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FIELD EVALUATION OF BONDED CONCRETE OVERLAYS

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by

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INTRODUCTION

A field program of strain and deflection measurements was conducted by the Construction Technology Laboratories (CTL) for the Iowa Department of Transportation. The objective of the field measurement program was to obtain information on bonded concrete resurfaced pavements that can be used as a data base for verifying bonded resurfacing thickness design procedures. Data gathered during the investigation included a visual condition survey, engineering properties of the original and resurfacing concrete, load related strain and deflection measurements, and temperature related curl (deflection) measurements.

Resurfacing is basically the addition of a surface layer to extend the life of an existing pavement. Portland cement concrete has been used to resurface existing pavements since about 1913.

For many years concrete resurfacings were designed based on experience or engineering judgment. Use was also made of the Corps of Engineers procedure for design which requires a coefficient that rates the condition of the existing pavement. However, since the rating for this procedure is based on the amount of surface cracking it is subjective. In the last few years, several more rational procedures have been developed for concrete surfacings. These procedures incorporate an evaluation of the existing pavement by nondestructive load testing and/or use the finite element methods of analysis

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to establish overlay thickness requirements. A recent design procedure for bonded overlays developed by the Portland Cement Association (PCA) is based on the finite element method of analysis.[1] This procedure incorporates the strength characteristics of the existing and overlay pavement to compute overlay thickness. The procedure currently used by the Iowa DOT to establish bonded overlay thickness requires use of the Road Rater equipment to evaluate the existing pavement.

Field load testing was conducted by CTL at five sites in Iowa during April 1986. This report presents the results of field testing, analysis of results, and recommendations to incorporate study results in Iowa's design procedure for bonded concrete overlays.

RESEARCH OBJECTIVES

Objectives of the study were as follows:

1. Perform condition survey and load testing of the overlaid pavement sections.
2. Analyze field data.
3. Prepare a report containing a discussion of use of the field data to verify design procedures for bonded concrete overlays.

PAVEMENT TEST SECTIONS

Field measurements were obtained at five pavement sections located in the State of Iowa. A brief description of each pavement section follows:

Section 1. This test section, located along the westbound lanes near Mile Post 190 on I-80, is a 24-ft-wide roadway. The original pavement, constructed in 1964, is jointed reinforced concrete with joints spaced at

Numbers in raised parentheses refer to references at end of text.

76 ft-6 in. This pavement is nominally 10 in. thick and has not been overlaid. The outside shoulder consists of a granular base and asphalt concrete wearing surface.

Section 2. This test section is located adjacent to (just west of) Section 1 and is also a 24-ft-wide roadway. The pavement is jointed reinforced concrete with joints spaced at 76 ft-6 in. The pavement section had been overlaid with portland cement concrete. The original pavement, constructed in 1964, is nominally 10 in. The overlay was constructed in 1984 and is nominally 4 in. thick. The outside shoulder consists of a granular base and asphalt concrete wearing surface.

Section 3. This test section is located along the northbound lane near Sta. 435+20 on County Road T-61, just south of Eddyville along the Monroe and Wapello County Line. The original pavement, constructed in 1972, is reinforced concrete with joints spaced at 40 ft. The pavement section has been overlaid with portland cement concrete. The overlay, constructed in 1985, is plain concrete. In the overlay, transverse joints were provided to match the joints in the existing pavement at spacing of 40 ft and intermediate joints were provided at spacing of 20 ft. The original pavement thickness is nominally 6 in. and thickness of overlay is about 4 in. The shoulder consists of a granular base.

Section 4. This test section is located along the eastbound lanes of I-80 near Mile Post 39 just west of the Avoca interchange. The pavement is continuously reinforced concrete (CRC) and is overlaid. The existing pavement thickness is nominally 8 in. and overlay is nominally 3 in. thick. The outside shoulder consists of a granular base and asphalt concrete wearing surface.

The original pavement was constructed in 1966 and exhibited D-cracking deterioration at time of overlay in 1979.

Section 5. This test section is located adjacent to (just east of) Section 4. The original pavement is jointed reinforced concrete with joints spaced at 76 ft-6 in. The pavement is overlaid with portland cement concrete. Thickness of the original pavement is nominally 10 in. and the overlay is nominally 3 in. thick.

The original pavement was constructed in 1965 and exhibited D-cracking deterioration at time of overlay in 1979.

BONDED OVERLAY CONSTRUCTION

When a bonded concrete overlay is used, measurements are taken to ensure a complete bond with the existing pavement so that the overlay becomes an integral part of the base slab. A schematic of a bonded overlay is shown in Fig. 1.

This section summarizes Iowa's construction procedures for bonded overlays. The procedures described were used for the overlay construction at Sections 4 and 5 along the eastbound lanes of I-80 in Pottawattawie County just west of Avoca.

A 4-1/2-mile section of I-80 was resurfaced in 1979 with nominally 3-in.-thick bonded plain concrete. The resurfaced pavement was an 8-in.- thick CRC except for about 2,100 ft of 10-in. thick jointed reinforced concrete near the east end of the project. The resurfaced pavement exhibited considerable D-cracking along joints and cracks.

The existing surface milled to a depth of about 1/4 in., was then cleaned by sandblasting and air-blasting. A water-cement grout was sprayed onto the cleaned surface just ahead of the overlay placement. Work also included

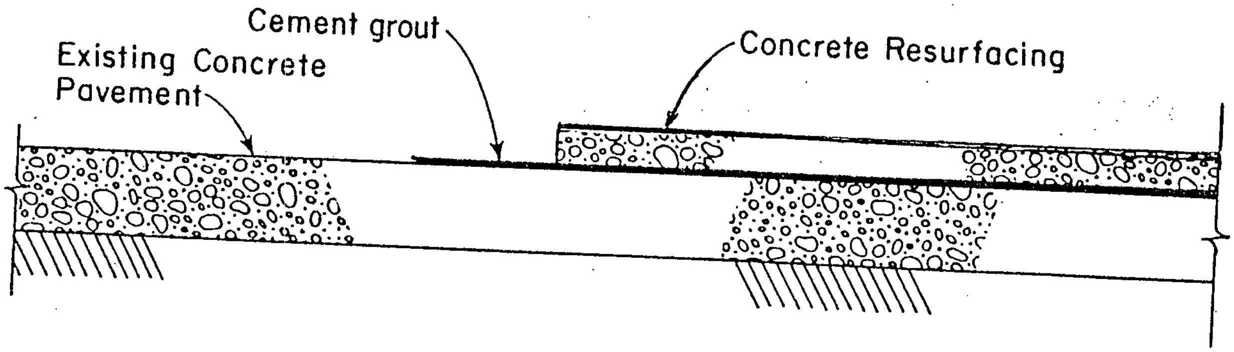


Fig. 1 Bonded Concrete Overlay

installation of edge drains and installation pressure relief joints in the existing pavement and the overlay. Transverse joints were provided in the bonded overlay to match joints in the existing pavement along the jointed portion of the project.

CONDITION SURVEY OF TEST SECTIONS

A visual condition survey was conducted at each test section by walking along the length of the test section. For Sections 1, 2, and 5, the length surveyed was about 300 to 350 ft. For Sections 3 and 4, the length surveyed was about 100 ft. Extent and severity of visible cracking was noted. For jointed pavements, severity of faulting was also noted. Results of the condition survey presented in the following paragraphs and figures.

Test Section 1

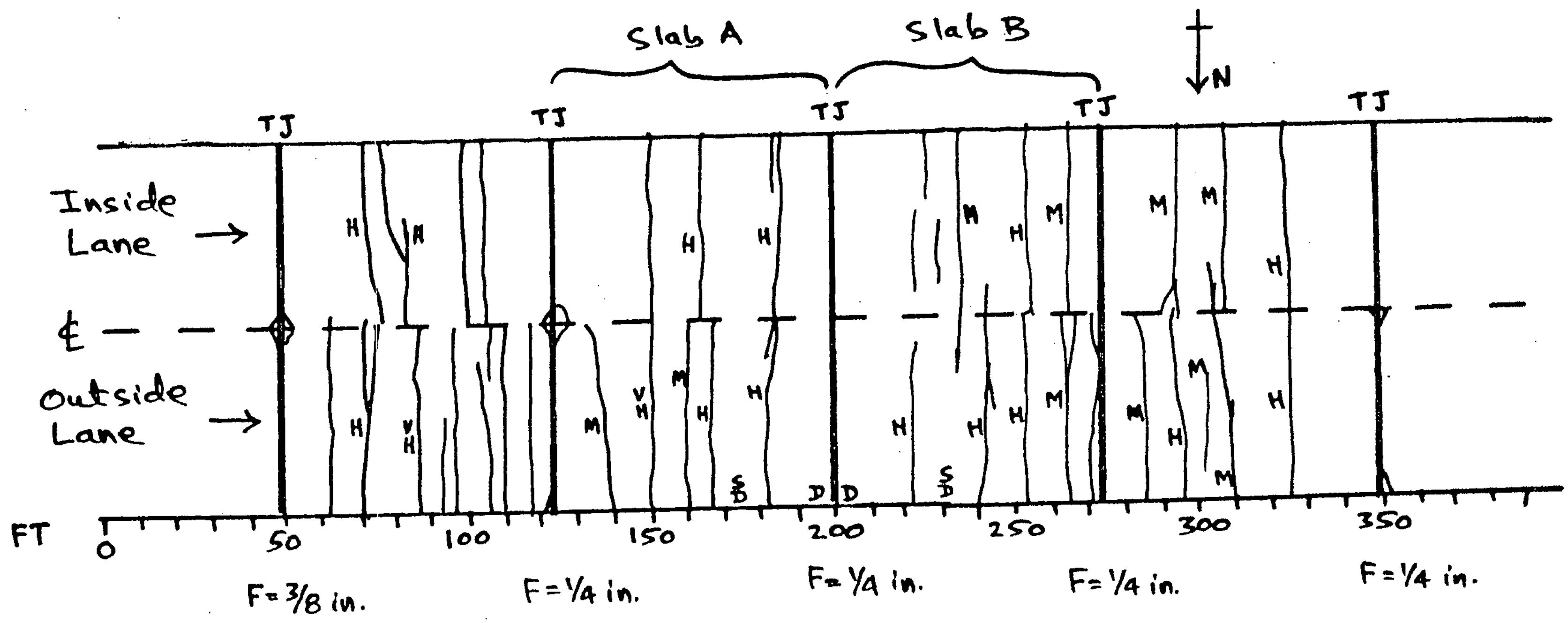
The condition survey for Section 1 is given in Fig. 2. As seen in Fig. 2, there is a large amount of transverse cracking within the test section area. Cracking was generally of low to medium severity. Few cracks did exhibit high severity. Transverse joints were faulted about 1/4 to 3/8 in.

Two slab panels, denoted Slab A and Slab B, selected for instrumentation are also indicated in Fig. 2.

Test Section 2

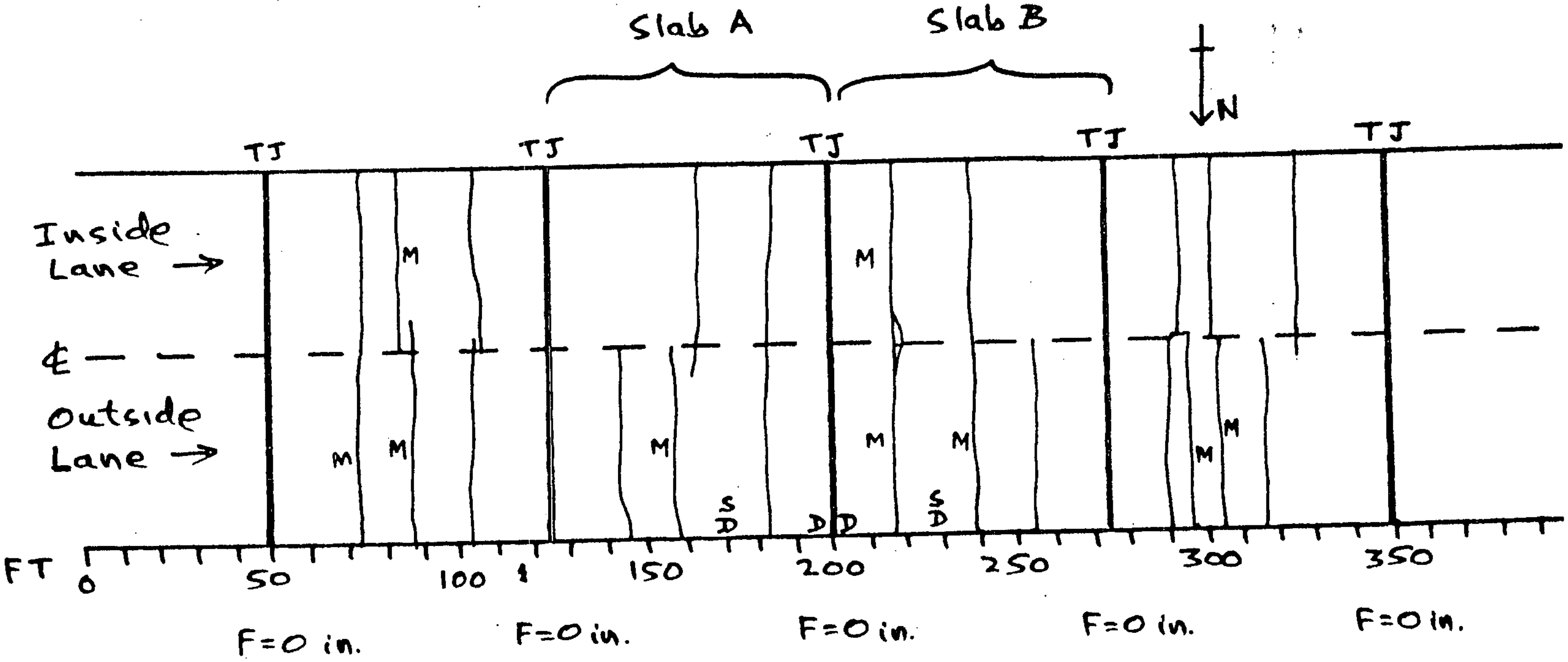
The condition survey for Section 2 is given in Fig. 3. Cracking in Section 2 is not as extensive as for Section 1. Cracking was generally of low to medium severity. Faulting was not evident at the transverse joints within and near the test section.

Two slab panels, denoted Slab A and Slab B, selected for instrumentation are also indicated in Fig. 3.



Crack Severity : VH = very high ; H= High ; M = Medium
without notation = low
F = faulting at transverse joint
S = strain gage location
D = deflectometer location
TJ = transverse joint

Fig. 2 Condition Survey for Section 1



Crack Severity : M= medium ; without Notation = Low

F= faulting at transverse joint
S= strain gage location
D= deflectometer location
TJ= transverse joint

Fig. 3 Condition Survey for Section 2

Test Section 3

No cracking or damage was visually evident at Section 3. Joint spacing at this location is 20 ft for the overlay and 40 ft for the existing pavement. There was no midslab cracking nor faulting at joints.

Test Section 4

Section 4 is a continuously reinforced concrete pavement. The condition survey for Section 4 is given in Fig. 4. Crack spacing within the length of pavement surveyed ranged from 1 ft to about 12 ft with most cracks spaced 5 ft or more. All cracks were tight.

Locations of instrumentation (strain gages and deflectometers) are also identified in Fig. 4.

Test Section 5

The condition survey for Section 5 is given in Fig. 5. Cracking was generally of low severity. Faulting was not evident at transverse joints within the length of pavement surveyed.

