were clear to the naked eye when the cores were split open at the reinforcing steel. Evidence of relative movement between steel and concrete in cores WRDLD2F, WRDLD4, and WRDLD5F was revealed in poor bar imprints and dye color in the imprint area. There was no evidence on the top surface of the deck that the voids existed in any of these cores, nor was there evidence of such trouble found on any other bridge deck surface.

The trouble was caused when the dowel moved with respect to the fresh, plastic -- or near plastic -- concrete as loads moved on adjacent lanes. Such action is illustrated in Figure 2. It is believed that the trouble can be eliminated by bringing the dowel out straight, not bending it, about 20 inches. The dowel should then be lap spliced its full length with a bar in the fresh concrete and the splice should be firmly tied or welded to provide complete continuity between the two bars.

The bridge deflection measurements provided information for laboratory experiments on cracking of very young concrete. The laboratory experiments showed that concrete under controlled conditions can be flexed from the time of casting up until about the time of setting without cracking from flexure. It was found that there does not appear to be much danger of flexural cracking in the concrete when traffic continues to use the bridge during casting and curing of the deck. Transverse curvatures calculated from measured bridge beam deflections were small, the largest being 1.139×10^{-5}/inch, the average of eight bridges being 0.566×10^{-5}/inch. The laboratory tests indicated that a curvature of about 3.6×10^{-5}/inch was required to develop cracking in the fresh material at approximately time of set. From these data and projections from calculations, the normal bridge carrying normal traffic can be left open to traffic during placement and curing of concrete adjacent to the traffic lane.

CONCLUSIONS

A study was made of bridges that were widened while normal traffic was maintained in lanes as close as adjacent to fresh concrete used for widening or staged reconstruction. Bridges that had been in service for years and bridges that were under construction were studied. Laboratory beams designed to simulate a new deck placed and cured during normal traffic usage were studied. From the results of the study it is concluded that for bridges made of prestressed concrete and steel beams spanning up to approximately 100 ft:

1. No evidence of problems in concrete placed and cured while traffic was maintained was found in bridges that have been in service for years after the new concrete was placed.
2. Traffic can be maintained during placement and curing without causing flexural problems in the fresh concrete.
3. Voids develop in fresh concrete around dowels bent at a right angle in a horizontal plane upon emerging from the old concrete. No surface evidence was found that these voids cause problems in the performance of the deck. No such voids were found in dowels that extended into the new concrete without bends.
4. Vibrations caused by normal bridge traffic have no detrimental effect on the concrete, the reinforcing steel, nor the interaction between the reinforcing steel and concrete.

RECOMMENDATIONS

Extend all dowel reinforcing bars approximately 24 bar diameters straight into the new deck area. Lap splice these dowels 20 bar diameters with the reinforcing bars that serve as transverse reinforcing for the new concrete. Tie the top-mat steel system to the dowels to prevent relative vertical movement between the dowels and the top-mat steel.

This will provide a space between the end of the top rebars and the break joint for a temporary filler, sometimes used in construction by stages, and sufficient lap length for transfer of flexural stress. The ties are needed to insure that the dowels and the rebars move in harmony, thus preventing the occurrence of voids found at the bent dowels.

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APPENDIX A.

NOTES ON VISUAL INSPECTION OF BRIDGES

This is a list of bridges that were visually inspected as described under Field Inspection. Notes on each bridge follows this list.

1 US 290 - GC&SF RR op. at Brenham
2 I-35 - Ave. D op. at Temple (overlaid)
3 I-35 - AT&SF RR op. at Temple (overlaid)
4 I-45 & FM 517, Dickenson, Galveston County
5 I-45 & FM 518, League City, Galveston County
6 I-45 & FM 519, Near LaMarque, Galveston County
7 SH 7 & Big Sandy Creek
8 SH 7 & Little Sandy Creek
9 SH 7 & Keechi Creek
10 FM 1860 & TP&L Spillway
11 US 84 & Coryell Creek
12 US 84 & Greenbriar Creek
13 SH 36 & Leon River (South of Gatesville)
14 US 84 & Leon River, Gatesville
15 US 84 & Dodds Creek
16 US 84 & Cowhouse Creek
17 US 84 & Langford Creek
18 US 84 & Lampasas River
19 US 281 & Partridge Creek
20 US 281 & Cowhouse Creek
21 SH 36 & Big Bear Creek
22 SH 36 & Little Bear Creek
23 SH 36 & Warring Creek
24 US 281 & Mesquite Creek
25 US 281 & Honey Creek
26 US 79 & GC&SF RR at Milano (overlaid)
27 US 183 & S. San Gabriel River (overlaid)
28 US 183 & N. San Gabriel River (overlaid)
29 SH 22 & North Rocky Creek
30 SH 22 & South Rocky Creek

Number	Name & Description	Condition
1	US 290 & GC&SF RR, Brenham. Passing lane added to southbound lane structure. Five-continuous span, I-beam spans, max. span 70 ft, 45° skew on old structure; 0° skew on new structure (passing lane). Widened 1971 Inspected July 6, 1979	Transverse cracks spaced 12" to 4' apart in both old and new concrete. Some of these crossed the longitudinal joint between old and new concrete. The longitudinal joint appeared as a thin crack with no deterioration. The concrete in the northbound traffic structure, which was not widened, was in or about the same condition as that in the southbound traffic structure, which was widened.
2	I-35, Avenue D, Temple. Three-span continuous I-beam (60, 70, 60 ft). Center strip removed and decked over, and southbound traffic structure widened, one lane with continuous I-beams. (Northbound structure redecked.) Constructed 1952, Widened 1970, Inspected July 16, 1979	Deck overlaid with asphaltic concrete, therefore top surface of the deck was not visible. No deterioration was evident from underside.
3	I-35 & AT&SF RR, Temple. Southbound traffic. Three-span, continuous I-beam (70, 90, 70 ft). 41° 49' skew. Widened one lane with continuous I-beams. Constructed 1952, Widened 1970, Inspected July 16, 1979	Overlaid with asphaltic concrete, therefore top surface of the deck was not visible. No deterioration was evident from underside.
4	I-45 & FM 517, Dickenson. Three-span, zero skew, continuous I-beam (54, 72, 54 ft). Constructed as twin bridges, reconstructed in three stages under traffic conditions. Reconstructed 1977, Inspected July 26, 1979	Transverse cracks spaced 2' to 3' apart in some cases joined by longitudinal cracks. No spalling--no distress attributable to construction under traffic.

Number	Name & Description	Condition
5	I-45 & FM 518, League City. Three-span continuous skew I-beam (54-72-54 ft). Constructed as twin bridges; reconstructed in three stages under traffic.	Transverse cracks spaced 2' to 3' apart; wider cracks in new (central) part than in older (side) part. Some transverse cracks joined by longitudinal cracks in older concrete. No spalling. No evidence that widening has caused any problems.
	Reconstructed 1977, Inspected July 26, 1979	
6	I-45 & FM 519 (15$\pm$ mi. north of Galveston). Three-span, simple, prestressed concrete beam (110, 110, 100 ft). Reconstructed in three stages under traffic.	Very narrow transverse cracks spaced about 15' apart, relatively short (do not extend all the way across the lanes). No longitudinal nor diagonal cracks visible. The deck was in excellent condition.
	Reconstructed 1979, Inspected July 26, 1979	
7	Big Sandy Creek - SH 7 (East of Marlin). Four 40-ft simple I-beams, 44 ft roadway. About 9 ft added, in widening, on each side.	Transverse cracking 8' to 15' apart in new concrete; these are more numerous in eastbound lane than in westbound lane. No longitudinal cracks noted except that of construction joint between old part and new part.
	Widened 1961, Inspected Aug. 7, 1979	
8	Little Sandy Creek & SH 7 (East of Marlin). Four 25 ft slab spans 15 in. thick. Widened about 9½ ft each side.	There were no cracks found in either new or old concrete. Deck is in good condition.
	Widened 1961, Inspected Aug. 7, 1979	

Number	Name & Description	Condition
9	Keechi Creek & SH 7 (East of Marlin). Four 25 ft slab spans 12 in. thick. Each side widened about 9½ ft. Widened 1961, Inspected Aug. 7, 1979	There were no cracks nor other deterioration noted in this bridge.
10	FM 1860 & TP&L Spillway (Lake Creek Reservoir SE of Waco approx. 10 mi.). Simple span 46 ft, concrete girder, 28 ft roadway. Approx. 5 ft-9 in. added to each side. There is very little traffic on this structure. Widened 1956, Inspected Aug. 7, 1979	The deck is in good condition. No deterioration was found.
11	Coryell Creek & US 84 (between McGregor and Gatesville). Twelve simple @ 34 ft-6 in., plus two at 46 ft. R.C. girder spans. The 30 ft (approx) roadway has been widened about 5 ft (plus curbs) on each side. Widened 1959, Inspected Aug. 7, 1979	The deck shows no signs of deterioration that can be charged to widening. It is in good condition.
12	Greenbriar Creek & US 84. Five 34 ft-6 in. simple spans, R.C. beams widened approx. 12 ft on each side. Widened 1959, Inspected Aug. 7, 1979	Transverse cracks 3' to 4' apart in widened portion on one span; other spans show very little cracking. Deck is in good condition.