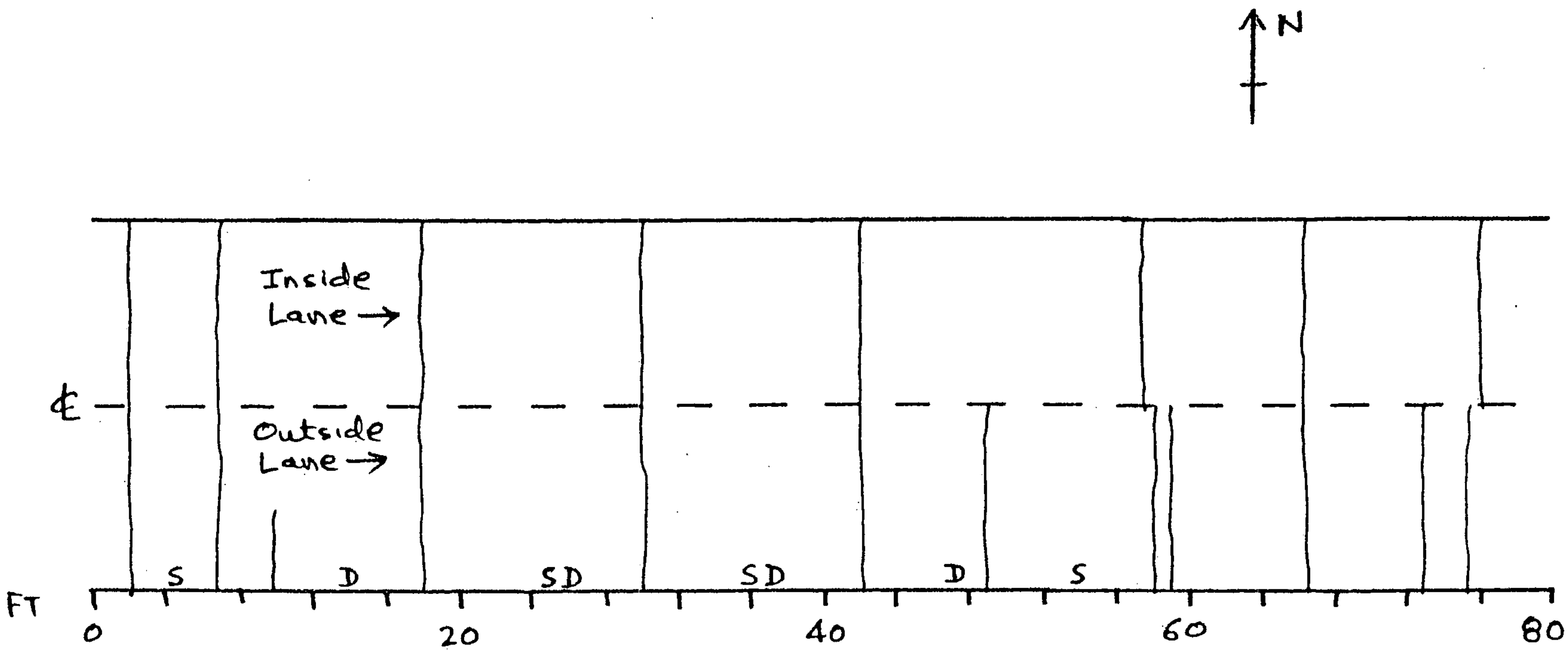
Two slab panels, denoted as Slab A and Slab B, selected for instrumentation are also indicated in Fig. 5.

CORE TESTING

The installation of deflectometers used to measure slab deflections required coring 4-1/4-in.-diameter holes along the pavement edge. The 4-in.-diameter cores recovered were used for compressive, split-tensile, and shear strength testing. Test results are summarized in the following.

Test Section 1

Four cores were obtained from Section 1. Two cores were tested for compressive strength and two cores were tested for split-tensile strength.

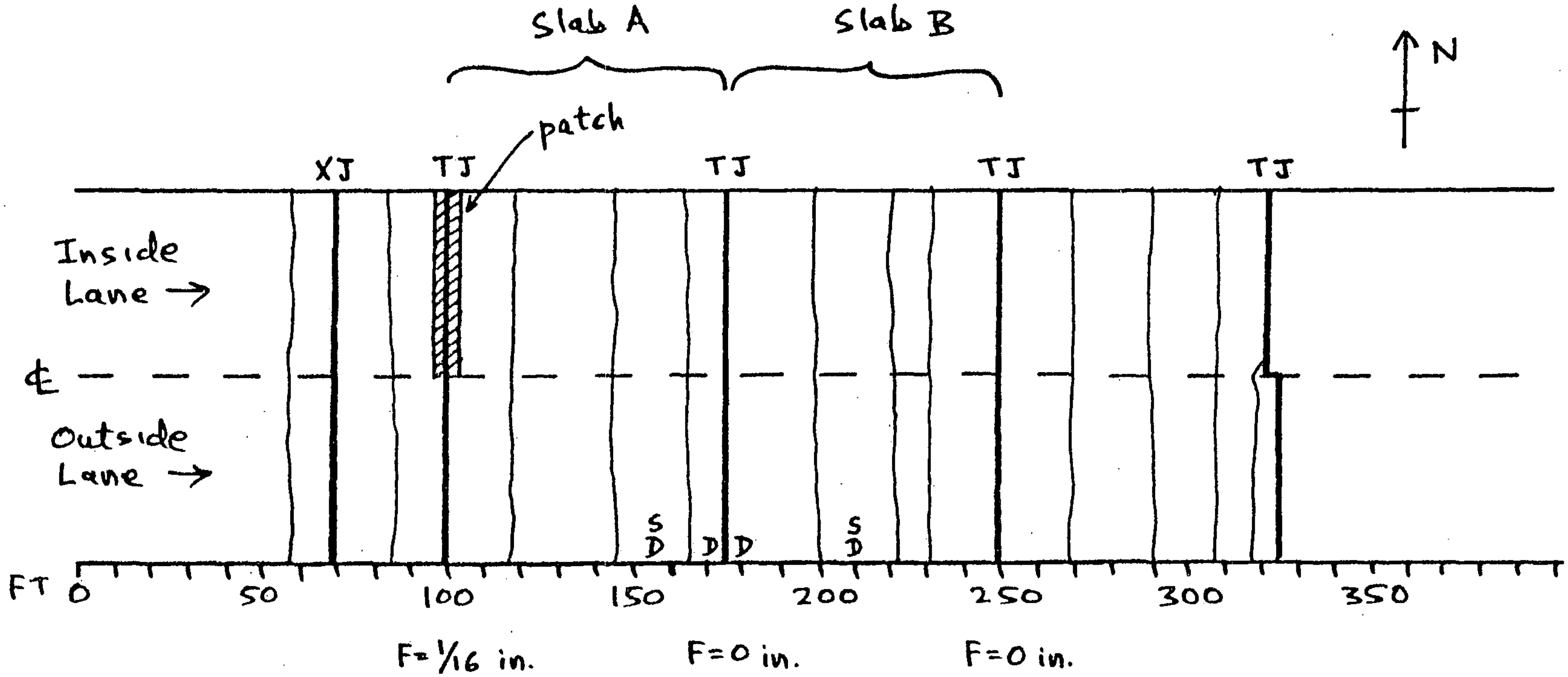


Cracking Severity: All cracks low severity – tight cracks
(Crack mapping accurate for outside lane)

S = strain gage location

D = deflectometer location

Fig. 4 Condition Survey for Section 4



Crack Severity: All cracks low severity – tight cracks

F = faulting at transverse joints
S = strain gage location
D = deflectometer location
XJ = transverse expansion joint
TJ = transverse contraction joint

Fig. 5 Condition Survey for Section 5

Compressive strength	=	8,590 psi
Split-tensile strength	=	630 psi
Pavement thickness	=	10.5 psi

Test Section 2

Four cores were obtained from Section 2. Interface shear strength was tested for all four cores. Then, core portions were used to conduct one compressive strength and one split-tensile strength test of the original concrete plus four split-tensile strength tests of the overlay concrete.

Compressive strength of original concrete	=	8,160 psi
Split-tensile strength of original concrete	=	730 psi
Split-tensile strength of overlay concrete	=	660 psi
Interface shear strength	=	490 psi
Overlay thickness	=	4.3 in.
Existing pavement thickness	=	10 in.

Test Section 3

Four cores were obtained from Section 3. Interface shear strength was tested for all four cores. Then, core portions were used to conduct three compressive strength tests and one split-tensile strength test of the original concrete plus four split-tensile strength tests of the overlay concrete.

Compressive strength of original concrete	=	6,860 psi
Split-tensile strength of original concrete	=	680 psi
Split-tensile strength of overlay concrete	=	670 psi
Interface shear strength	=	550 psi
Overlay thickness	=	4.5 in.
Existing pavement thickness	=	6 in.

Test Section 4

Four cores were obtained from Section 4. Interface shear strength was tested for all four cores. Then, core portions were used to conduct three compressive strength tests and one split-tensile strength test of the existing concrete plus four split-tensile strength tests of the original concrete and four split-tensile strength tests of the overlay concrete.

Compressive strength of original concrete	=	6,920 psi
Split-tensile strength of original concrete	=	600 psi
Split-tensile strength of overlay concrete	=	730 psi
Interface shear strength	=	370 psi
Overlay thickness	=	4.3 in.
Existing pavement thickness	=	8 in.

Test Section 5

Three cores were obtained from Section 5. Interface shear strength was tested for all three cores. Then, core portions were used to conduct one compressive strength and one tensile strength test of original concrete plus three split-tensile strength tests of the overlay concrete.

Compressive strength of original concrete	=	6,770 psi
Split-tensile strength of original concrete	=	660 psi
Split-tensile strength of overlay concrete	=	780 psi
Interface shear strength	=	500 psi
Overlay thickness	=	4.0 in.
Existing pavement thickness	=	10 in.

Summary

Results of core tests indicate that strength of the original pavement concrete at all test sections was very high. For the original pavement core concrete compressive strength ranged from 6,770 to 8,590 psi and split-tensile strength ranged from 600 to 730 psi. For the overlay concrete, split-tensile strength ranged from 660 to 780 psi. The interface shear strength for the four sections with the bonded overlay ranged from 370 to 550 psi.

Assuming that the 28-day concrete compressive strength of concrete (at time of construction) was about 5,000 psi, test results indicate a compressive strength gain of about 35 to 72 percent in a period of about 20 years for the original concrete.

INSTRUMENTATION

All pavement test sections were instrumented to measure load induced strains and deflections at the pavement surface. In addition, pavement temperature and slab curl were monitored with respect to time. Curl is the change in the vertical profile of the slab resulting from a change in the slab temperature.

For test sections with jointed pavement, two adjacent slab panels were instrumented. Each slab panel was instrumented to obtain strains and deflections at midslab edge and deflection at a joint corner. For Section 4 with the continuously reinforced concrete pavement, several "cracked" segments of the pavement were instrumented to obtain four replicate readings of edge longitudinal and interior transverse strains and edge deflection.

Typical strain gage and deflectometer locations for the jointed pavements of Sections 1, 2, 3, and 5 are shown in Fig. 6. Exact locations of the gages and deflectometers for Section 1, 2, and 5 are shown in Figs. 2, 3, and 5, respectively. Instrumentation for Section 4 is shown in Fig. 7. The instrumentation plan was established to provide maximum values of strains and deflection due to edge loading.

A brief description of instrumentation procedures used at the test sections follows:

Load Strains

Load strains were measured with 4-in.-long electrical-resistant strain gages bonded to the pavement surface. All gages were placed in recessed grooves to protect them from direct application of wheel loads. The procedure for applying gages was:

1. Grind a recess sufficient to remove the texture grooves in the pavement surface
2. Heat the concrete surface, when necessary
3. Clean the recess with acetone

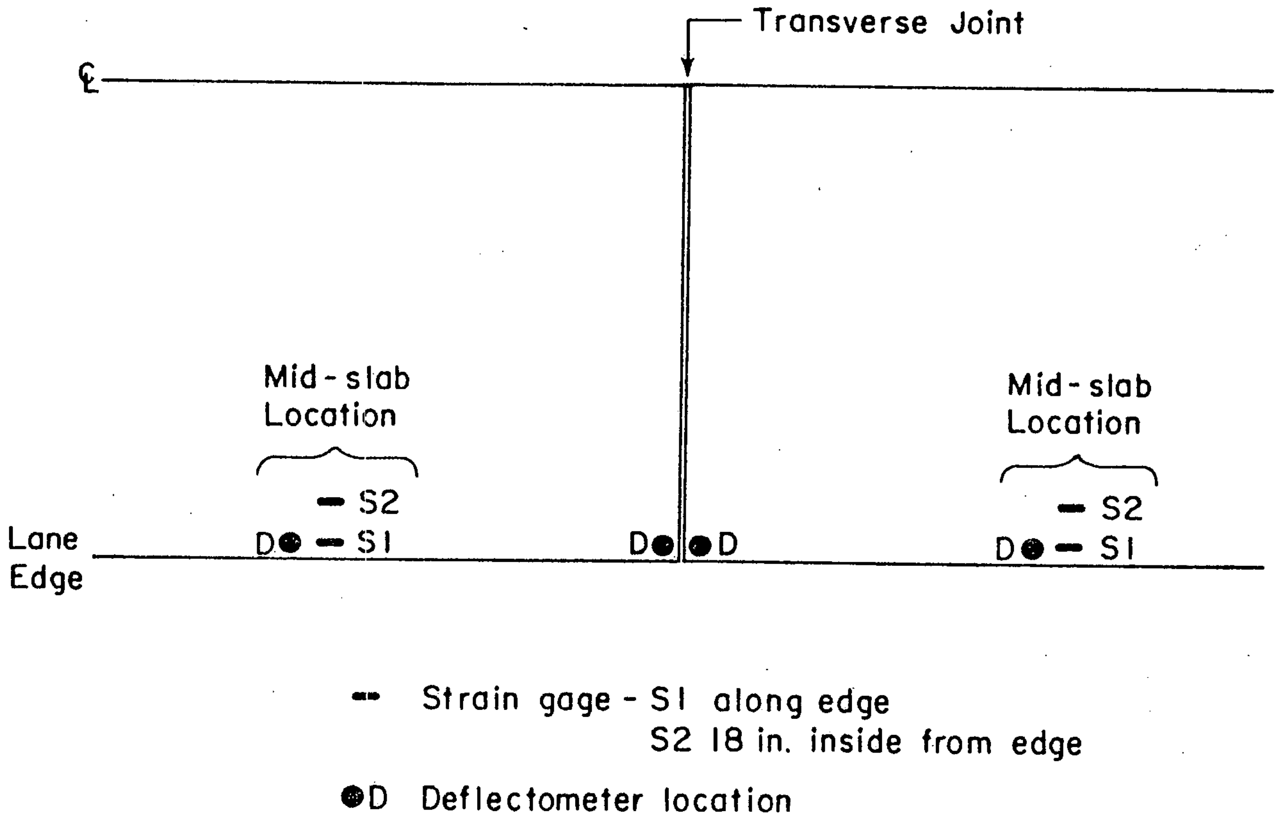
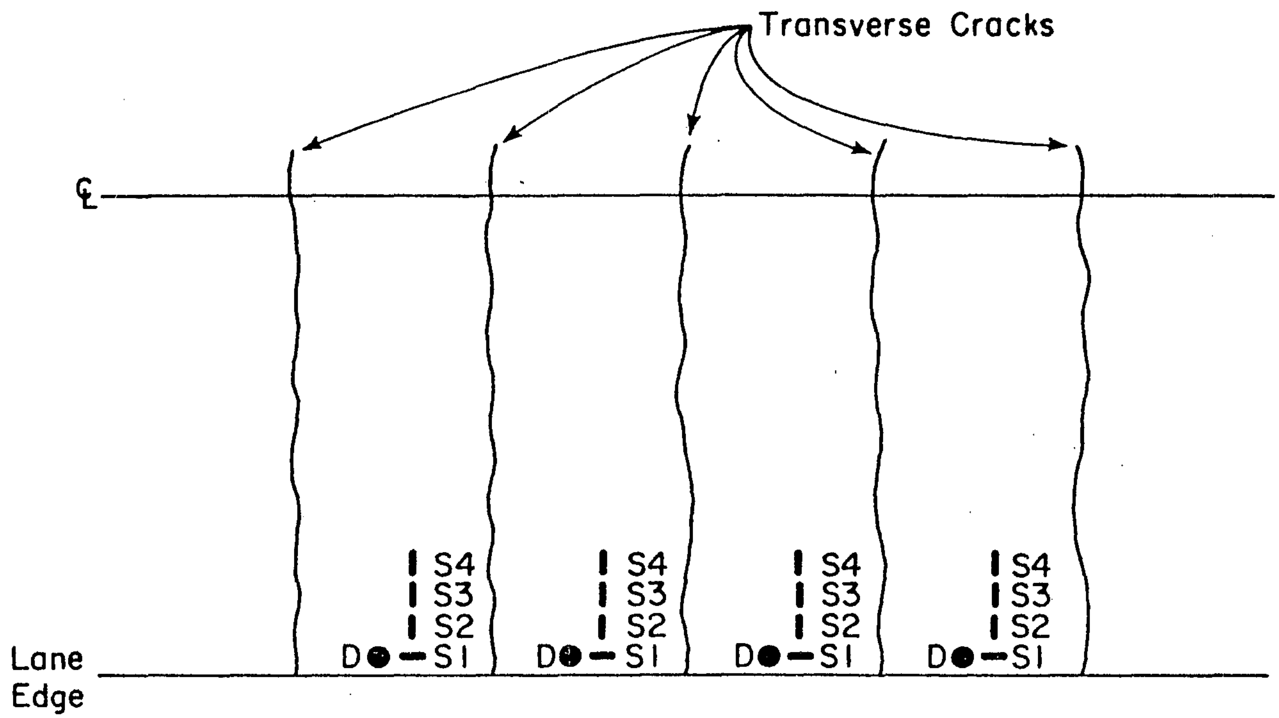


Fig. 6 Typical Instrumentation for Sections 1, 2, 3, and 5



▬ Strain gage - S1 along edge (longitudinal)
S2 6 in. from edge (transverse)
S3 18 in. from edge (transverse)
S4 26 in. from edge (transverse)

●D Deflectometer location

Fig. 7 Instrumentation for Section 4

4. Apply a thin coat of adhesive
5. Place the gage in the adhesive and remove all air bubbles
6. Connect lead wires to the gage
7. Run lead wires in recessed grooves to the pavement edge
8. Waterproof the gage
9. Fill gage and lead wire recesses with silicon rubber

Views of the installed gages are given in Fig. 8.

Load Deflections

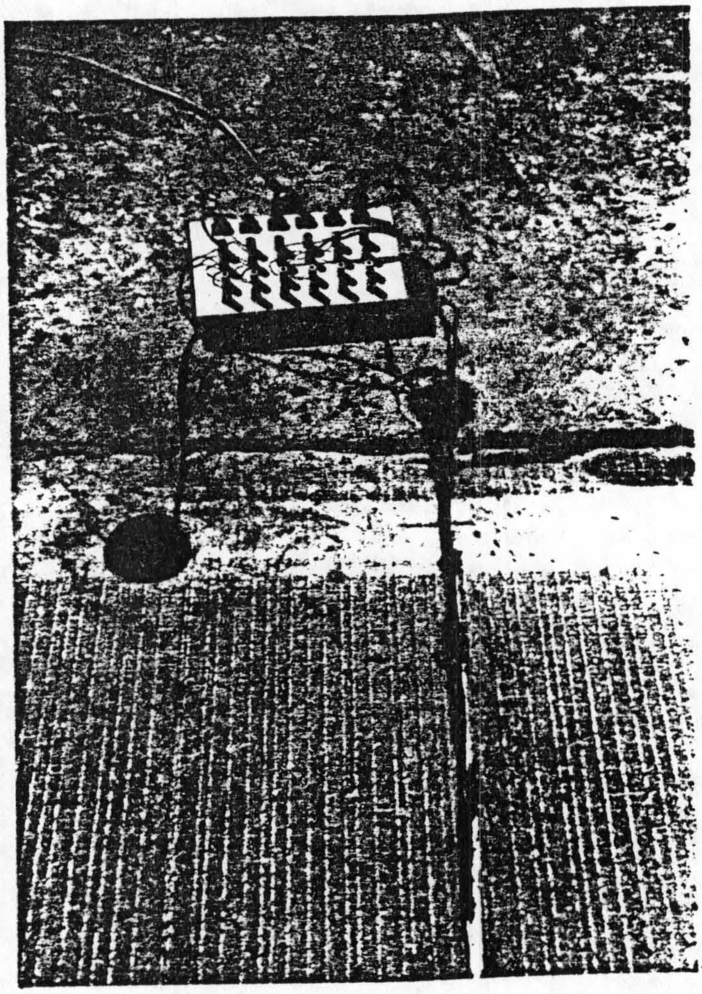
Load deflections were measured with resistance-bridge deflectometers mounted in core holes located near the pavement edge. Readings were referenced to encased rods driven in the subgrade to a depth of 6 ft. Views of the deflectometer installation are given in Fig. 9. The installation procedure used for the deflectometers allowed passage of the trucks directly over the deflectometer locations.

Curl Measurements

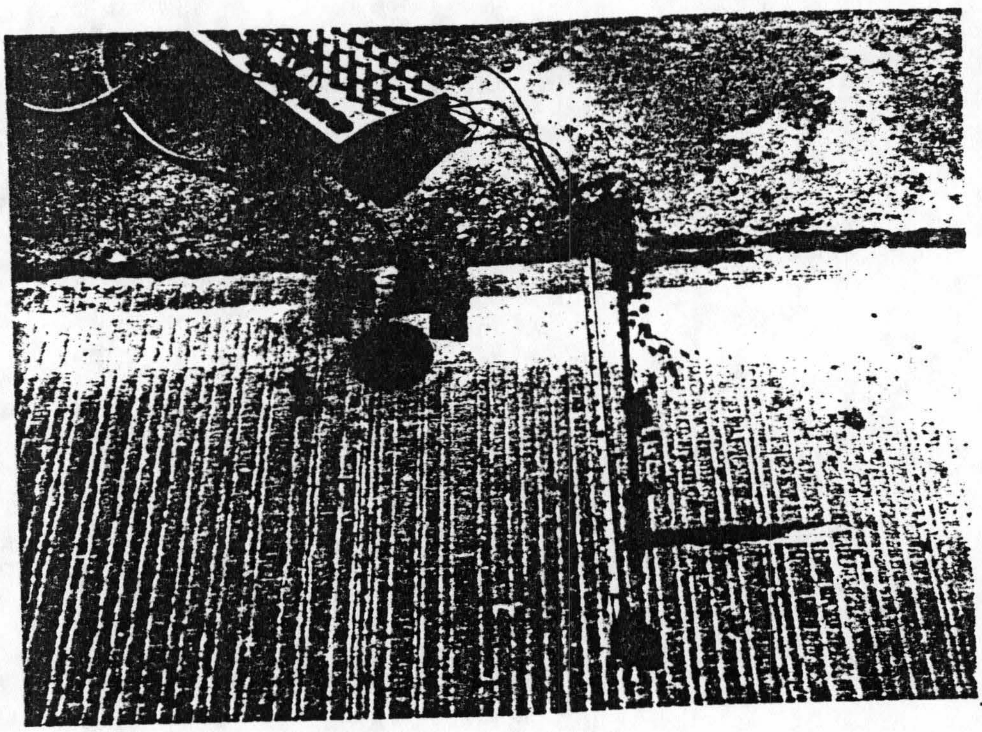
Pavement curl was measured with 0.001-in. indicators placed at the same locations as the deflectometers. Curl readings were referenced to the encased rods placed in the subgrade. Curl readings were taken approximately once an hour.

Temperature Measurements

Changes in pavement temperature were measured with copper-constantan thermocouples placed at the surface of the concrete pavement and at the bottom of the pavement in the core holes used for placing deflectometers. Air temperature was monitored with a thermocouple shaded from direct sun.

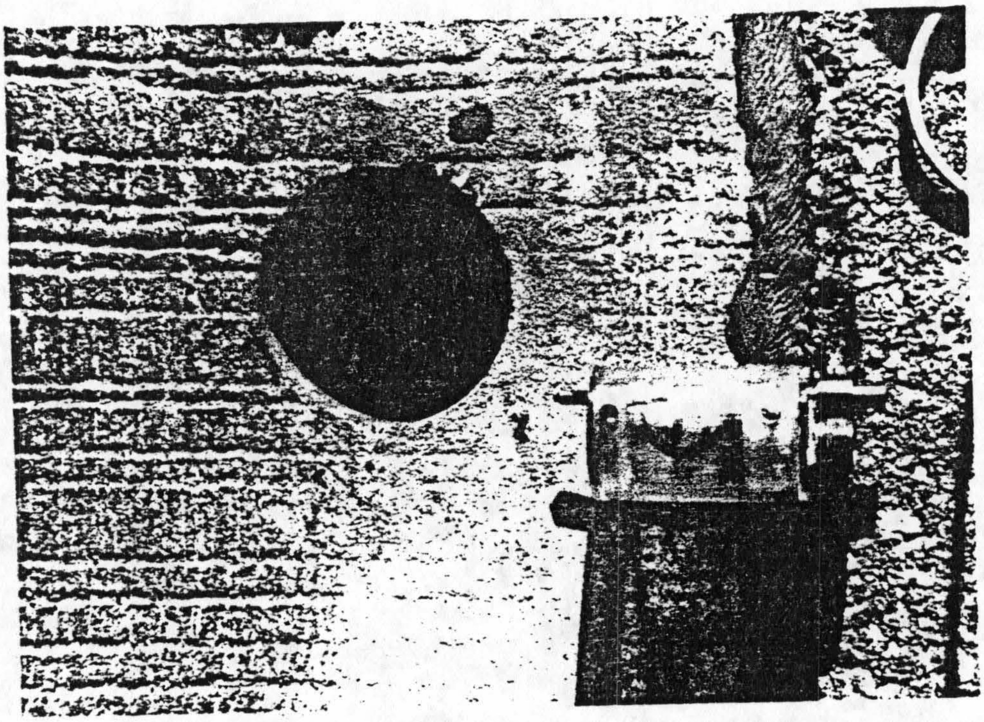


a) Strain Gages at Section 4

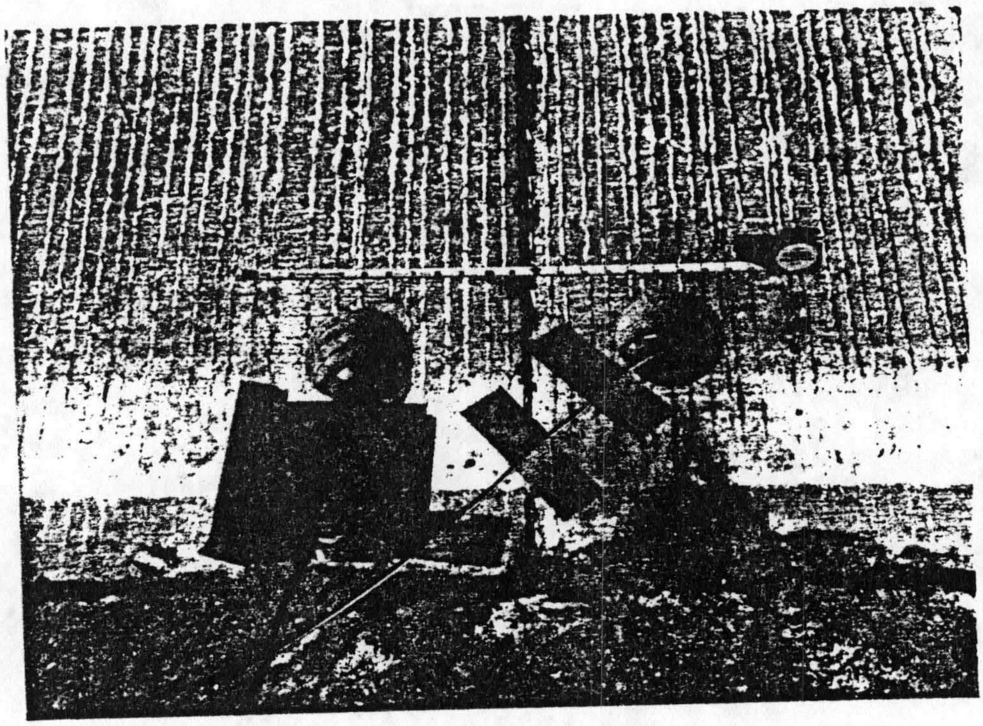


b) Strain Gages at Section 5

Fig. 8 Views of Installed Gages



a) Core Hole with Deflectometer Housing Located Inside Core Hole.



b) Installed Deflectometers at a Transverse Joint.

Fig. 9 Views of Deflectometer Installation.

Monitoring Equipment

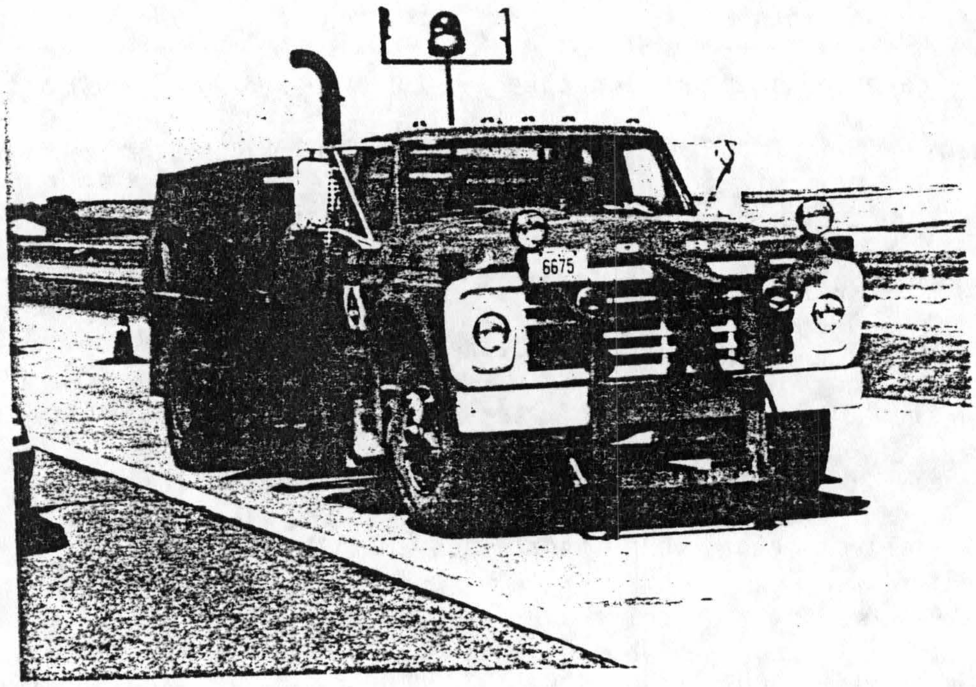
Data were monitored and recorded with equipment carried in Construction Technology Laboratories' field instrumentation van. Strain and deflection data were recorded with a high-speed computer-based data acquisition system. Up to twenty channels of instrumentation were monitored and recorded simultaneously for each vehicle loading. Computer programs were written to monitor, record, and tabulate all field data. All analog data from strain gages and deflectometers were digitized and stored on computer floppy disks. Readings from each item of instrumentation were digitized simultaneously at the rate of approximately 200 points per second. Detailed loading curves for each strain gage and deflectometer were stored on computer floppy disks for future examination.