Number	Name & Description	Condition
13	SH 36 & Leon River about 5 mi. south of Gatesville. Nineteen 35 ft simple span concrete girder spans plus three span, 230 ft. Continuous I-beam unit. Roadway is approx. 30 ft wide. This bridge carries heavy trucks and considerable lighter traffic. Widened 1960, Inspected Aug. 7, 1979	There are numerous rebar exposures from insufficient cover in new portion on east side; scattered popouts occur over a number of the spans along southern portion of new material. There is some minor spalling at some of the exposed rebars. A longitudinal crack near midwidth of the added width were noted in two of the 35 ft spans.
14	US 84 & Leon River at Gatesville. Eighteen 40 ft simple I-beam spans, plus one 3 span cantilever beam unit; 68 ft roadway. The widened part rests on PC beams on 40 ft spans, and on I-beams on cantilever unit. Widened 1979 Inspected Aug. 7, 1979 Cores and deflection measurements were taken later from the deck.	The deck is in good condition. There are scattered narrow transverse cracks. There are longitudinal cracks in the eastbound traffic side (not west) in the new concrete in several of the 40 ft spans, and in one of the long spans. The longitudinal cracks occur over the first new inside beam in short spans, and over the center new beam in the long span. They are narrow cracks showing no spalling.
15	US 84 & Dodds Creek (West of Gatesville). Four 40 ft simple span I-beam widened approx. 8 ft on each side. Widened 1964 Inspected Aug. 7, 1979	The old part of the deck is overlaid with epoxy concrete; the new (widened) part is bare. The visible deck is in good condition. One span of new concrete (south side, easternmost span) is cracked transversely at 8 ft spacing. The concrete in this area is much lighter in color than the other additions.

Number	Name & Description	Condition
16	US 84 & Cowhouse Creek (about 7 mi. west of Arnett). Two 35 ft simple I-beam spans and one 215 ft cantilever-suspended span unit (approx. 65, 85, 65 ft). Widened 1964, Inspected Aug. 7, 1979	The old part of the deck is overlaid with epoxy concrete; the new (widened) part is bare. The bare portion is in good condition; it has transverse cracks spaced 3' to 12' apart.
17	US 84 & Langford Branch (between Arnett and Lampasas R). Five 40 ft simple span, R.C. girder, about 45^{o} skew; 9 ft widening with R.C. beam on each side of bridge. Widened 1964, Inspected Aug. 7, 1979 Widened 1964 Inspected Aug. 7, 1979	The deck is in good condition; no cracks were found.
18	US 84 & Lampasas R. (Hamilton County). Six 34 ft-6 in. and one 46 ft R.C. girder spans. Approx. 13 ft added to each side. Widened 1965 Inspected Aug. 7, 1979	The old part of the deck is overlaid with epoxy concrete; the new (widened) part is bare. Two diagonal and about three transverse cracks in second span from west, in westbound traffic lane. There is no spalling from these cracks. No other signs of deterioration appear.
19	US 281 & Partridge Creek (about 7 mi. north of US 284, Hamilton County). Four 40 ft simple I-beam spans widened about 9 ft on each side using one prestressed concrete beam on each side. Widened 1961 Inspected Aug. 14, 1979	A thin longitudinal crack appears where the new concrete joins the old (over the outside I-beams). A few transverse cracks appear in the widened concrete, some having reflected from the old deck. There are more cracks in the old deck (over steel beams) than in new parts (over PC beam).

Number	Name & Location	Condition
20	US 281 & Cowhouse Creek in Hamilton County, about 9 mi. north of US 284. Seven 40 ft (approx.) and one 50 ft (approx.) simple I-beam spans widened 9 ft on each side with PC beam. Widened 1961 Inspected Aug. 14, 1979	There is slight spalling along crack where new concrete joins old concrete in southbound lane; there are very few transverse cracks. In the northbound lane, southernmost span, transverse cracks are spaced about 12 ft apart in new concrete; about 24 ft apart in other spans.
21	SH 36 & Big Bear Creek about 4 mi. northwest of Hamilton in Hamilton County. Five 40 ft (approx.) simple R.C. girder spans, 45^{o} skew, widened 11 ft on each side. Widened 1968 Inspected Aug. 14, 1979	The old deck is scaled, spalled, and cracked -- primarily diagonal cracks. The northbound lane, first span from south, has a long longitudinal crack in new concrete about 5½ ft from joint between old and new concrete and another about midway between new beam and old outside beam.
22	SH 36 & Little Bear Creek about 4 mi. northwest from Hamilton in Hamilton County. Four 40 ft (approx.) I-beam, simple spans, 30^{o} skew. Extension is about 10'-6" on each side; 44 ft roadway. Widened 1968 Inspected Aug. 14, 1979	Dirt from construction on south traffic approach lane obscured the south traffic lane on the bridge. The north traffic lane widened portion has lots of plastic shrinkage cracks and narrow cracks at construction joint between new and old concrete. There is no evident damage.
23	SH 36 & Warring (Warren) Creek about 7½ mi. northwest from Hamilton in Hamilton County. Three span cantilever-suspended I-beam unit 230 ft long (approx. 67, 96, 67 ft) 30^{o} skew. Approx. 11 ft added to each side of the bridge; 44 ft roadway. Widened 1968, Inspected Aug. 14, 1979	There is transverse and random cracking in the old unit, more cracks than in other bridges inspected. No spalling. New concrete in good condition.

Number	Name & Description	Condition
24	US 281 & Mesquite Creek, Hamilton County: Four simple R.C. girder spans approx. 35 ft long, 45^{o} skew, 44 ft roadway, 10 ft extension on each side. Widened 1961, Inspected Aug. 14, 1979	Transverse cracks spaced at 2½ to 4 ft but no damage to the deck.
25	US 281 & Honey Creek, Hamilton County, about midway between Olin and Hico. Simple span I-beam, 2 @ 30 ft, 2 @ 40 ft with PC beams used in widening about 10 ft on each side. Widened 1961, Inspected Aug. 14, 1979	The old slab is badly cracked in spots but it is not spalled. The new part is in good condition with very few cracks.
26	US Route 79 - GC&SF RR (Milano). Main span approx. 70 ft, simple, 40^{o} skew I-beam structure widened with I-beams. (Other spans widened with PC beams.) Widened 1969, Inspected July 6, 1979	Deck overlaid with asphaltic material therefore top concrete surface was not visible. No deterioration was evident from underside.
27	US 183 & S. San Gabriel River. Widened 1964, Inspected July 13, 1979	<u>Overlaid</u>
28	US 183 & North San Gabriel River. Widened 1969 Inspected July 13, 1979	<u>Overlaid</u>. Spalled longitudinal joint between new and old concrete. No steel crosses the joint.

Number	Name & Description	Condition
29	SH 22 & North Rocky Creek, approx. 10 mi. SE from Meridian, Bosque County. Three span, simple steel I-beam (approx.) 40, 50, 40 ft), 30^{0} skew, 44 ft roadway. The widened part rests on steel I-beams. Widened 1959, Inspected Aug. 14, 1979	The old deck and the joint between the old deck and the extension are covered with asphaltic material; the remainder is bare. The part of the deck that is visible is in excellent condition.
30	SH 22 & South Rocky Creek, approx. 12 miles SE from Meridian, Bosque County. Three 50 ft steel I-beam spans, widened approx. 10 ft-9 in. on each side, using steel I-beams; 44 ft roadway. Widened 1959, Inspected Aug. 14, 1979	The deck is in good condition; no deterioration found.