All monitoring and recording instrumentation was calibrated prior to testing.

LOAD TESTING

Loading was applied using two trucks supplied by Iowa DOT. One truck was loaded to provide a 20 kip nominal single-axle load. The second truck was loaded to provided a 34-kip nominal tandem-axle load. The two trucks used are shown in Fig. 10. Characteristics of the trucks are given in Table 1.

It had been planned to use Iowa DOT's Model 400 Road Rater equipment in conjuction with CTL's load testing. This was planned to establish a correlation between Road Rater deflections and measured responses under the 20- kip SAL and 34-kip TAL. The Road Rater unit is an electronically controlled, hydraulically powered unit mounted in the rear of a van. A dynamic load is applied at a fixed frequency. The actual dynamic load applied is a function of displacement of the mass used to impart the loading. For rigid pavement,



a) 20-kip Single-Axle Load

b) 34-kip Tandem-Axle Load

Fig. 10 Load Trucks

TABLE 1 - CHARACTERISTICS OF TRUCKS

Item	Truck with	
	SAL	TAL
Iowa DOT Number	A-86675	A-22599
Rear-Axle Track, in. (c.c. of duals)	72	72
Tandem-Axle Base, in.	-	50
Dual Spacing, in.	13	13-1/4
Tire Pressures (Rear), psi	80-85	100
Gross Weight, lb	29,100	47,000
Front-Axle Load, lb	9,020	13,940
Rear-Axle(s) Load, lb	20,080	34,060

Iowa DOT uses peak-to-peak dynamic load of about 2,000 lb at a frequency of 30 cycles per second. The Road Rater has been used by Iowa DOT to determine AASHTO structural numbers for flexible pavements, to determine subgrade support values for rigid pavements, and to determine overlay requirements for both rigid and flexible pavements.[2,3]

The Road Rater unit was available only for testing at Sections 1, 2, and 3 at the end of CTL's field testing program.

Strains and deflections were recorded for the 20-kip single axle and 34-kip tandem-axle loadings with the trucks moving at creep speed. Two wheel paths were used. For one wheel path, tire placement was at 2 in. from the pavement edge. For the second wheel path, tire placement was at 18 in. from the pavement edge. The tire placement distance is the distance from the pavement edge to the outside edge of the outside tire sidewall. Care was taken to ensure that wheel paths of the trucks coincided with the desired paths painted on the pavement.

Sections 1 and 2 were tested on April 25, 1986, Section 3 was tested on April 26, 1986, and Sections 4 and 5 were tested on April 23, 1986. Each day, testing was generally started between 8:00 and 9:00 a.m. and testing was repeated several times until about 2:00 p.m. Specific testing times were governed by traffic control requirements and preparation times required at each test section.

DATA ANALYSIS

This section presents a summary of the field data and comparison of field data with results of theoretical analysis of bonded overlay sections. As stated previously, curl was measured at each deflectometer location generally between 8:00 a.m. and 2:00 p.m. Because of variations in slab curl with changes in temperature, measured deflections due to load along a slab edge or corner are affected by the time of testing. In addition, measured slab strains may also be affected by time of testing but at a lower level. Therefore, care has to be exercised in interpreting deflection and strain measurements if these measurements are made at different times of a day or on different days.

Curling and Warping Effects

Soon after concrete is placed, drying shrinkage of the concrete begins. Drying shrinkage in a slab-on-grade occurs at a faster rate at the slab surface than at the slab bottom. In addition, because the subgrade and subbase may remain wet, the slab bottom remains relatively moist. Thus, total shrinkage at the bottom is less than at the top. This differential in shrinkage results in a lifting of the slab from the subbase at edges and corner. Movements of this type resulting from moisture differentials are referred to as warping. Over a period of time, the warping behavior is modified by creep effects. However, warping is almost never recoverable.

In addition to warping, a slab-on-grade is also subjected to curling. Curling is the change in the slab profile due to temperature differential between slab top and bottom. Curling is a daily phenomenon. Slabs are curled upward from their warped shape during the night when temperatures are low and curled downward from their warped shape during the midday period when temperatures are higher.

The variation with time of pavement curl and deflections under load at slab edge and corner are shown in Figs. 11 to 15 for Sections 1 to 5, respectively. As shown in Figs. 11 and 15, corner curl was highest for

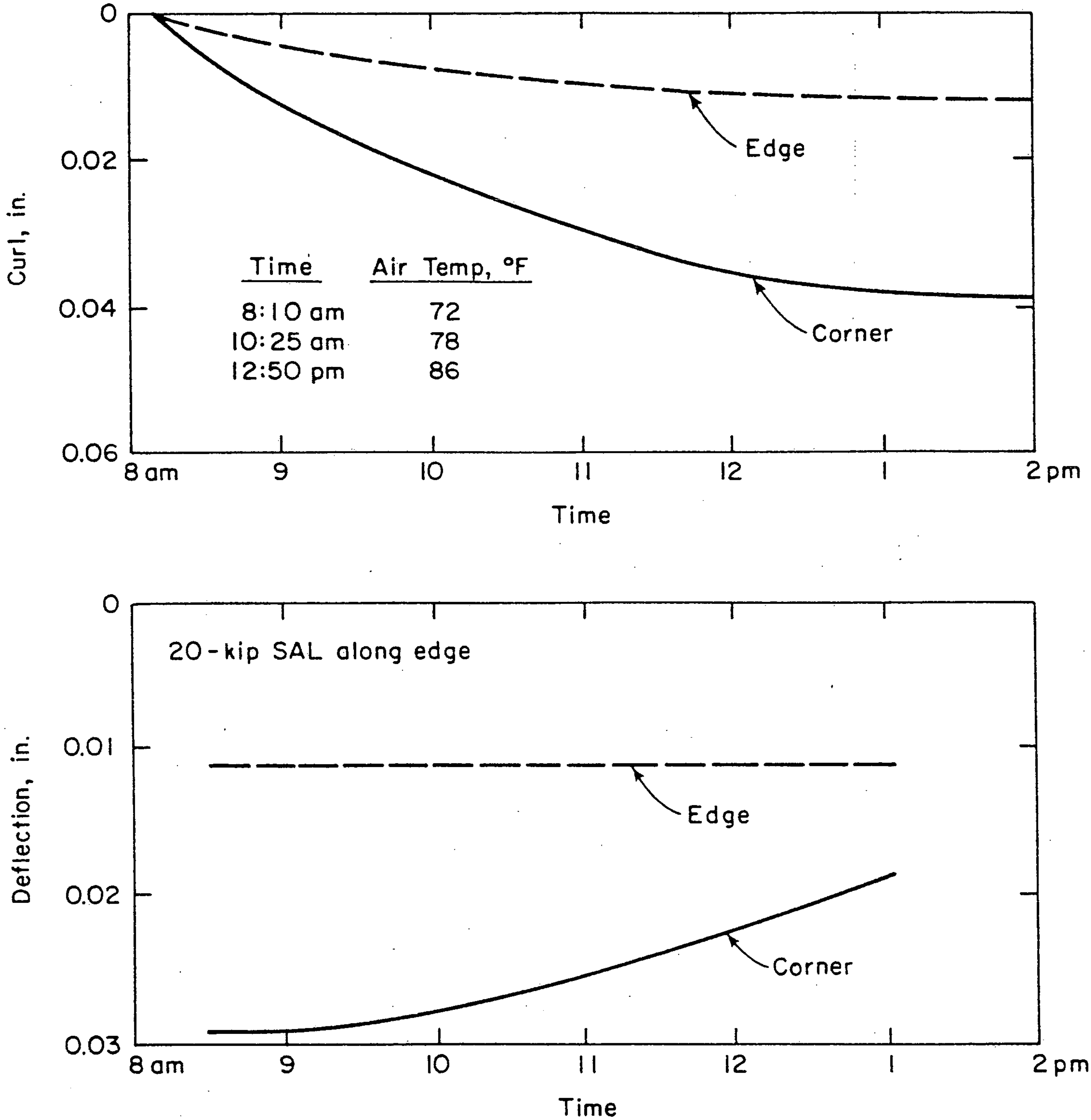


Fig. 11 Variation of Curl and Deflection with Time at Section 1

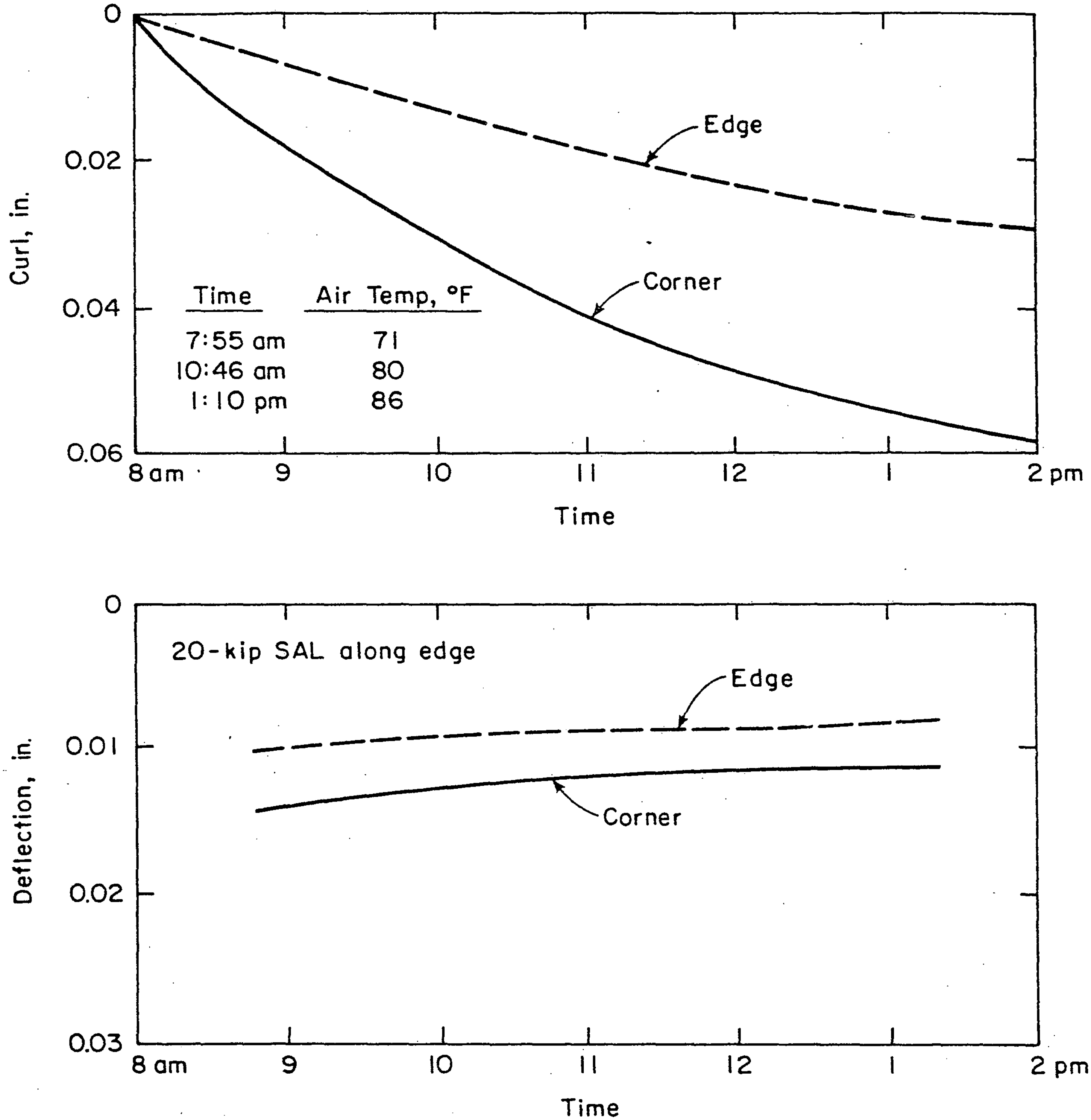


Fig. 12 Variation of Curl and Deflection with Time at Section 2

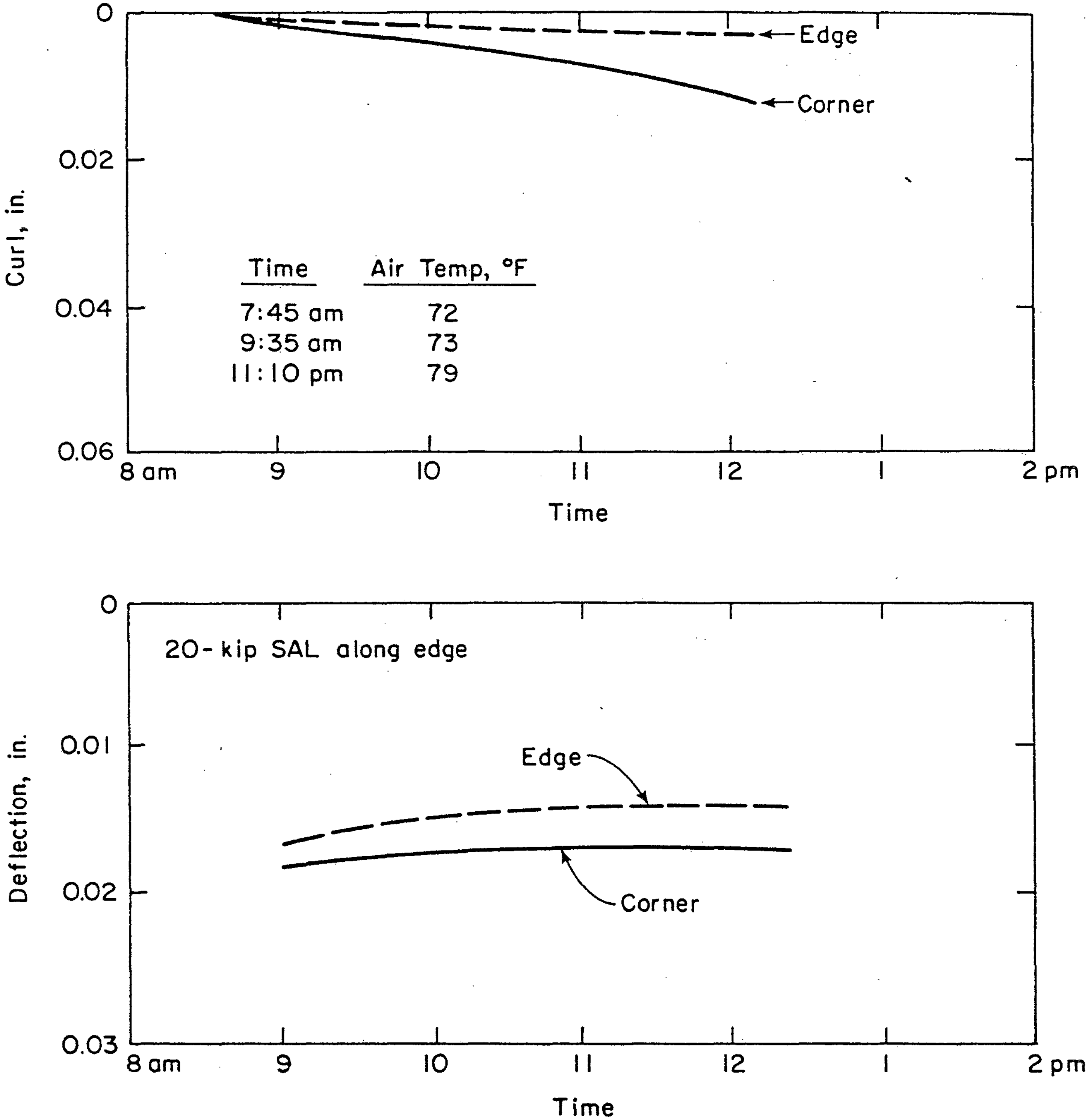


Fig. 13 Variation of Curl and Deflection with Time at Section 3

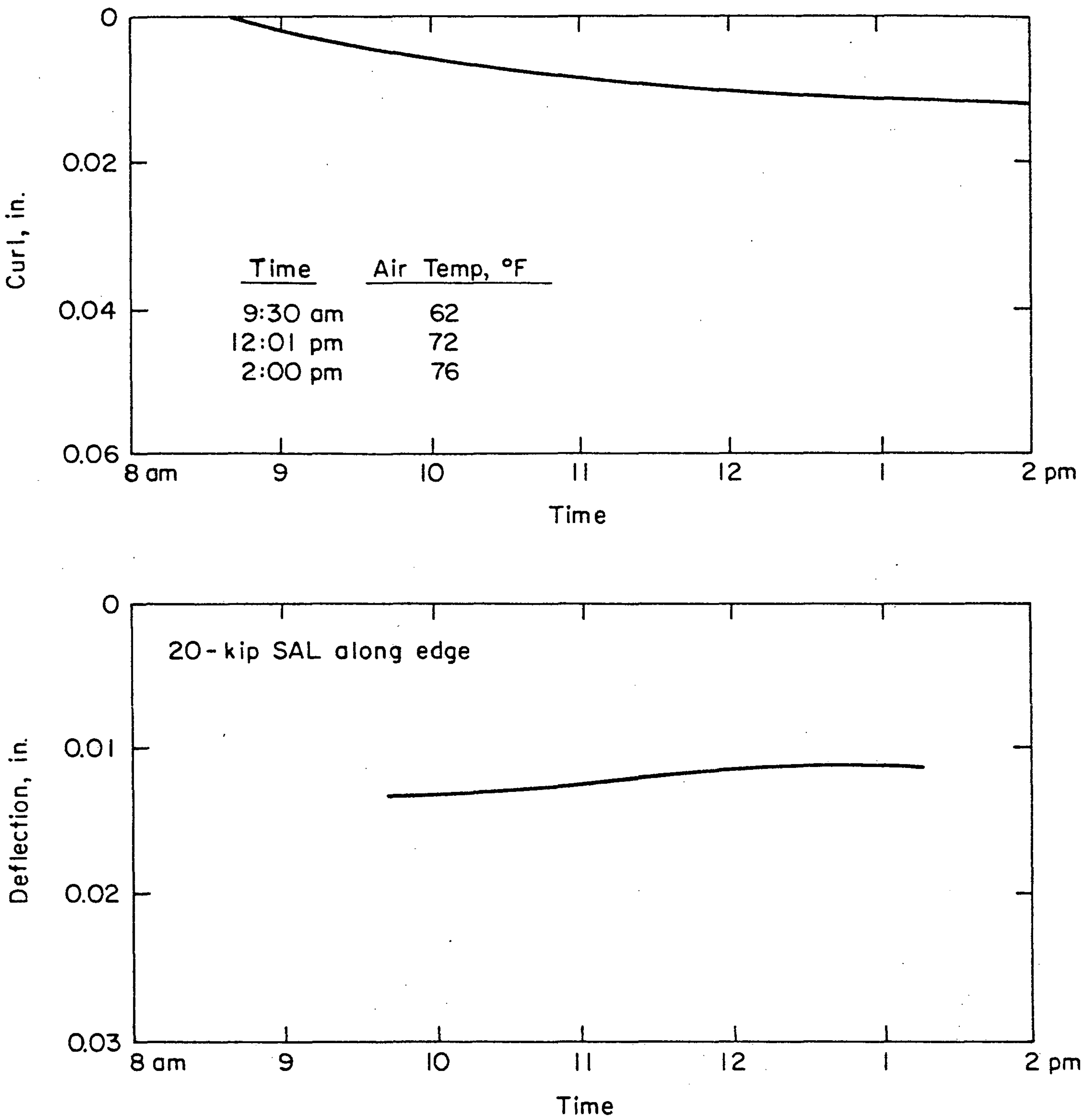


Fig. 14 Variation of Curl and Deflection with Time at Section 4

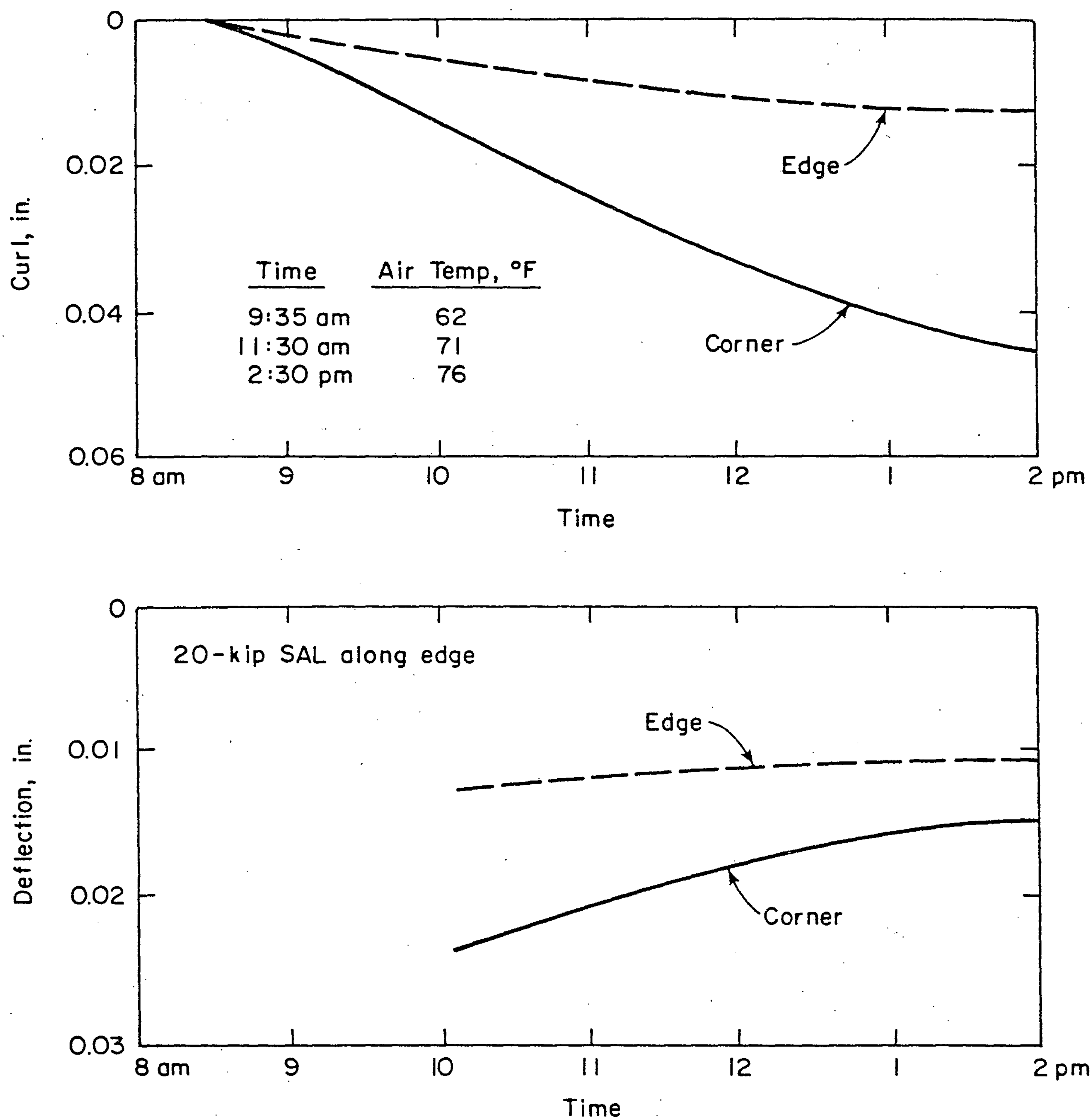


Fig. 15 Variation of Curl and Deflection with Time at Section 5

jointed pavement Sections 1, 2 and 5 with joint spacing of 76 ft-6 in. Edge curl at all sections was low and thus had almost no effect on deflections due to truck loading over a period of time.

Summary of Measured Strains and Deflections

Pavement responses (strains and deflections) measured at Sections 1 through 5 are listed in Tables 2 through 6, respectively. Responses listed are generally an average of two readings (from Slab A and Slab B) for Section 1, 2, 3, and 5. For Section 4, responses listed are generally an average of four readings.

The strains reported in the tables are those measured at the slab surface. It is assumed that strains at the slab bottom are equal in magnitude but opposite in sign. Thus, a reported value of 20 millionths compressive strain at the slab surface would imply a 20 millionths tensile strain at the slab bottom.

Typical graphical recordings of edge strain at Section 2 are shown in Fig. 16 for 20-kip single-axle and 34-kip tandem-axle loadings. Similar graphical recordings for edge deflection at Section 2 are shown in Fig. 17 and for joint deflection at Section 2 are shown in Fig. 18.

A summary of the measured responses is presented for all test sections in Table 7 for the 20-kip SAL and 34-kip TAL along the lane edge.

It is seen from Table 7 that measured responses were much lower at Section 2 with a total nominal slab thickness of 14 in. compared to responses at Section 1 with a total nominal slab thickness of 10 in. Measured strain values at Section 2 were less than half of those at Section 1. Measured, deflection values at Section 2 were also much lower indicating the beneficial effects of the 4-in.-thick (nominal) overlay at Section 2.

TABLE 2 - MEASURED RESPONSES AT SECTION 1

Response Type	Axle Load	8:30 a.m.	9:30 a.m.	10:30 a.m.	11:30 a.m.	1:00 p.m.
		Test Time				
WHEEL PATH: 2 in. from edge						
Edge Strain	SAL	30	27	27	26	26
	TAL	16	19	19	19	20
Long. Strain @ 18 in.	SAL	29	32	30	28	28
	TAL	14	15	15	15	18
Edge Deflection, in.	SAL	0.012	0.013	0.012	0.012	0.012
	TAL	0.020	0.018	0.017	0.015	0.015
Corner Deflection, in.	SAL	0.029	0.029	0.026	0.024	0.018
	TAL	0.033	0.028	0.026	0.024	0.022
WHEEL PATH: 18 in. from edge						
Edge Strain	SAL	16	16	14	13	13
	TAL	11	10	9	10	10
Long. Strain @ 18 in.	SAL	16	16	13	13	16
	TAL	10	8	9	8	10
Edge Deflection, in.	SAL	0.008	0.009	0.009	0.008	0.008
	TAL	0.014	0.012	0.012	0.011	0.011
Corner Deflection, in.	SAL	0.021	0.019	0.019	0.017	0.013
	TAL	0.023	0.021	0.020	0.019	0.017

NOTES: 1. SAL = 20-kip single-axle load
TAL = 34-kip tandem-axle load
2. For TAL, strain values listed are the larger of the two peak values under the two axles
3. Strain readings are in millionths.

TABLE 3 – MEASURED RESPONSES AT SECTION 2

Response Type	Axle Load	Test Time 9:00 a.m.	9:50 a.m.	11:00 a.m.	11:50 a.m.	1:20 p.m.
WHEEL PATH: 2 in. from edge						
Edge Strain	SAL	10	11	11	12	13
	TAL	11	11	10	9	12
Long. Strain @ 18 in.	SAL	12	13	14	14	13
	TAL	15	13	13	14	15
Edge Deflection, in.	SAL	0.010	0.009	0.009	0.009	0.008
	TAL	0.015	0.014	0.013	0.013	0.012
Corner Deflection, in.	SAL	0.014	0.013	0.012	0.012	0.011
	TAL	0.018	0.016	0.015	0.014	0.014
WHEEL PATH: 18 in. from edge						
Edge Strain	SAL	7	5	5	5	6
	TAL	5	5	6	6	6
Long. Strain @ 18 in.	SAL	9	10	9	10	9
	TAL	7	8	9	8	9
Edge Deflection, in.	SAL	0.007	0.007	0.007	0.007	0.006
	TAL	0.011	0.010	0.010	0.010	0.009
Corner Deflection, in.	SAL	0.010	0.009	0.009	0.008	0.008
	TAL	0.014	0.012	0.011	0.011	0.010

NOTES:
1. SAL = 20-kip single-axle load
   TAL = 34-kip tandem-axle load
2. For TAL, strain values listed are the larger of the two peak values under the two axles.
3. Strain readings are in millionths

TABLE 4 - MEASURED RESPONSES AT SECTION 3

Response Type	Axle Load	8:00 a.m.	8:50 a.m.	Test Time 9:55 a.m.	10:35 a.m.	11:20 a.m.
WHEEL PATH: 2 in. from edge						
Edge Strain	SAL	42	38	37	35	34
	TAL	33	32	29	27	28
Long. Strain @ 18 in.	SAL	34	33	31	30	28
	TAL	26	28	25	26	26
Edge Deflection, in.	SAL	0.016	0.015	0.014	0.014	0.014
	TAL	0.020	0.022	0.022	0.021	0.021
Corner Deflection, in.	SAL	0.018	0.017	0.017	0.017	0.017
	TAL	0.024	0.024	0.024	0.024	0.023
WHEEL PATH: 18 in. from edge						
Edge Strain	SAL	23	23	20	20	20
	TAL	22	21	21	20	20
Long. Strain @ 18 in.	SAL	22	24	24	22	22
	TAL	22	20	20	20	20
Edge Deflection, in.	SAL	0.010	0.011	0.010	0.010	0.010
	TAL	0.017	0.016	0.016	0.016	0.016
Corner Deflection, in.	SAL	0.012	0.012	0.012	0.012	0.012
	TAL	0.018	0.018	0.017	0.017	0.017

NOTES:
1. SAL = 20-kip single-axle load
   TAL = 34-kip tandem-axle load
2. For TAL, strain values listed are the larger of the two peak values under the two axles.
3. Strain readings are in millionths

TABLE 5 - MEASURED RESPONSES AT SECTION 4

Response Type	Axle Load	Test Time 9:45 a.m.	11:15 a.m.	1:15 p.m.	2:15 p.m.
WHEEL PATH: 2 in. from edge					
Edge Strain	SAL	30	29	26	27
	TAL	21	20	23	20
Trans. Strain (at 18 in.)	SAL	-9	—	-15	-16
	TAL	-12	-23	-23	-22
Trans. Strain (at 26 in.)	SAL	-13	—	25	-24
	TAL	-13	-18	-22	-23
Edge Deflection, in.	SAL	0.013	0.012	0.011	0.011
	TAL	0.016	0.016	0.015	0.015
WHEEL PATH: 18 in. from edge					
Edge Strain	SAL	16	14	16	15
	TAL	14	14	15	15
Trans. Strain (at 18 in.)	SAL	-2	-6	-6	-7
	TAL	-5	-10	-9	-8
Trans. Strain (at 26 in.)	SAL	-7	-9	-9	-7
	TAL	-9	-12	-15	-11
Edge Deflection, in.	SAL	0.008	0.008	0.008	0.008
	TAL	0.013	0.012	0.012	0.011

NOTES: 1. SAL = 20-kip single-axle load
TAL = 34-kip tandem-axle load
2. For TAL, strain values listed are the larger of the two peak values under the two axles.
3. Negative value of strain indicates tensile strain at slab surface.
4. Strain readings are in millionths