APPENDIX B

INFORMATION ON BRIDGE DECK CORES

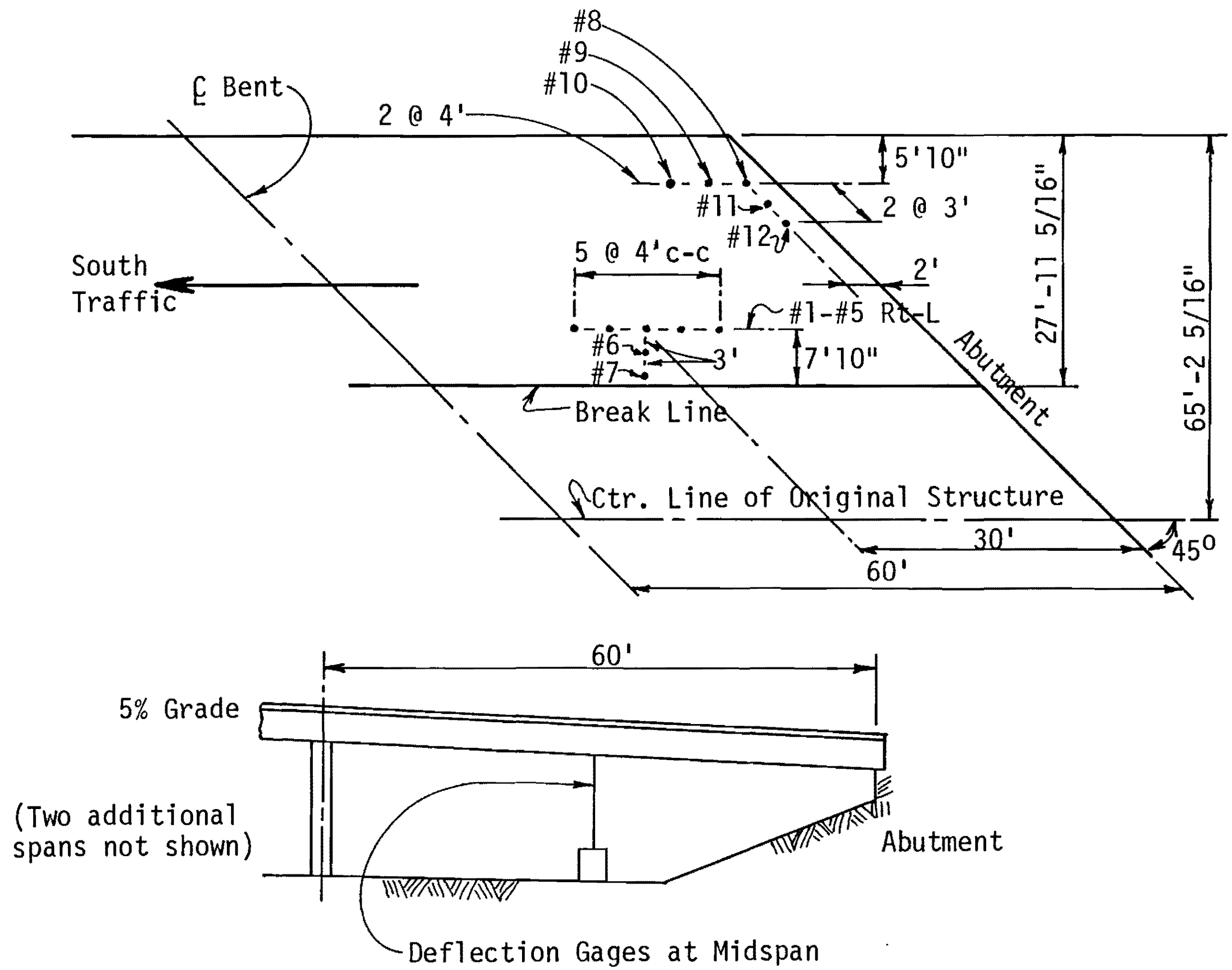

Figure B-1. I-35 & Avenue D Bridge: Core Locations.

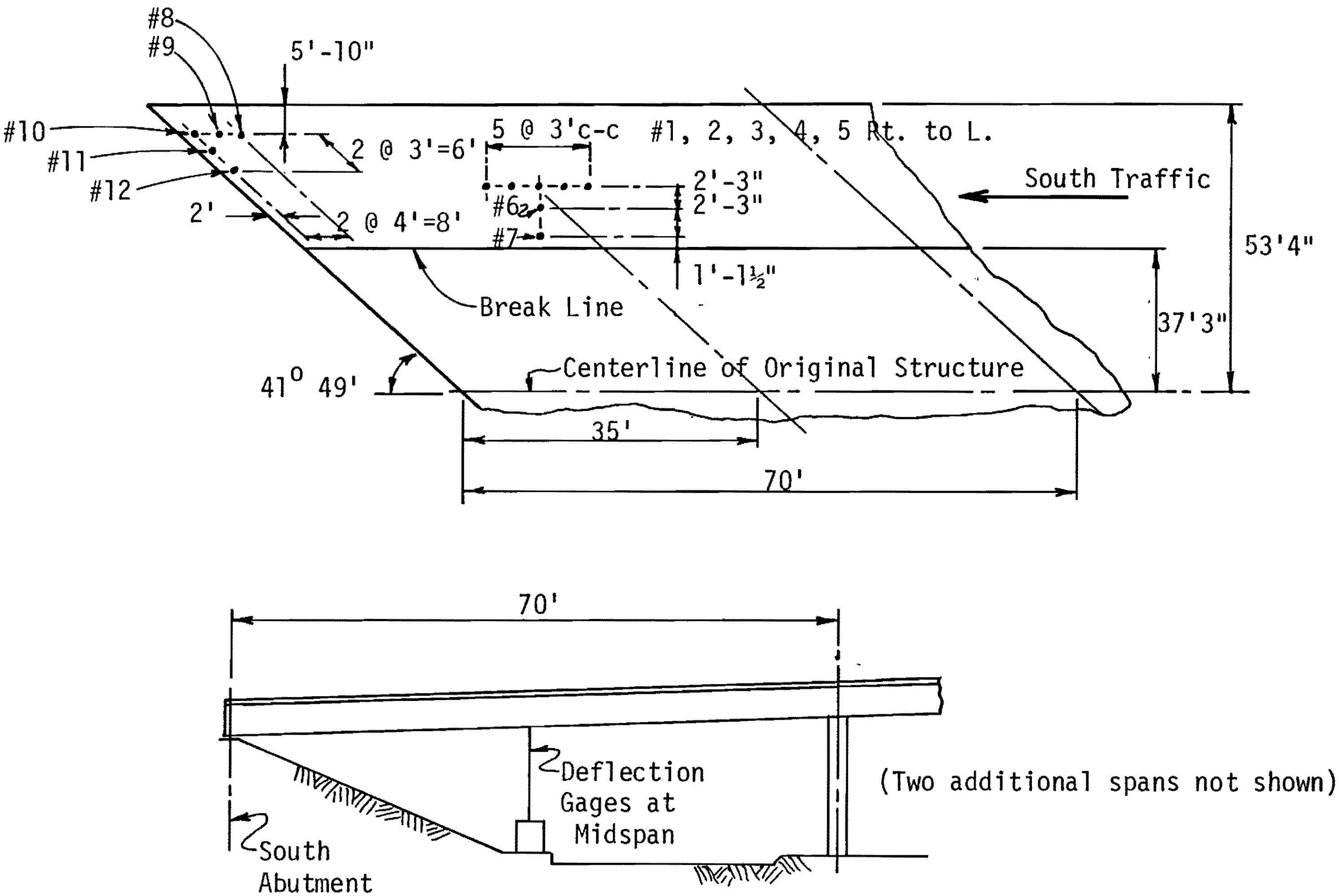

Figure B-2. I-35 & ATSF Railroad: Core Locations.

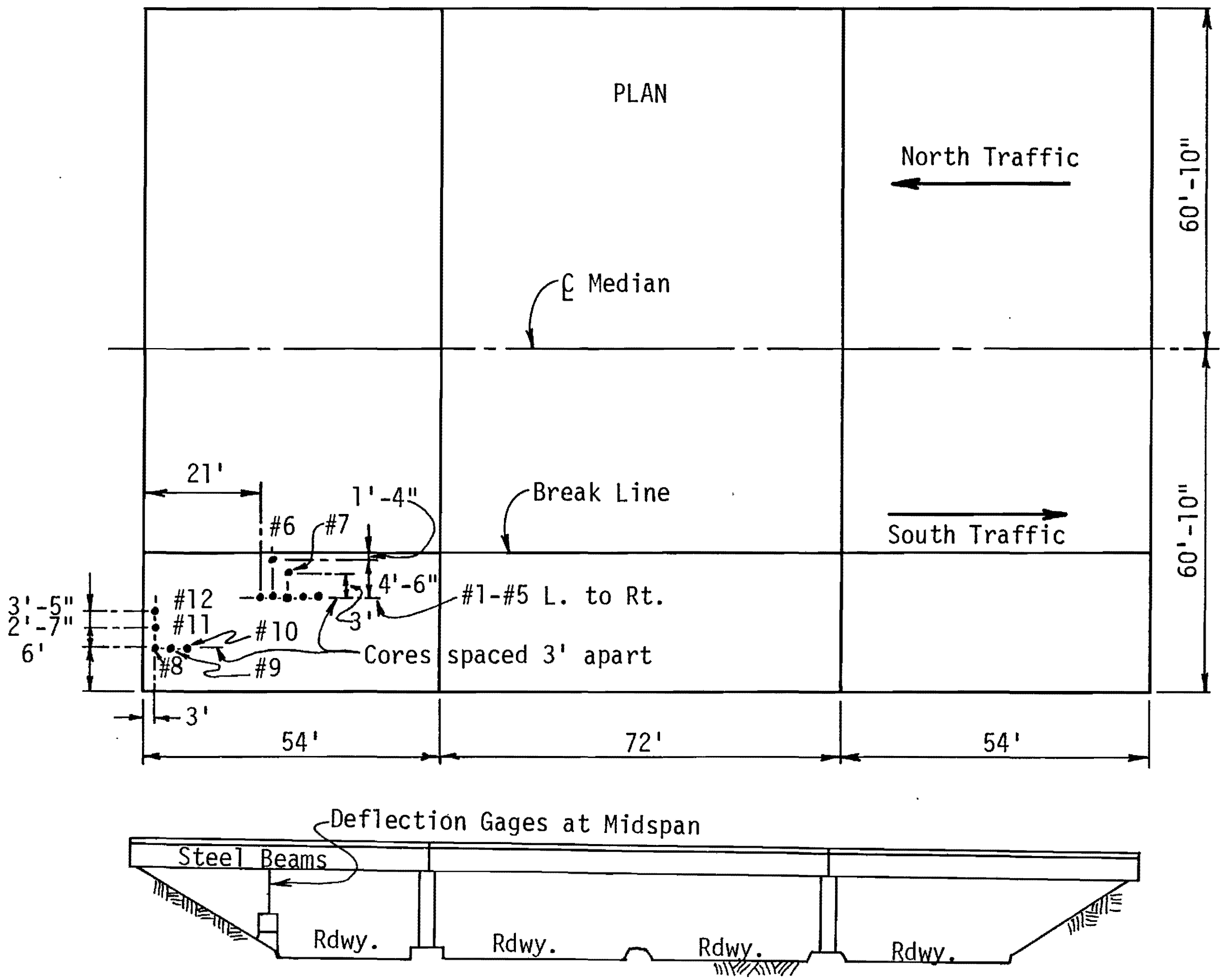

Figure B-3. I-45 & FM 517: Core Locations.

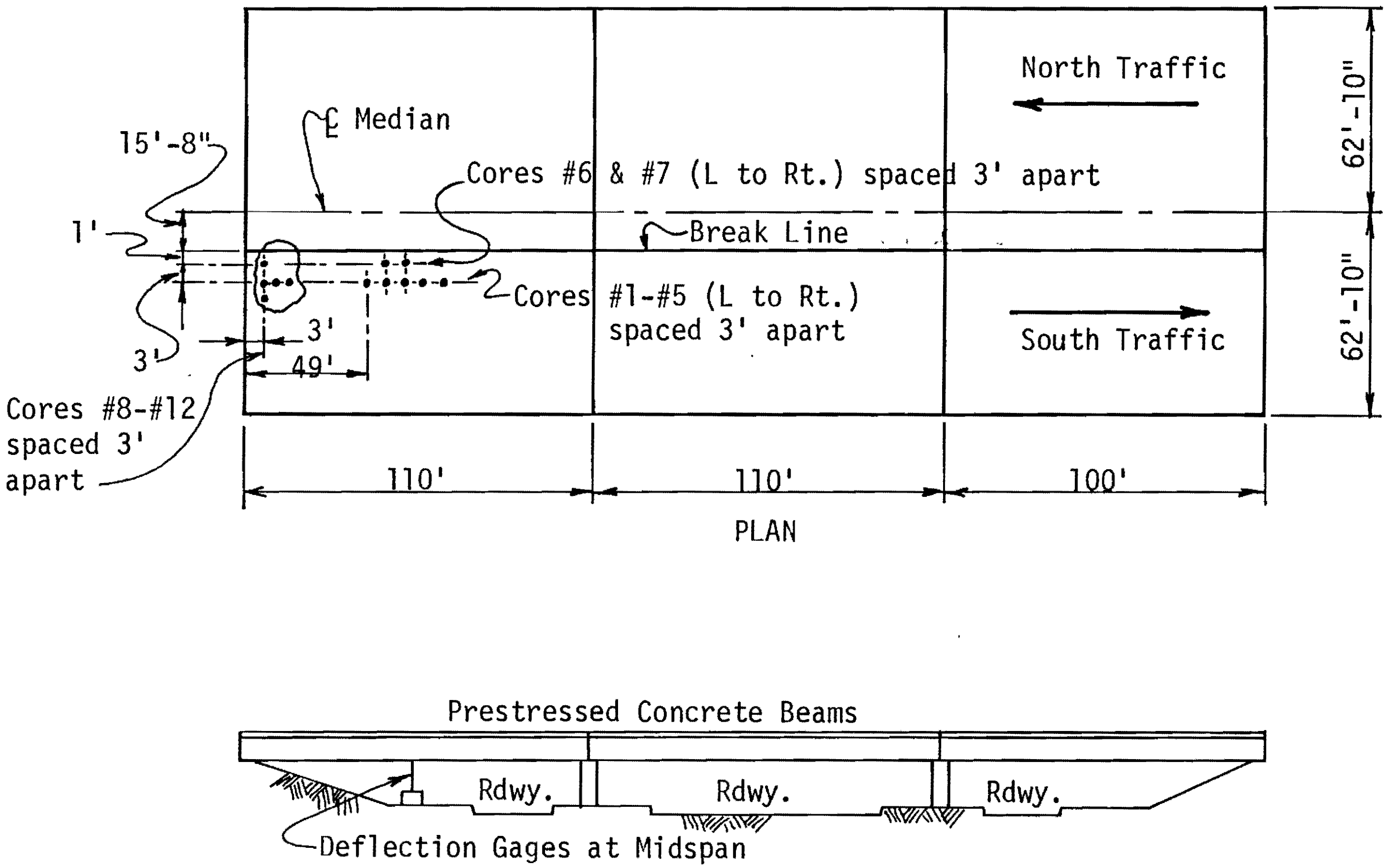

Figure B-4. I-45 & FM 519: Core Locations.