TABLE 6 – MEASURED RESPONSES AT SECTION 5

Response Type	Axle Load	Test Time 10:15 a.m.	11:40 a.m.	1:35 p.m.	2:40 p.m.
WHEEL PATH: 2 in. from edge					
Edge Strain	SAL	22	22	21	23
	TAL	19	21	20	20
Long. Strain @ 18 in.	SAL	12	11	10	10
	TAL	10	9	10	12
Edge Deflection, in.	SAL	0.012	0.011	0.011	0.010
	TAL	0.017	0.016	0.015	0.015
Corner Deflection, in.	SAL	0.023	0.018	0.016	0.014
	TAL	0.024	0.019	0.017	0.017
WHEEL PATH: 18 in. from edge					
Edge Strain	SAL	16	12	13	15
	TAL	14	14	11	11
Long. Strain @ 18 in.	SAL	15	14	16	16
	TAL	12	14	12	14
Edge Deflection, in.	SAL	0.009	0.008	0.007	0.007
	TAL	0.012	0.012	0.012	0.012
Corner Deflection, in.	SAL	0.015	0.011	0.010	0.010
	TAL	0.019	0.014	0.012	0.012

NOTES: 1. SAL = 20-kip single-axle load
TAL = 34-kip tandem-axle load
2. For TAL, strain values listed are the larger of the two peak values under the two axles.
3. Strain readings are in millionths

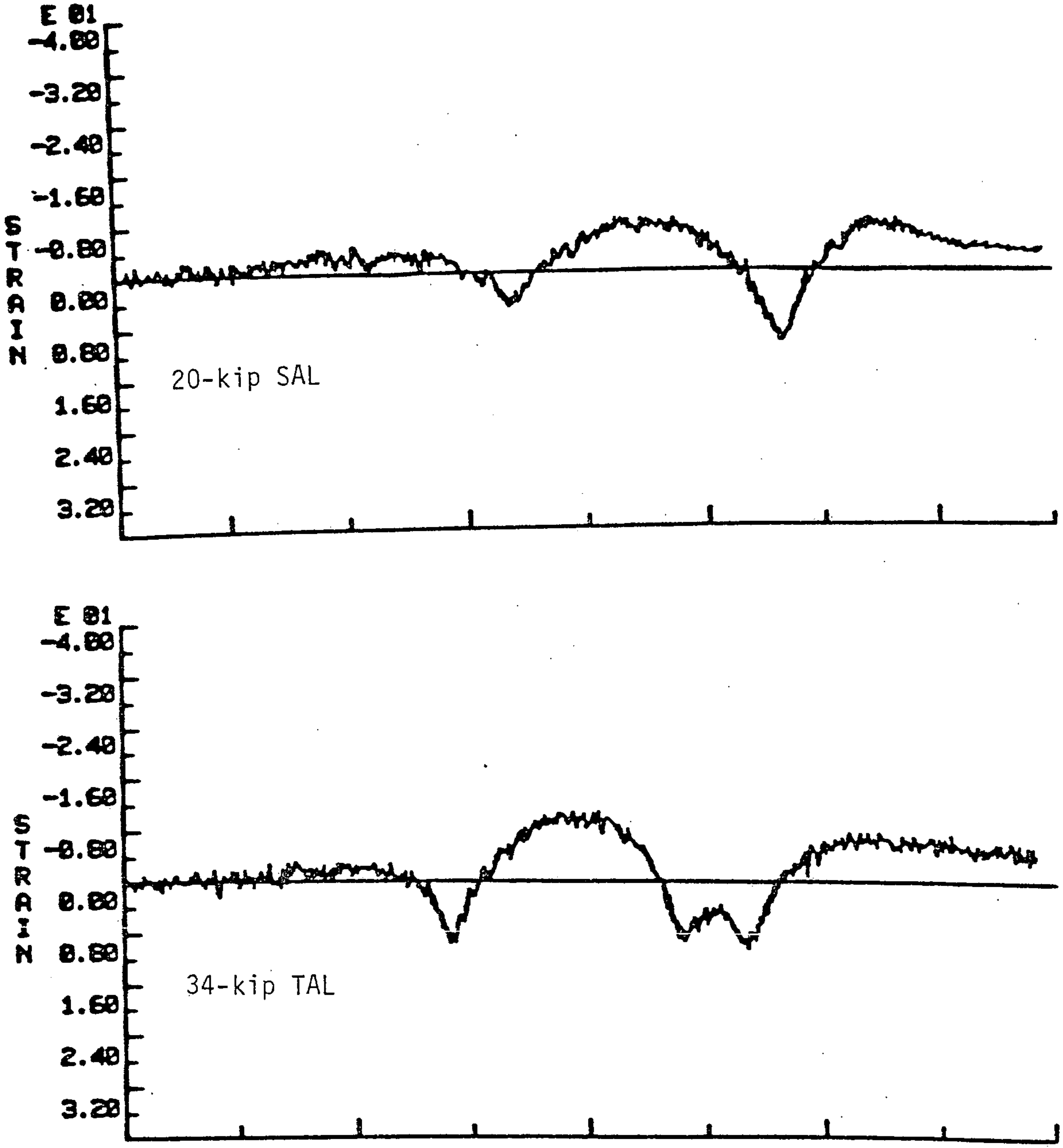


Fig. 16 Typical Recordings for Edge Strain

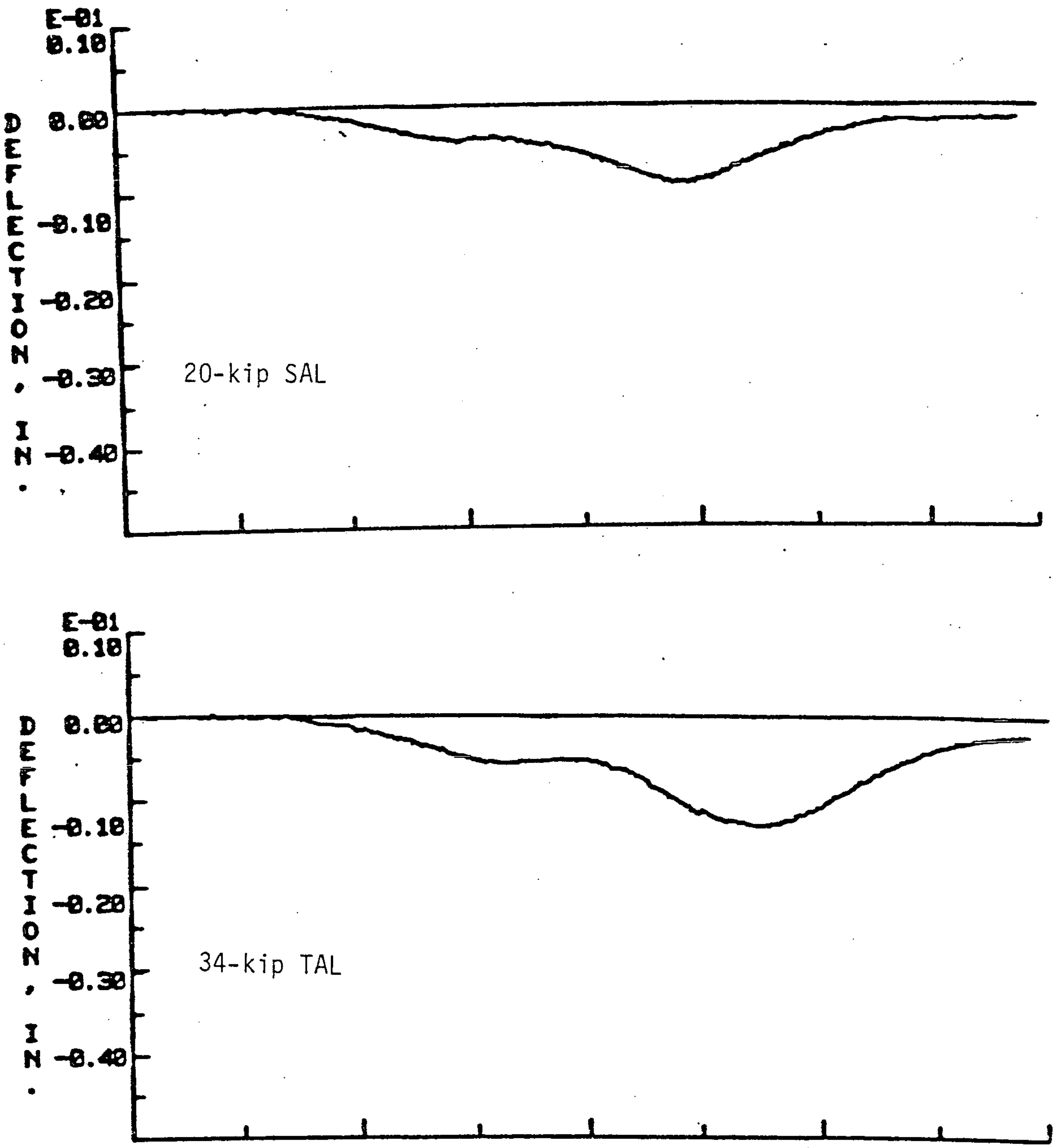


Fig. 17 Typical Recordings for Edge Deflection

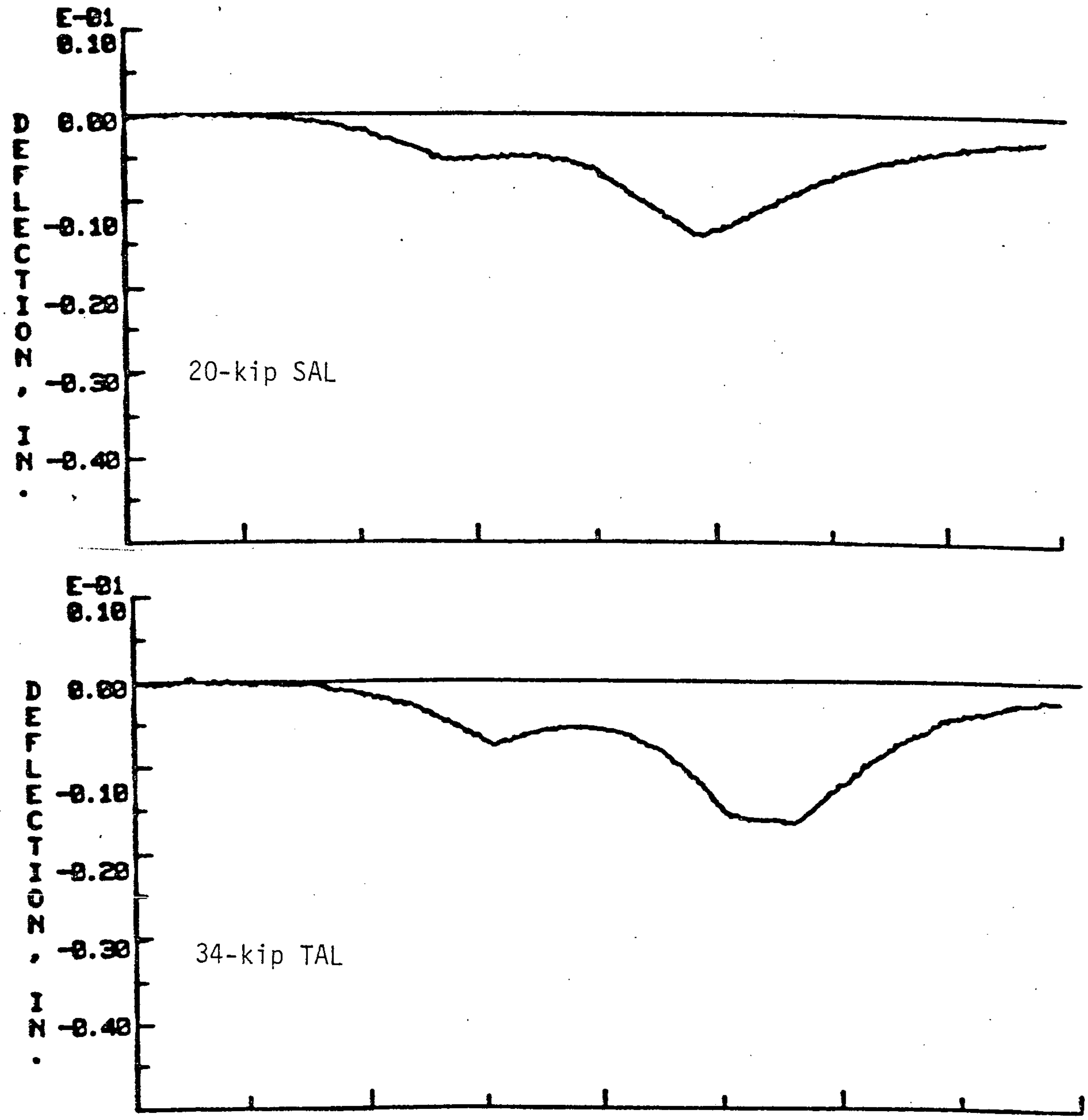


Fig. 18 Typical Recordings for Corner Deflection

TABLE 7 – SUMMARY OF MEASURED RESPONSES

Response Type	Test Section 1	2	3	4	5
20-kip SAL at edge					
Edge Strain	26–30	10–13	34–42	26–30	21–23
Long. Strain (@ 18 in.)	28–32	12–14	28–34	—	10–12
Trans. Strain (@ 18 in.)	—	—	—	(-9)-(16)	—
Trans. Strain (@ 26 in.)	—	—	—	(-13)-(-25)	—
Edge Deflection, in.	0.012–0.013	0.008–0.010	0.014–0.016	0.011–0.013	0.010–0.012
Corner Deflection, in.	0.018–0.029	0.012–0.015	0.017–0.018	—	0.014–0.023
34-kip TAL at edge					
Edge Strain	16–20	9–12	28–33	20–23	19–20
Long. Strain (@ 18 in.)	14–18	13–15	25–28	—	9–12
Trans. Strain (@ 18 in.)	—	—	—	(-12)-(-23)	—
Trans. Strain (@ 26 in.)	—	—	—	(-13)-(-23)	—
Edge Deflection, in.	0.015–0.020	0.012–0.015	0.020–0.022	0.015–0.016	0.015–0.017
Corner Deflection, in.	0.022–0.033	0.014–0.018	0.023–0.024	—	0.017–0.024
Total Slab Thickness, in. (nominal/actual)	10.5/10.5	14.0/14.3	10.0/10.5	11.0/12.3	13.0/14.0

NOTES: 1. Negative value of strain indicates a tensile strain at slab surface.
2. Ranges of values given for different times of testing.
3. Strain readings are in millionths.

Measured responses at Section 4 (overlaid CRCP) were a little larger than at Section 5 (overlaid JRCP). This difference is accounted for by the larger thickness of the existing pavement at Section 5. Section 3 had generally the highest measured responses with strains under a 20-kip SAL ranging from 34 to 42 millionths. Edge deflections under the 20-kip SAL at Section 3 ranged from 0.014 to 0.016 in.

Corner deflections at the sections with jointed pavement generally were about 30 to 60 percent greater than edge deflections.

For Section 4 (overlaid CRCP), the magnitudes of tensile transverse strains measured at the slab surface at 26 in. inward from the edge were almost equal to edge longitudinal strains.

It should be noted that the Road Rater unit was used at Sections 1, 2, and 3. At Section 2, the Road Rater was used at midslab, and at 2 in. and 18 in. from the edge. At Section 3, the Road Rater was used at midslab, at 7 in. from the edge, and at a joint location at 9 in. inside from the edge. Because the Road Rater was not placed directly over CTL's instrumentation and because deflections measured by the Road Rater are generally of low magnitude (about 0.001 to 0.002 in.), CTL's data acquisition system was not able to provide usable data for the case of the Road Rater loadings.