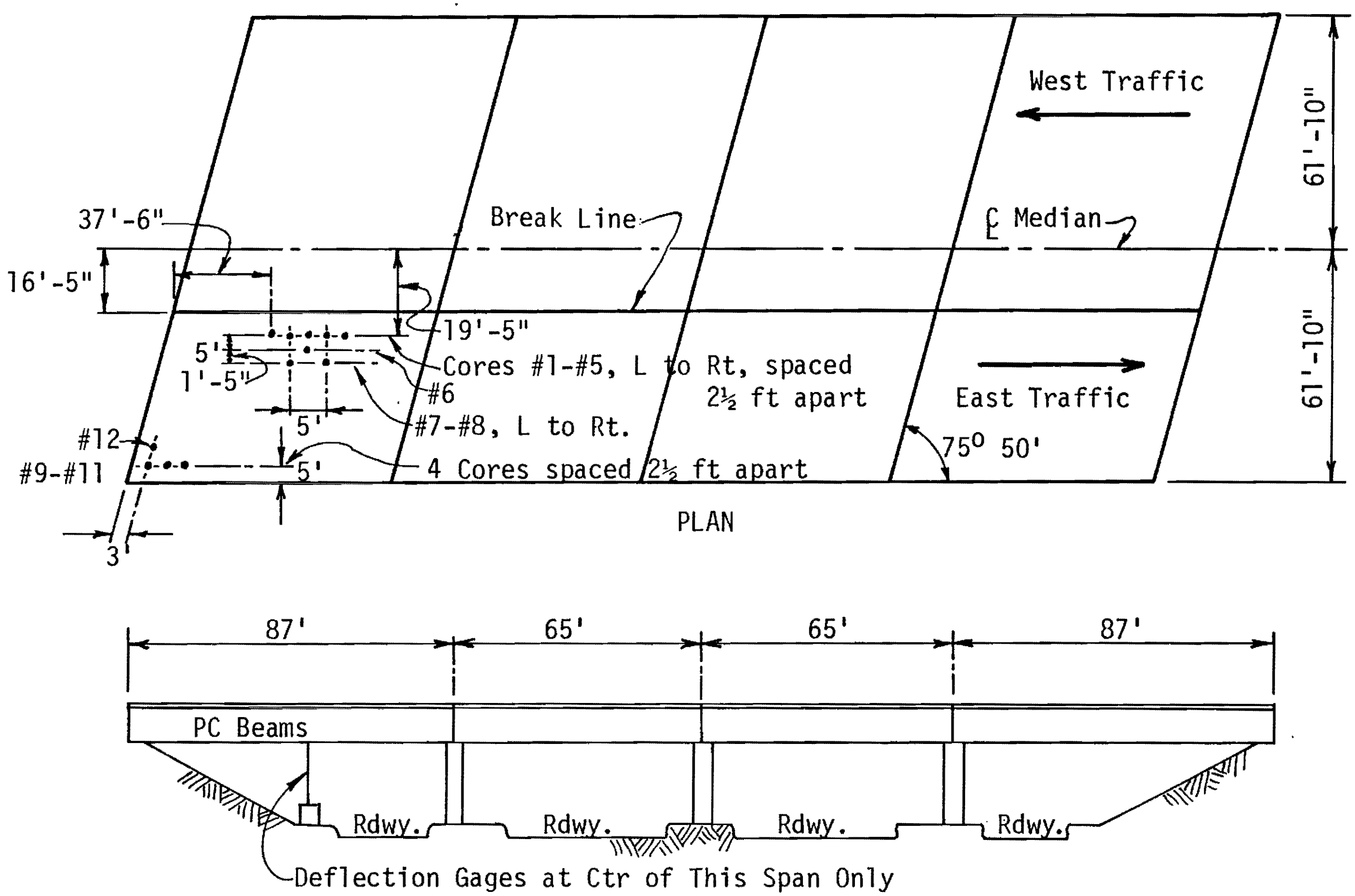

Figure B-5. I-10 & Dell Dale Avenue: Core Locations.

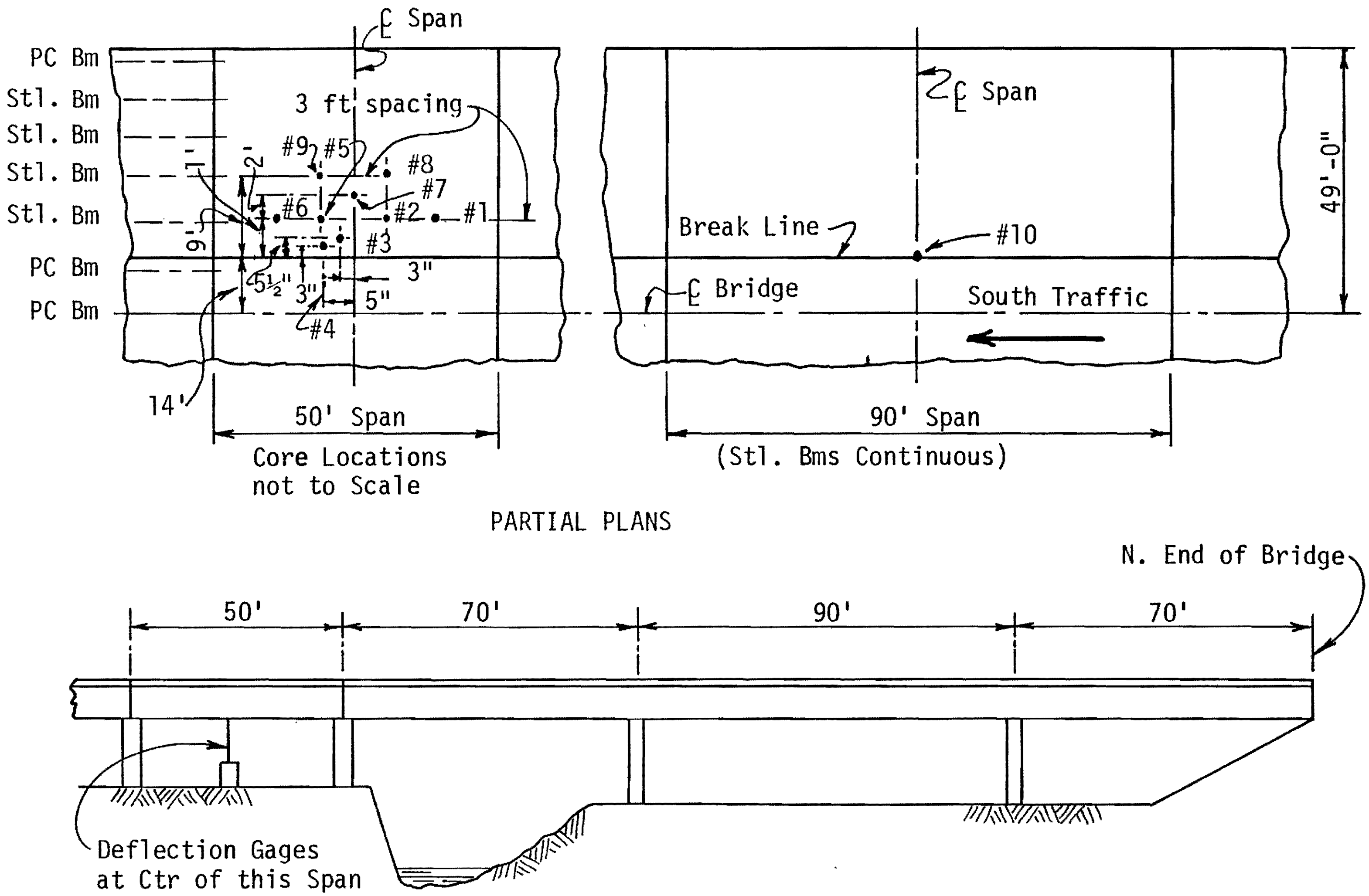

Figure B-6. US 75 & White Rock Creek, Southbound: Core Locations.