Analysis of Results

A comparison was made between measured responses and calculated theoretical responses. Pavement responses (edge stresses and edge deflections) were calculated using a finite element computer program, Program JSLAB. Program JSLAB, developed by the Construction Technology Laboratories for the Federal Highway Administration, can analyze jointed slabs.(4) Load input is in terms of wheel loads at any location on the slabs. Loss of support, variable support or material properties, as well as bonded and unbonded concrete overlays can be considered. In the program, the subbase/subgrade support is characterized by the modulus of subgrade reaction.

Analysis was conducted for various thicknesses of pavement slabs subjected to 20-kip SAL and 34-kip TAL at the midslab pavement edge. Analysis was conducted for a single slab 12 ft wide and 20 ft long. Values of modulus of subgrade reaction used were 100, 300, and 500 pci. The overlaid sections were assumed to behave monolithically.

The measured strains were converted to stresses by assuming that the modulus of elasticity of concrete was 5,000,000 psi. Use of this value of the modulus of elasticity is justified considering the high compressive and split-tensile strengths of the concrete at the test sections. In addition, it is assumed that the overlaid pavements at Sections 2 to 5 behave monolithically as evidenced by the high interface shear strengths between the overlay and the existing pavement.

The measured and computed edge stresses and deflections are compared in Fig. 19 for the 20-kip SAL and in Fig. 20 for the 34-kip TAL. It is seen that the <u>measured</u> stresses as well as deflections are a function of the <u>total</u> pavement thickness. Measured deflections are lower for larger total pavement thickness.

The modulus of subgrade reaction, k, values at the five test sections were reported to be about 200 pci. It is seen that the measured edge deflections correspond well with computed edge deflections at a k value of about 200 pci for both the SAL and the TAL. Measured edge stresses also correspond well with computed edge stress except for Sections 1 and 2. Measured edge stresses at Sections 1 and 2 are much lower than would have been anticipated, especially considering reasonably good agreement between measured and computed

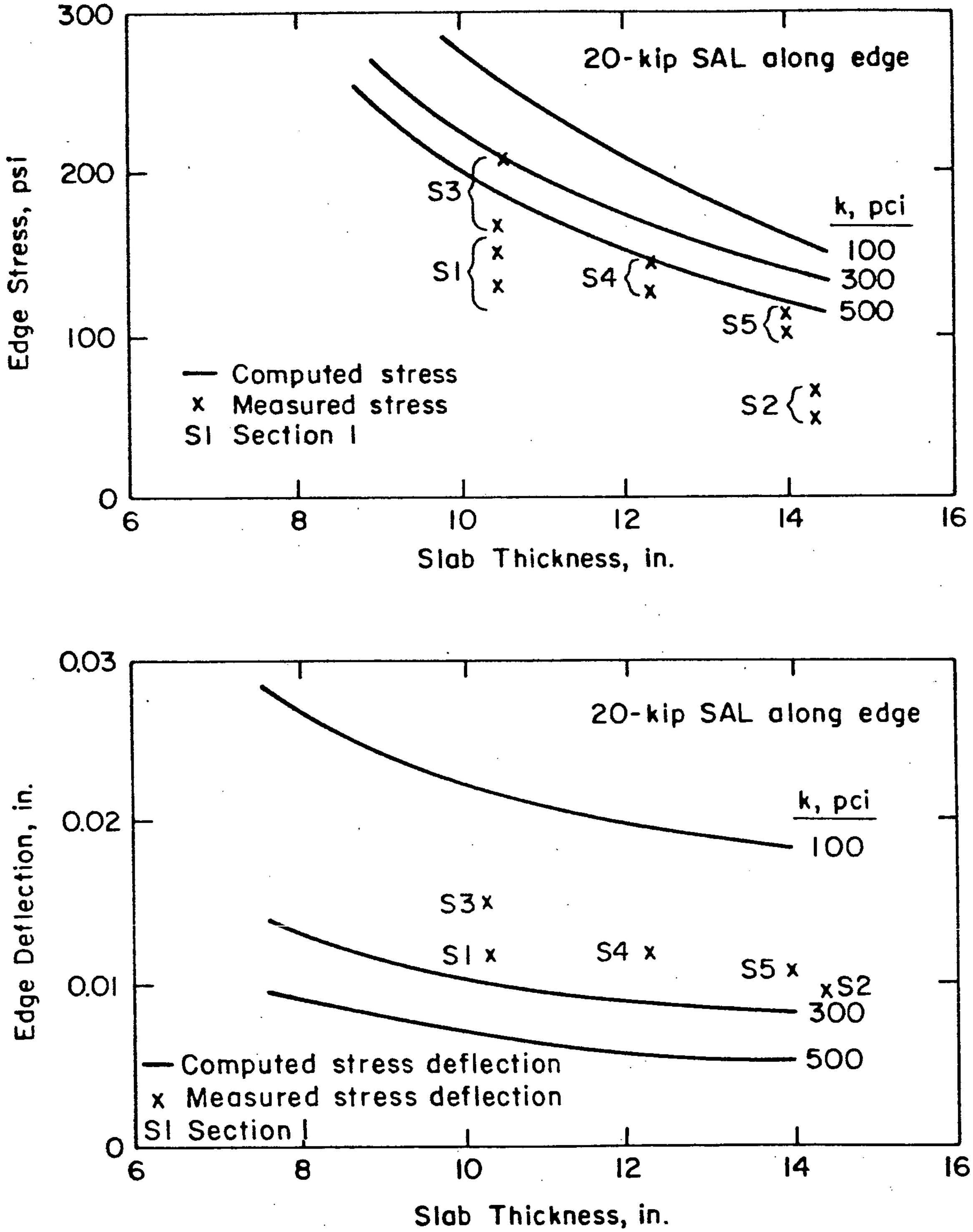


Fig. 19 Comparison of Measured and Calculated Responses for the 20-kip and Single-Axle Loading

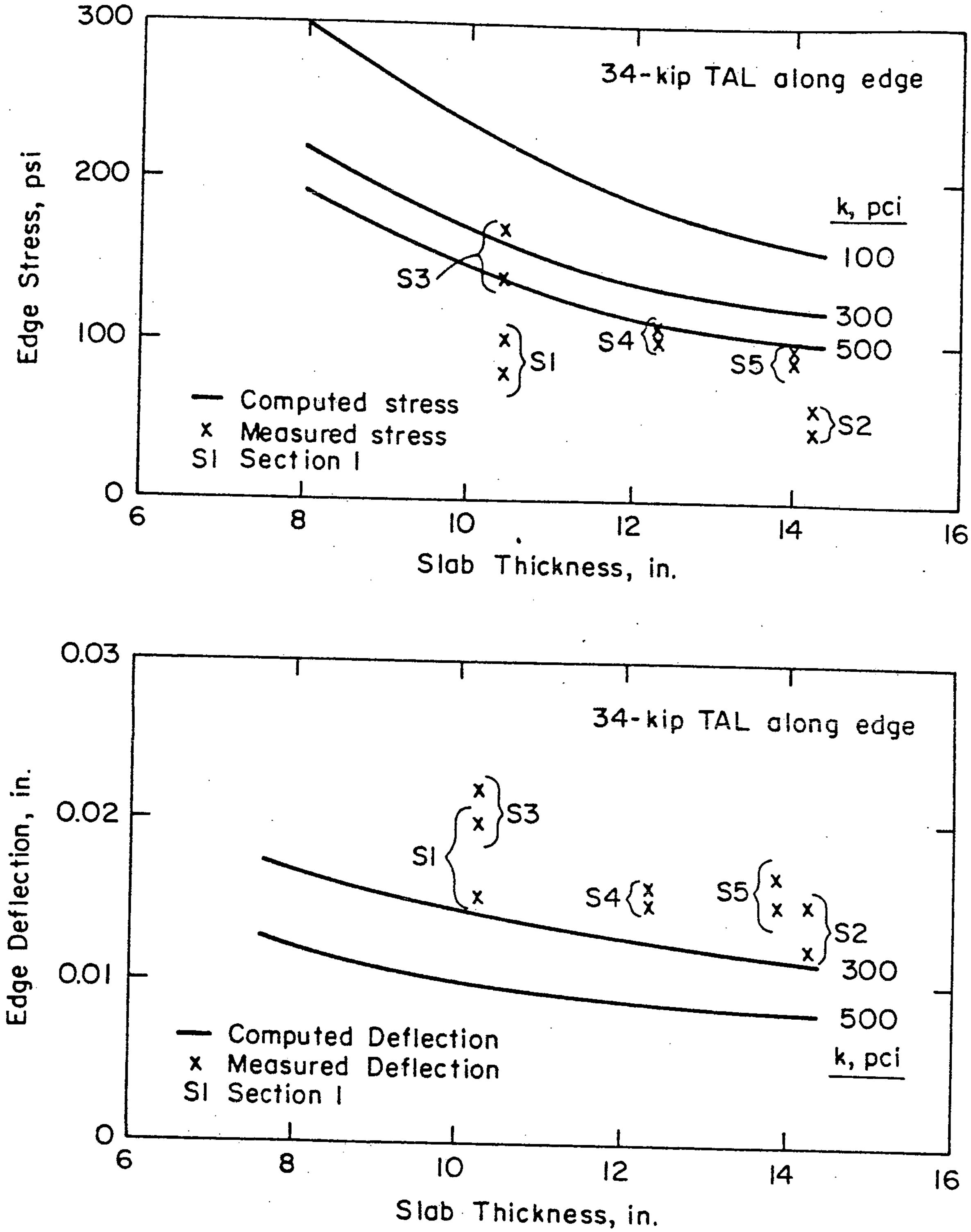


Fig. 20 Comparison of Measured and Calculated Responses for the 34-kip Tandem-Axle Loading

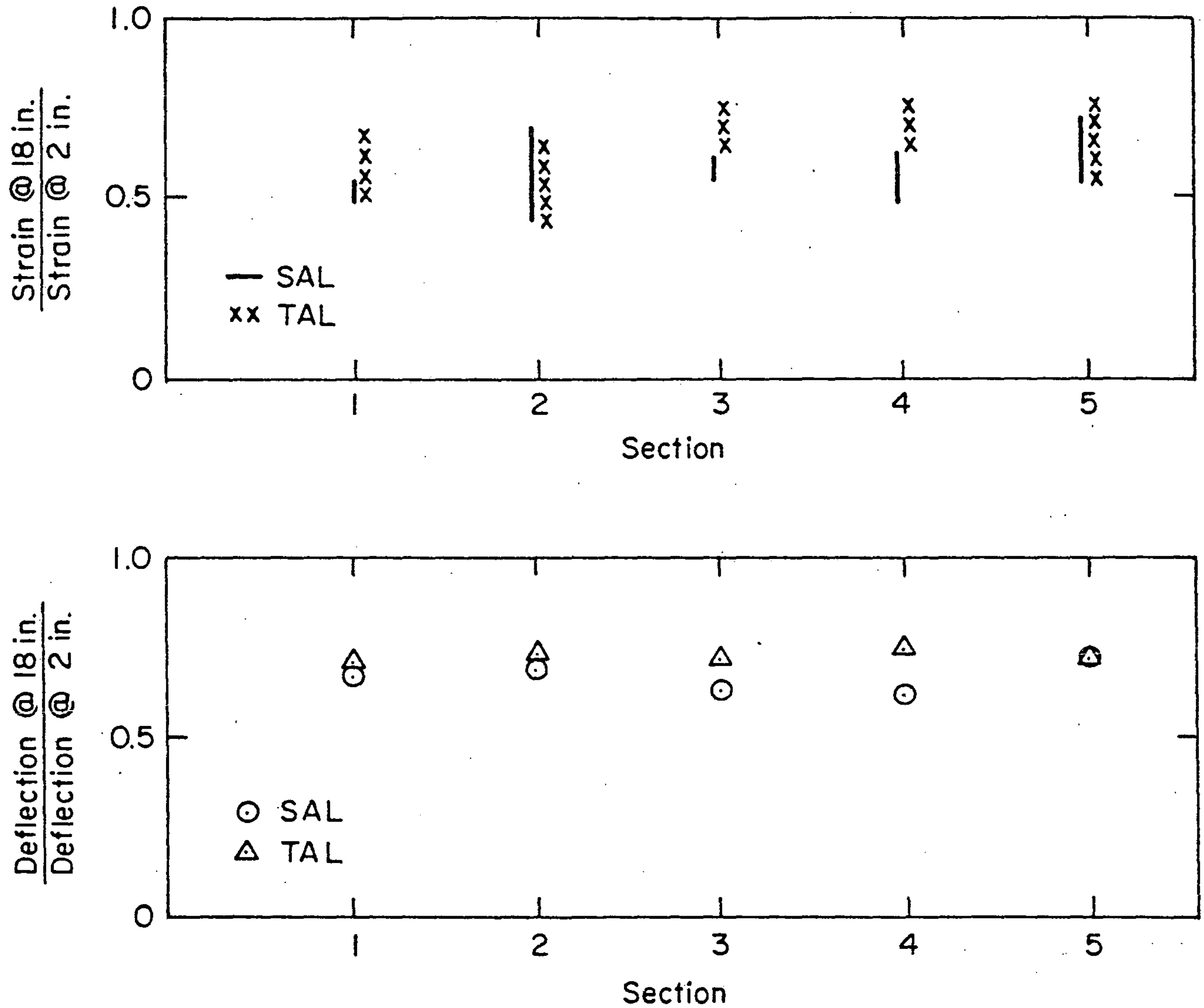
edge deflections at these sections. One reason for lower measured edge stresses could be that the effective panel length (distance between transverse cracks) in the existing pavement is much shorter than 20 ft assumed in the theoretical analysis. The condition survey for Section 1, shown in Fig. 2, indicates an effective panel length of about 15 ft in the panels containing the instrumentation. The condition survey, for Section 2, shown in Fig. 3 indicates an effective panel length of about 20 ft in the overlay in the panels containing the instrumentation. However, the effective panel length in the existing pavement at Section 2 may be less than 20 ft.

Based on the comparisons shown in Figs. 19 and 20, it is seen that the overlaid pavements are behaving monolithically and that the overlaid pavements are responding as full-depth pavement.

Effect of Wheel Path

The field investigation was planned to also provide information on the effect of wheel path. As discussed previously wheel paths used for both the SAL and the TAL were 2 in. and 18 in. inside from the edge. The 2 in. wheel path simulated the edge loading condition. The effect of having a wheel path just 18 in. away from the edge is shown in Fig. 21. There is a significant reduction in measured edge stresses and edge deflections at all five sections for the wheel path at 18 in. compared to the wheel path at 2 in. Similar reductions were also measured for joint deflections.

Thus, lane widening at time of overlay, if practical, and lane widening at time of new construction if a tied-concrete shoulder is not used should be given serious consideration. Keeping truck traffic away from the free lane edge can significantly improve pavement performance by reducing critical stresses and deflections.



Notes: 1) Strains and Deflections (measured) are at mid-slab edge
2) @ 18 in. denotes wheel path at 18 in. from slab edge

Fig. 21 Comparison of Responses for Wheel Paths of 18 in. and 2 in. Inside from the Edge

APPLICATION OF STUDY RESULTS

Results of the analysis indicate that the four overlaid pavement sections evaluated as part of the reported study are performing as monolithic pavements with high interface shear strength at the interface. The strength of the existing pavement at all of the four overlaid test sections was high. In addition, cores obtained from Sections 4 and 5 did not indicate D-cracking related damage in the overlay concrete.