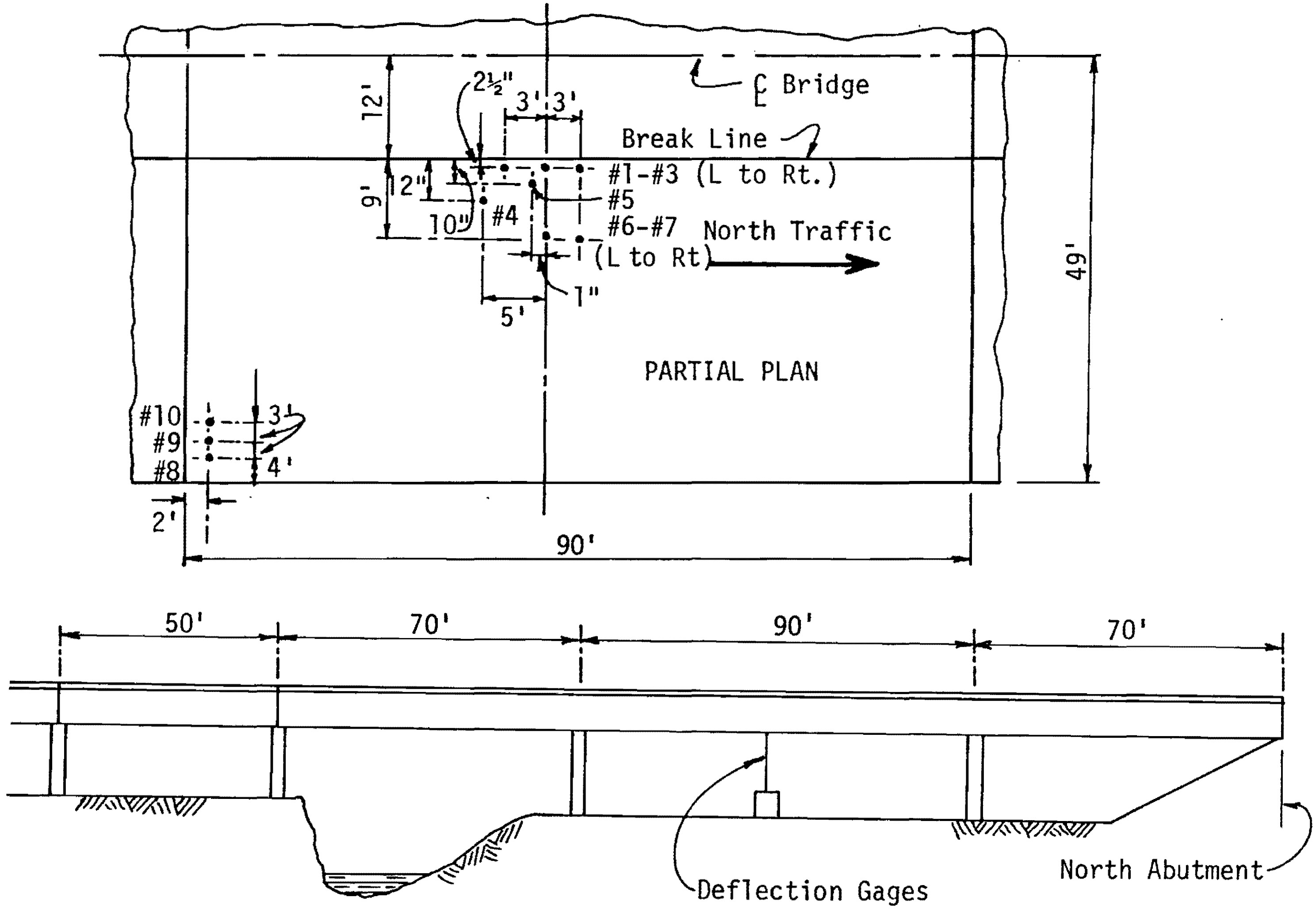

Figure B-7. US 75 & White Rock Creek, Northbound: Core Locations.

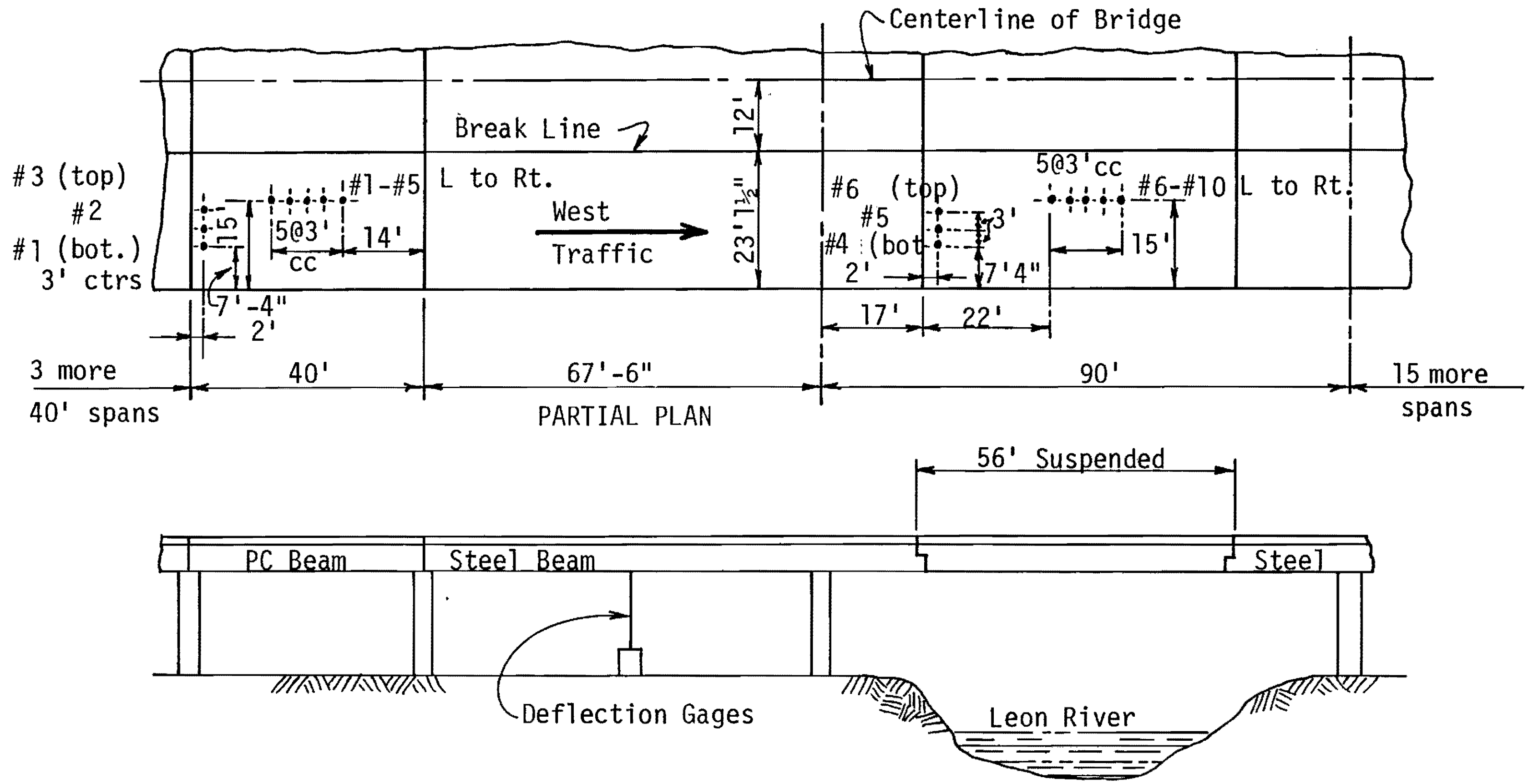

Figure B-8. US 84 & Leon River Bridge: Core Locations.

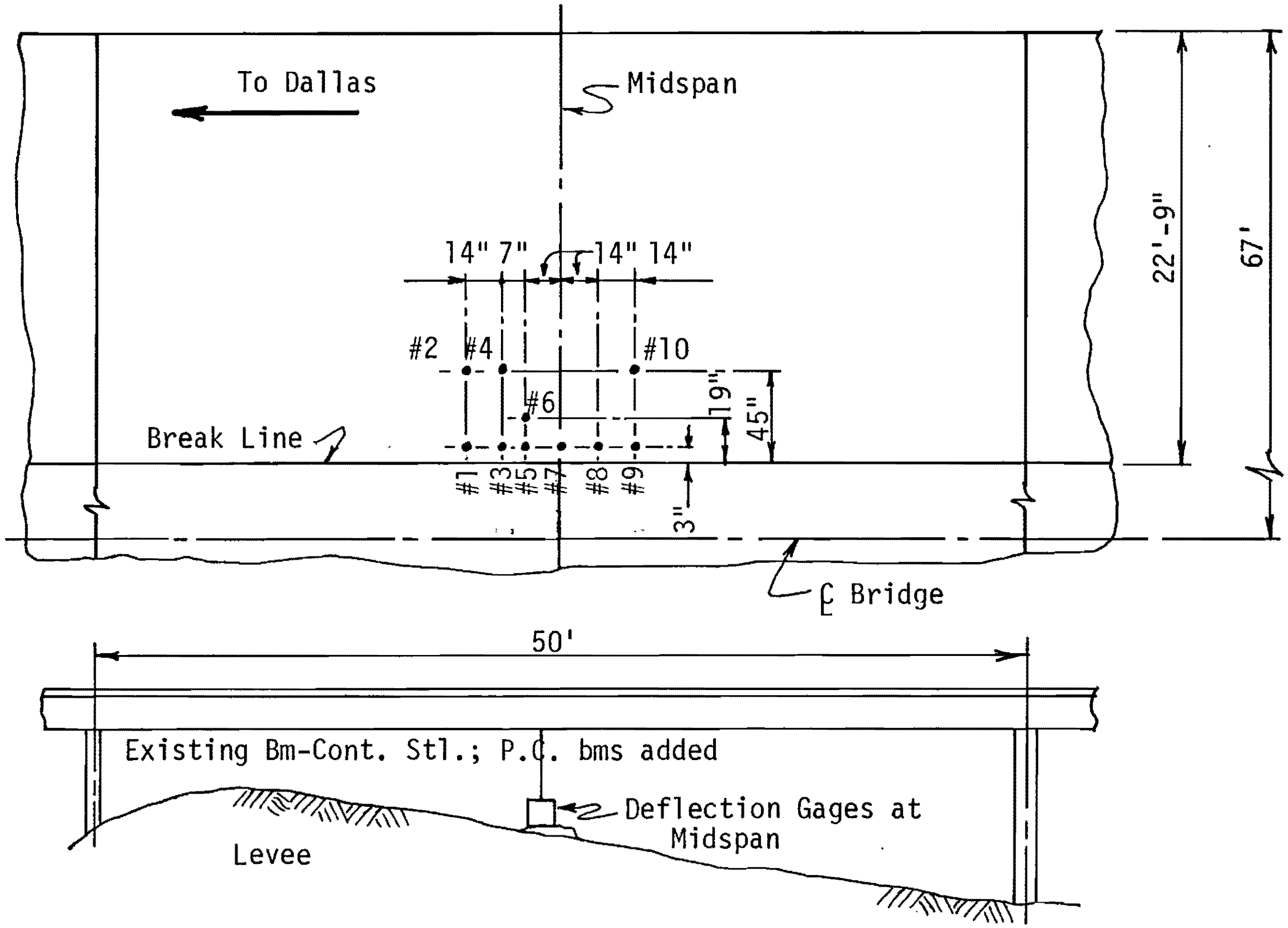

Figure B-9. SH 183 & Elm Fork Trinity River: Core Locations.

TABLE B-1. BRIDGE CORE DATA.

Note: Hairline crack on a surface treated with fluorescent crack indicator indicates one that is seen under black light. In general, such cracks cannot be detected with the naked eye without black light, but can sometimes be detected with the aid of a 2 power glass without black light.

CORE MARK	LENGTH (in.)	TREAT-MENT	CONTAINS REINF.	REMARKS
Bridge: I-35 at Ave. D, Temple, Texas (12 cores). This bridge is overlaid with asphalt.				
ADTU1	5 1/4	TPF*	None	Random hairline cracks, top and bottom.
ADTU2	5 3/8	TPF	Steel	Random hairline cracks, top and bottom.
ADTU3	5 3/4	TPF	Steel	Random hairline cracks, top and bottom.
ADTU4	4 7/8	TPF Dye	Steel	Hairline cracks on top surface running parallel to traffic; air voids and large aggregate more dense at bottom than at top.
ADTU5F	7 5/8	TPF US	Steel	Random hairline cracks, top and bottom.
ADTD1	5 1/4	TPF	None	Random hairline cracks on top, not on bottom.
ADTD2	3 5/8	TPF	None	Random hairline cracks, top and bottom.
ADTD3	3 1/8	TPF	None	Random hairline cracks, top and bottom.
ADTD4	4 1/2	TPF	None	Random hairline cracks, top and bottom.
ADTD5F	7 1/4	TPF	Steel	One 0.03 inch wide crack, perpendicular to traffic runs full length and width of core, intersecting #5 top bar. Random hairline cracks, top and bottom.

*TPF meaning: T: trimmed by squaring ends with a saw
P: trimmed surface is polished
F: fluorescent crack detector applied to polished surface

US meaning: Sonic modulus and strength tests made on these.

Dye meaning: Dye test made for study of bond.

CORE MARK	LENGTH (in.)	TREAT-MENT	CONTINUOUS REINF.	REMARKS
ADTD6F	8 3/4	TPF US	Steel	Random hairline cracks, top and bottom.
ADTD7	4 1/4	TPF Dye	Steel	Random hairline cracks, top and bottom.

Bridge: I-35 at AT&SF RR, Temple, Texas (12 cores). (Relatively soft and porous concrete) This bridge is overlaid with aspahlt.

CORE MARK	LENGTH (in.)	TREAT-MENT	CONTINUOUS REINF.	REMARKS
RRTU1	2 1/2	T	None	No visible defects.
RRTU2F	6 7/8	TPF	Steel	Crack across top, perpendicular to traffic extending to below top steel; random hairline cracks.
RRTU3	4	None	Steel	Two narrow hairline cracks on top surface; one perpendicular to traffic, the other parallel, each is 1 in. long beginning at edge of core; depth is indeterminate.
RRTU4	6 5/8	TPF	Steel	Random hairline cracks, top and bottom.
RRTU5	4 1/2	TPF US	Steel	Random hairline cracks, top and bottom.
RRTD1	4	TPF US	Steel	Diagonal crack 0.01 in. wide on bottom of broken core extending upward 2 in. Random hairline cracks, top and bottom.
RRTD2	3	T	Steel	No visible defects.
RRTD3	2 1/2	TPF	Steel	Random hairline cracks, top and bottom. One pronounced hairline crack extends across the top surface and runs perpendicular to traffic; depth indeterminate.
RRTD4	4 1/2	T	Steel	Crack of indeterminate depth across bottom (at level of bottom steel) perpendicular to traffic.

CORE MARK	LENGTH (in.)	TREAT- MENT	CONTINUOUS REINF.	REMARKS
RRTD5	4 1/2	TPF	Steel	Hairline crack of undetermined depth, perpendicular to traffic, runs across bottom surface; no cracks seen on top.
RRTD6	4 1/4	TPF	None	Random hairline cracks, top and bottom.
RRTD7F	6 3/8	T	Steel	No visible defects.

Bridge: US 84 at Leon River, Gatesville, Texas (16 cores)
40 ft simple span, fourth span from west end.

CORE MARK	LENGTH (in.)	TREAT- MENT	CONTINUOUS REINF.	REMARKS
LRGU1	4	TPF Dye	Steel	Random hairline cracks, top and bottom.
LRGU2	4 1/2	None	Steel	Narrow crack, depth indeterminate, perpendicular to traffic runs across top of core.
LRGU3	5 1/8	None	Steel	Top surface crack diagonal to traffic; mud lump, 5/8 in. diameter centered one in. from top.
LRGD1F	7 1/4	None	Steel	No visible defects. Core broke at bottom mat of steel leaving imprint of a steel bar; mill scale adhering to concrete indicates good bond.
LRGD2	3	None	None	No visible defects.
LRGD3F	7 1/8	TPF US	Steel	Very thin crack on top surface, full width, diagonal to traffic, depth indeterminate. Random hairline cracks top and bottom.
LRGD4F	7 1/4	TPF US	Steel	Random hairline cracks, top and bottom.
LRGD5F	7 1/2	None	Steel	Very narrow crack across top, perpendicular to traffic, about 1/4 inch deep.

CORE MARK	LENGTH (in.)	TREAT-MENT	CONTINUOUS REINF.	REMARKS
Suspended Span:				
LRGU4	3	TPF	Steel	Random hairline cracks, top and bottom.
LRGU5F	7 3/4	None	Steel	Random hairline cracks of indeterminate depth on top surface.
LRGU6	3 1/2	None	Steel	Narrow shallow crack across top parallel to traffic.
LRGD6	4	None	None	One inch deep crack perpendicular to traffic across top. Steel imprint at break in core indicates good bond.
LRGD7	4 1/2	None	Steel	Narrow crack on top, diagonal to traffic, depth indeterminate.
LRGD8F	8 1/4	None	Steel	Clearly visible, 1/2 in. deep, top crack perpendicular to traffic.
LRGD9F	8	TPF	Steel	Cracked from both top and bottom, perpendicular to traffic, about 2 in. of uncracked concrete near mid-depth separates these cracks. Random hairline cracks, top and bottom.
LRGD10F	8 1/2	TPF Dye	Steel	Random hairline cracks on top; none evident on bottom.
Bridge: I-45 at Dell Dale Ave., Houston, Texas (12 cores)				
DDHU1F	7 1/2	TPF US	None	Random hairline cracks, top and bottom.
DDHU2	6	None	None	No visible defects.
DDHU3	5 1/2	None	Steel	No visible defects.
DDHU4	5 3/4	TPF	None	Random hairline cracks on top, none on bottom.
DDHD1	4 1/2	None	Steel	No visible defects.
DDHD2	4 1/2	None	Steel	No visible defects.

CORE MARK	LENGTH (in.)	TREAT- MENT	CONTINUOUS REINF.	REMARKS
DDHD3F	7 3/8	TPF	Steel	Random hairline cracks on top, none on bottom.
DDHD4F	7 1/4	TPF US	Steel	Random hairline cracks, top and bottom.
DDHD5	5	None	Steel	No visible defects; mill scale on longitudinal bar imprint at break indicates good bond.
DDHD6	4 1/4	None	Steel	No visible defects; mill scale on longitudinal bar imprint at break indicates good bond.
DDHD7F	7 1/4	TPF	Steel	Random hairline cracks on top, none on bottom.
DDHD8	5 1/2	None	Steel	No visible defects; mill scale on longitudinal bar imprint at break indicates good bond.