Comparison of the condition surveys for Section 1 (non-overlaid JRCP) and Section 2 (overlaid JRCP) indicate that all cracking in the existing pavement is not reflected through the overlay and that the cracks that did reflect through have remained tightly closed. Similarly, the condition survey of Sections 4 and 5 indicate that cracks reflected through the overlay continue to remain tightly closed even after almost seven years of service.

The field investigation conducted by CTL verifies that for properly constructed bonded overlays, pavement strengthening is achieved and that the overlaid pavement behaves monolithically as a full-depth concrete pavement. A design procedure for pavement strengthening was recently developed by CTL for Portland Cement Association. This procedure is simple to use and requires knowledge of concrete and subbase/subgrade properties. The procedure is presented next.

Design Procedure for Bonded Overlays

The PCA design procedure for bonded overlays requires evaluation of the in-site material properties of the existing pavement and use of design charts to obtain the thickness of the concrete overlay.

In-Situ Material Evaluation

Primary objectives of the material evaluation program are to identify the subgrade and subbase materials and to determine strength-related properties of the concrete. These data are needed for determination of the bonded resurfacing thickness.

The subgrade and subbase materials are identified and design moduli of subgrade reaction values at the subbase surface are established for the different sections of the project. The modulus of subgrade reaction at the surface of the subbase may be back-calculated from results on nondestructive load testing, if performed.

For the existing pavement concrete, it is necessary to obtain representative values of the flexural strength and the modulus of elasticity. Because it is not practical to obtain beam specimens from the pavement, it is recommended that splitting tensile tests be made on pavement cores.

The splitting tensile test consists of loading in compression a concrete cylindrical specimen placed on its side. Load is increased until failure by splitting along the vertical axis takes place. The vertical axis of the core under test should correspond to the transverse axis of the core in-place. The test should be conducted in accordance with procedures of ASTM Designation: C496, "Splitting Tensile Strength of Cylindrical Concrete Specimens." Splitting tensile strength of the specimen is calculated as follows:

$$f_t = \frac{2\ P}{\Pi l d}$$

where:

f_t = splitting tensile strength, psi

P = maximum applied load, lb

l = specimen length, in.

d = specimen diameter, in.

One core should be taken every 300 to 500 ft. Cores should be obtained at midslab and about 1 ft from the outside lane edge. Core diameter should be nominally 4 in. or more. Bottom of cores may be trimmed about 1/2 in.

For each section, the effective value of the splitting tensile strength is determined as follows:

$$\text{Design } f_{t_e} = \bar{f}_t - 1.65\ s$$

where:

f_{t_e} = effective splitting tensile strength for the section

$\bar{f}_t$ = average value of the splitting tensile strength for the section, psi.

s = standard deviation of the strength values for the section , psi.

Obtaining the effective value of the splitting tensile strength using the cited equation implies that only 5% of the pavement section has splitting tensile strength less than the effective value.

The design flexural strength value for a section is then obtained using the following relationship:

$$f_r = Af_{t_e}$$

where:

f_r = design flexural strength, psi

f_{t_e} = effective splitting tensile strength, psi

A = regression constant

Values of A reported in the literature range from about 1.35 to about 1.55. When available, a value of A based on local experience should be used. In the absence of local experience, a value of A of 1.45 is suggested.

The modulus of elasticity of the existing concrete pavement may be determined by testing concrete cores in accordance with procedures of ASTM Designation: C469, "Static Modulus of Elasticity and Poissons Ratio of Concrete in Compression," or by using the following approximate relationship:

$$E_c = D\, f_r$$

where:

E_c = design modulus of elasticity for the section, psi

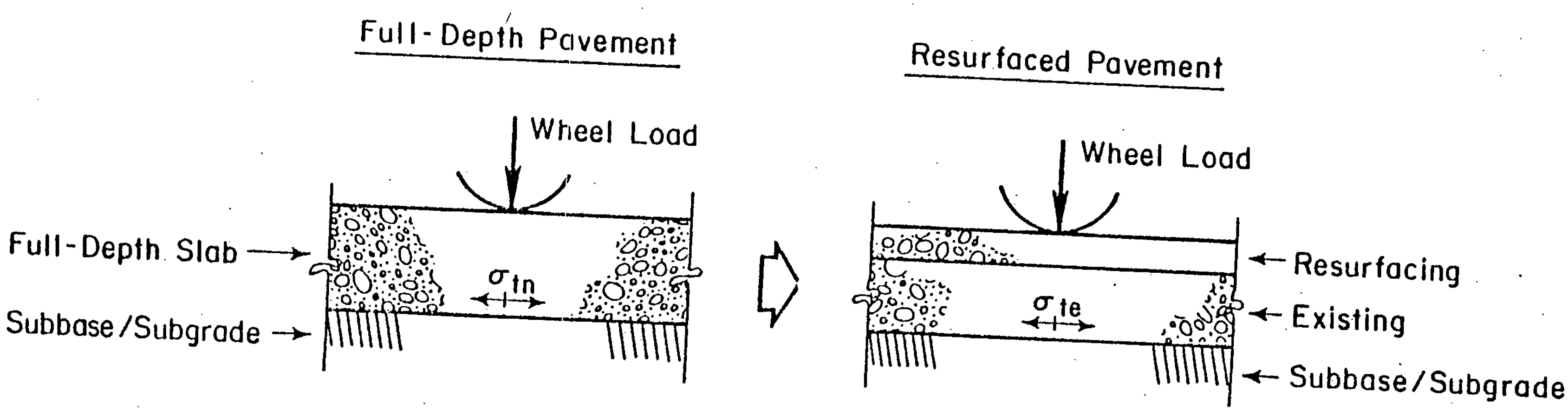
f_r = design flexural strength

D = constant = 6,000 to 7,000

<u>Determination of Overlay Thickness</u>

The design procedure for bonded resurfacing is based on providing a pavement such that the base slab plus the bonded resurfacing is structurally equivalent to a new full-depth concrete pavement designed to carry the anticipated future traffic. This implies resurfaced pavement will provide a service life that is equivalent to that provided if a new full-depth concrete pavement were to be constructed.

The premise of structural equivalency is based on ensuring that the critical normalized stress that may develop in a pavement with bonded resurfacing is equal to or less than the critical normalized stress in a new concrete pavement with similar support conditions. Normalization is done with respect to concrete flexural strength. The structural equivalency concept for bonded resurfacings is illustrated in Fig. 22. Thus, the design procedure requires knowledge of the following parameters for the resurfaced and full-depth pavements:



Resurfaced pavement to be structurally equivalent to full-depth pavement designed for anticipated future design traffic.

Thus, $\frac{\sigma_{te}}{f_{re}} < \frac{\sigma_{tn}}{f_{rn}}$

where

σ_{te} = critical edge stress in resurfaced pavement

σ_{tn} = critical edge stress in full-depth pavement

f_{re} = flexural strength of existing pavement concrete

f_{rn} = flexural strength of full-depth pavement concrete

Fig. 22 Stress Equivalency Concept

1. Design flexural strength

2. Critical tensile stress

For determination of critical tensile stresses, modulus of elasticity for the existing concrete pavement and for the new concrete pavement are needed. Program JSLAB was used to determine critical tensile stresses for the case of edge loading in a full-depth concrete pavement and in an existing pavement with bonded overlay.

The computed critical tensile stresses were then used to prepare design charts for determination of bonded overlay thickness. These design charts are applicable for the following conditions:

1. Modulus of elasticity of full-depth concrete pavement of 4,000,000 or 5,000,000 psi

2. Flexural strength of full-depth concrete pavement of 600 to 650 psi.

3. Value of constant D of 6,000 and 7,000 in the following relationship for the existing concrete pavement: $E_c = Df_r$

Design charts shown in Fig. 23 were prepared for the following three categories of existing pavement concrete flexural strength:

1. Existing pavement concrete flexural strength ranging from 425 to 475 psi.

2. Existing pavement concrete flexural strength ranging from 476 to 525 psi.

3. Existing pavement concrete flexural strength ranging from 526 to 575 psi.

Charts were not prepared for existing pavement concrete flexural strength values lower than 425 psi and for flexural strength values over 575 psi. For cases when flexural strength values are lower than 425 psi, the large theoretical thickness requirements may not warrant use of bonded overlay.

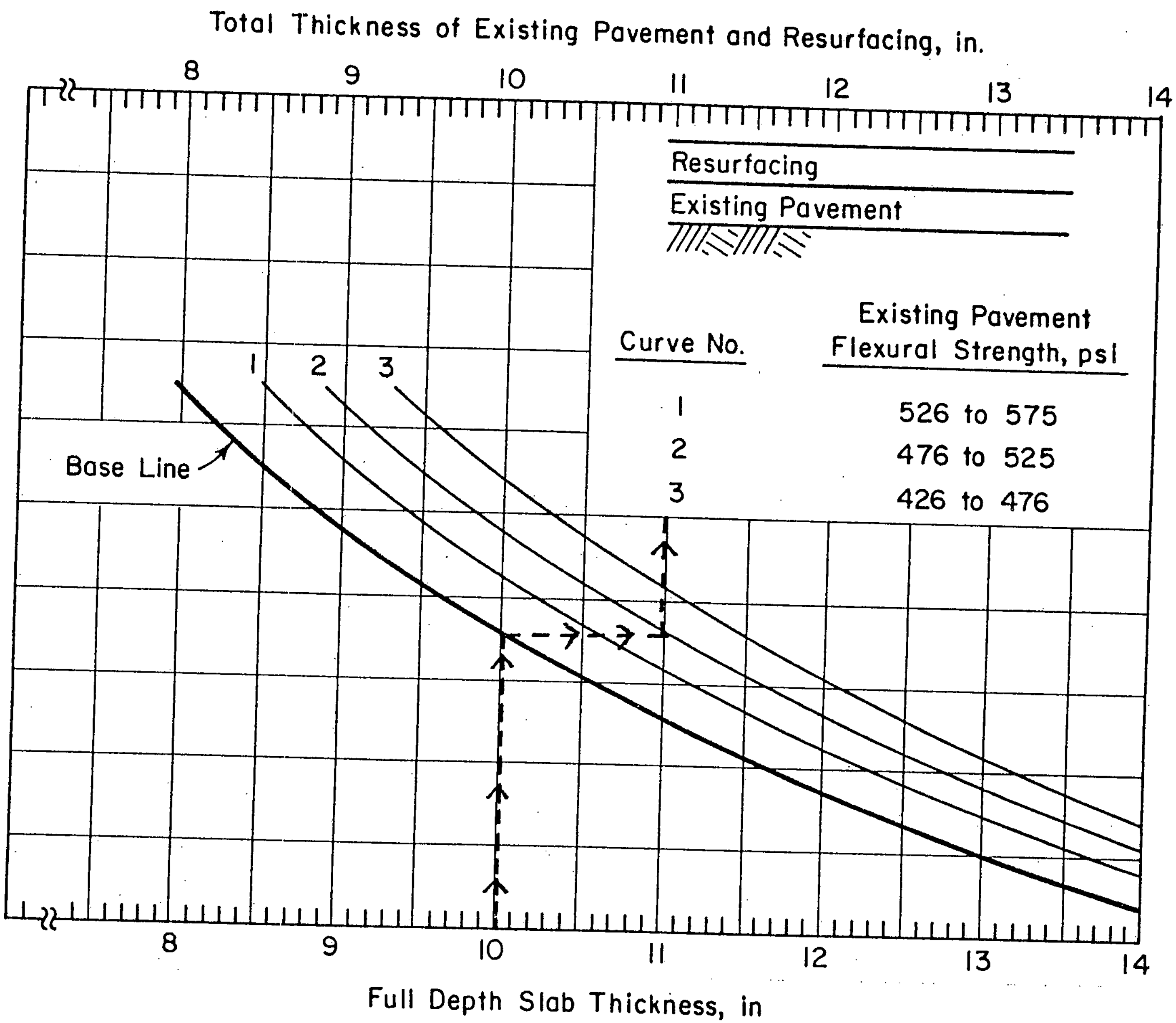


Fig. 23 Design Chart for Bonded Overlays

Also, an existing pavement with such low flexural strength may already exhibit distress that would not warrant use of bonded concrete overlay. In such a case, use of a directly placed or an unbonded concrete layer should be considered.

For the case when flexural strength of the existing pavement concrete is over 575 psi, then the overlay thickness required should equal the difference between the required full-depth pavement thickness and the existing pavement thickness plus the depth of surface milling of scarification.

It should be noted that the maximum bonded overlay thickness recommended is 5 in. When bonded resurfacing thickness exceeds 5 in. for highway application, then use of a directly placed or an unbonded concrete overlay may be a more cost-effective solution. In addition, use of bonded overlay less than 2 in. is not recommended for highway pavement strengthening.

The first step in the design process involves determination of the thickness of a full-depth new concrete pavement that would be needed for the anticipated future traffic. The support condition used to determine the thickness of the full-depth pavement should be equal to that for the existing pavement.

Design charts given in Fig. 23 are then used to determine the total thickness of the existing pavement plus the bonded over-lay, t_t. Use of the design charts is illustrated in Fig. 23. After the total thickness of the existing pavement plus bonded overlay, t_t, is determined, the actual as-constructed overlay thickness is determined as follows:

$$t_o = t_t - t_e$$

where:

t_o = as-constructed overlay thickness, in.

t_t = total thickness of existing pavement and bonded overlay obtained from the design charts, in.

t_e = existing pavement thickness after milling or scarification, in.

A sample problem is presented to illustrate use of the design procedure for bonded overlay. Assume the following design parameters have been determined for the pavement section to be overlaid:

Existing concrete pavement thickness after milling, t_e = 7.5 in.

Average splitting tensile strength, $\overline{f}_t$ = 430 psi

Standard deviation for splitting tensile strength tests, s = 50 psi

Modulus of subgrade reaction = 100 pci

Parameters for new concrete pavement:

Design thickness = 10 in.

Design flexural strength = 600 psi

Design modulus of elasticity of concrete = 4,000,000 psi

Design modulus of subgrade reaction = 100 pci

Design flexural strength of the existing concrete pavement is computed as follows:

Effective tensile strength = 430 - 1.65 (50)

= 358 psi

Using a value of 1.45 for regression constant A,

Design flexural strength = 1.45 (358)

= 520 psi

Using Fig. 23, the total thickness of the existing pavement and the resurfacing, t_t, should be 11.0 in.

The as-constructed resurfacing thickness is then determined as follows:

$$t_o = t_t - t_e$$
$$= 11.0 - 7.5$$
$$= 3.5 \text{ in.}$$

Thus, for the design example, a 3.5-in.-thick bonded resurfacing would be required.

CONCLUSIONS

Based on the work performed, the following conclusions are presented:

1. The four bonded concrete overlay sections evaluated are behaving monolithically.
2. Use of bonded concrete overlay over original concrete pavement do strengthen the original concrete pavement and therefore extend the service life of the original pavement.
3. Condition survey of the overlaid concrete pavements indicate that there is less surface cracking and the cracking that exists is generally tight and of low severity.
4. For properly constructed bonded concrete overlays, high interface shear strength can be achieved. Interface shear strength for the four sections with bonded concrete overlay ranged from 370 to 550 psi.

SUMMARY

A field study was conducted to evaluate the performance of bonded concrete overlays at four test sections in Iowa. Pavement deflections and strains were measured along the pavement edge. Deflections were also measured at joint corners for sections with jointed pavement.

Study results indicate that the bonded overlay sections are behaving monolithically and a high level of interface shear strength exists at the interface of the overlay and the existing concrete pavement. A design procedure is presented for determining the thickness of the bonded overlays to obtain pavement strengthening and to extend the service life of the existing concrete pavement.

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The opinions and findings expressed or implied in the paper are those of the authors. They are not necessarily those of the Iowa Department of Transportation.

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