Bridge: I-45 at FM 517, Houston, Texas (12 cores)

CORE MARK	LENGTH (in.)	TREAT- MENT	CONTINUOUS REINF.	REMARKS
F7HU1	5 1/2	None	Steel	No visible defects.
F7HU2F	7 3/4	TPF	Steel	One distinct hairline crack across top surface, perpendicular to traffic. Random hairline cracks, top and bottom.
F7HU3	5	None	Steel	No visible defects.
F7HU4	5 3/4	None	Steel	No visible defects.
F7HU5	6 1/4	TPF	Steel	Random hairline cracks, top and bottom.
F7HD1F	7 3/4	TPF	None	Random hairline cracks, top and bottom.
F7HD2	5 1/4	None	Steel	No visible defects.
F7HD3F	7 3/4	TPF US	Steel	Random hairline cracks, top and bottom.
F7HD4	3	None	None	No visible defects.

CORE MARK	LENGTH (in.)	TREAT-MENT	CONTINUOUS REINF.	REMARKS
F7HD5F	8 1/4	TPF US	Steel	Random hairline cracks, top surface only.
F7HD6	4 1/2	None	Steel	No visible defects.
F7HD7	5 1/2	None	None	No visible defects.
Bridge: I-45 at FM 519, Houston, Texas (12 cores)				
F9HU1	5 3/4	None	Steel	No visible defects.
F9HU2	6	TPF Dye	Steel	Random hairline cracks on top only.
F9HU3	5 1/4	None	Steel	No visible defects; sharp imprint of bar at break.
F9HU4	5 1/4	None	Steel	No visible defects.
F9HU5F	8	TPF US	Steel	One hairline crack, perpendicular to traffic, across top surface, depth indeterminate. Random hairline cracks, top and bottom.
F9HD1	5 1/2	None	Steel	No visible defects.
F9HD2	5	TPF Dye	Steel	Random hairline cracks on bottom only.
F9HD3F	7 5/8	TPF	Steel	Random hairline cracks, top and bottom.
F9HD4F	7 3/4	TPF US	Steel	Random hairline cracks, top and bottom.
F9HD5F	7 3/4	TPF	Steel	Random hairline cracks, top and bottom.
F9HD6	2 1/2	None	None	No visible defects. Top steel left sharp imprint at break.
F9HD7	4	None	Steel	No visible defects.

CORE MARK	LENGTH (in.)	TREAT-MENT	CONTINUOUS REINF.	REMARKS
Bridge: US 75 at White Rock Creek, Dallas, Texas Southbound traffic, 50 ft span except core WRD10F which comes from the 90 ft span (13 cores)				
WRDU1	6 1/4	TPF	None	Random hairline cracks.
WRDU2	6 3/4	TPF	Steel	No visible defects.
WRDU3	3	TPF	None	Random hairline cracks.
WRDD1F	8 1/2	TPF	Steel	Random hairline cracks.
WRDD2F	8 1/2	TPF US	Steel	Random hairline cracks; numerous large air bubbles.
WRDD3F	8 1/4	TPF Dye	Steel	The two top #5 bars, one perpendicular to traffic, the other parallel to traffic, have small crack-like voids on one exposed side of each bar. Random hairline cracks.
WRDD4F	8 1/4	TPF Dye	Steel	Random hairline cracks; many air bubbles.
WRDD5	8 1/4	TPF US	Steel	No visible defects.
WRDD6F	8 1/4	TPF	Steel	Random hairline cracks.
WRDD7	4 3/4	TPF Dye	Steel	No visible defects.
WRDD8	6	TPF	Steel	Random hairline cracks.
WRDD9	4 1/2	TPF	Steel	Random hairline cracks.
WRDD10F	8 1/4	None	Steel	Top of core is cracked parallel to traffic; concrete in upper part appears to have broken after set began. Core is separated horizontally at level of #5 new rebar (the end 3 in. is in the core), the #4 longitudinal bar tied to and just below the #5 new rebar, and the #5 rebar dowel from the old concrete (this bar is split by the core bit at its horizontal bend) which is just beneath the #4 bar. The two parts, separated in coring, leave an open crack, see photo to right, about 1/4 in. wide when they are carefully fitted together. The imprint of the #4 bar has no bar

CORE MARK	LENGTH (in.)	TREAT-MENT	CONTINUOUS REINF.	REMARKS
				deformation marks, and its surface appears to have been puddled when the concrete was plastic. Clearly, the relative movements of the old and new decks caused this.
Northbound traffic, 90 ft span (10 cores)				
WRDLU1F	8 1/2	TPF	Steel	Random hairline cracks, top and bottom.
WRDLU2	6	None	Steel	No visible defects; bar impression contains mill scale indicating good bond.
WRDLU3F	8 3/4	TPF US	Steel	Random hairline cracks, bottom only.
WRDLD1F	8 1/2	TPF	Steel	The #5 dowel bar from existing deck to new deck was cut in coring across the joint between new and old concrete. A definite void around the dowel in the new concrete exists. This indicates relative vertical movement between the dowel and the new concrete. No other distress is visible.
WRDLD2F	8	TPF Dye	Steel	Cored across joint between new and existing deck cutting #5 dowel. A #4 longitudinal bar, and the end of a #5 transverse bar, are wired together in the core. The concrete around the bars is not compact; it contains voids and cracks which are probably caused by relative movement between the dowel (and the other steel wired to it), and the fresh concrete. Random hairline cracks, top and bottom.
WRDLD3F	8 3/4	TPF	Steel	Top #5 bar perpendicular to traffic, and top #4 bar parallel to traffic have voids around them. The core broke at the level of these bars, revealing poor consolidation and evidence of relative movement between the steel and fresh concrete. Random hairline cracks, top and bottom.

CORE MARK	LENGTH (in.)	TREAT-MENT	CONTINUOUS REINF.	REMARKS
WRDLD4	6	None Dye	Steel	Void space visible at top and one side of #5 top bar laying perpendicular to traffic. The core broke at level of bottom steel, leaving imprint and mill scale from a bottom bar.
WRDLD5F	8 3/4	TPF Dye	Steel	Void space visible at top of #5 top bar; random hairline cracks, top and bottom.
WRDLD6	5 3/4	None	Steel	No visible defects.
WRDLD7	6 1/4	None	Steel	No visible defects.

Bridge: Route 183 at Elm Fork of Trinity River, Dallas, Texas (10 cores)

CORE MARK	LENGTH (in.)	TREAT-MENT	CONTINUOUS REINF.	REMARKS
EFD1	6	None	Steel	No visible defects.
EFD2F	8 1/4 to 10	None	Steel	Honeycomb 1/2 in. from top surface; numerous air bubbles 1½ in. from top; no cracks. This core was cut through bottom corrugated metal form, hence the length 8¼ to 10 in.
EFD3	6 1/4	None	Steel	Numerous air bubbles, but no cracks.
EFD4	5 3/4	None	Steel	Numerous air bubbles, air void on each side of cut at top of #5 bar perpendicular to traffic. No visible cracks.
EFD5F	9	None	Steel	Numerous air bubbles; no visible defects.
EFD6F	8 1/2	None	Steel	Numerous air bubbles; no visible defects.
EFD7F	8 7/8	None	Steel	Many air bubbles; no visible cracks.
EFD8F	9	None	Steel	No visible defects.

CORE MARK	LENGTH (in.)	TREAT- MENT	CONTINUOUS REINF.	REMARKS
EFD9F	9	None	Steel	Numerous air bubbles, more at about 1/2 in. from top than elsewhere.
EFD10	6 1/4	None	Steel	No visible defects.

APPENDIX C

COMPUTATION OF CURVATURE

The procedure used in computing curvatures assumes that an arc of a circle passes through three consecutive points on the deflected shape.

The equation of a circle,

$$(x - a)^2 + (y - b)^2 = R^2$$

where x and y are the coordinates of a point on the circle

a and b are the x and y coordinates of the center of the circle, respectively

R is the radius of the circle

was used in developing relations which were programmed to compute the curvatures of the deflected shape. The relations are summarized below:

- $A = x_1^2 - x_2^2 + y_1^2 - y_2^2$
- $B = x_1^2 - x_3^2 + y_1^2 - y_3^2$
- $a = \dfrac{A(y_1-y_3) - B(y_1 - y_2)}{2\left[x_1(y_2-y_3) + x_2(y_3-y_1) + x_3(y_1-y_2)\right]}$
- $b = \dfrac{A/2 - (x_1-x_2) \cdot a}{(y_1-y_2)}$
- $R = \sqrt{(x_1-a)^2 + (y_1-b)^2}$
- Curvature = 1/R