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Interim Report

INSTRUMENTATION AND FIELD TESTING OF WHITETOPPING PAVEMENTS IN COLORADO AND REVISION OF THE TWT DESIGN PROCEDURE

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DEPARTMENT OF TRANSPORTATION
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Whitetopping has recently been generating considerable interest and greater acceptance as an approach to asphalt pavement rehabilitation. A number of thin whitetopping (TWT) and ultra-thin-whitetopping (UTW) pavement test sections have been constructed during the past 10 years, and the pavements have demonstrated considerable advantages as a rehabilitation technique.
In 1996 the Colorado Department of Transportation (CDOT) sponsored a research project to develop a mechanistic design procedure for TWT pavements.[1,5] Construction Technology Laboratories, Inc. (CTL) installed the instrumentation, conducted the load testing on the instrumented test sections, performed a theoretical analysis, and developed a TWT design procedure for CDOT. Many variables were considered in the construction of the test sections, including concrete overlay thickness, slab dimension, existing asphalt layer thickness, different asphalt surface preparation techniques, and the use of dowel bars and tie bars. Based on the original design procedure development, there are several observations and conclusions regarding use of TWT pavements for rehabilitation that should be examined more extensively with a supplemental investigation. The items include subgrade support conditions, required thickness of asphalt beneath the concrete layer, and effects of variable joint spacings.

New TWT pavement test sections were constructed during 2001 in conjunction with a TWT project constructed by CDOT on SH 121 near Denver, Colorado. This provided an opportunity to instrument and load test additional TWT test sections and use the data to calibrate and verify the existing observations and design procedure. Therefore, the objective of this project is to instrument, load test, and monitor the new and original TWT test section performances to supplement and confirm the results of the 1996 study.

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by

Chung Wu
Matthew Sheehan

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EXECUTIVE SUMMARY

Whitetopping has recently been generating considerable interest and greater acceptance as an approach to asphalt pavement rehabilitation. A number of thin whitetopping (TWT) and ultra-thin-whitetopping (UTW) pavement test sections have been constructed during the past 10 years, and the pavements have demonstrated considerable advantages as a rehabilitation technique.

In 1996 the Colorado Department of Transportation (CDOT) sponsored a research project to develop a mechanistic design procedure for TWT pavements.[1, 5] Construction Technology Laboratories, Inc. (CTL) installed the instrumentation, conducted the load testing on the instrumented test sections, performed a theoretical analysis, and developed a TWT design procedure for CDOT. Many variables were considered in the construction of the test sections, including concrete overlay thickness, slab dimension, existing asphalt layer thickness, different asphalt surface preparation techniques, and the use of dowel bars and tie bars. Based on the original design procedure development, there are several observations and conclusions regarding use of TWT pavements for rehabilitation that should be examined more extensively with a supplemental investigation. The items include subgrade support conditions, required thickness of asphalt beneath the concrete layer, and effects of variable joint spacings.

New TWT pavement test sections were constructed during 2001 in conjunction with a TWT project constructed by CDOT on SH 121 near Denver, Colorado. This provided an opportunity to instrument and load test additional TWT test sections and use the data to calibrate and verify the existing observations and design procedure. Therefore, the objective of this project is to instrument, load test, and monitor the new and original TWT test section performances to supplement and confirm the results of the 1996 study.

Implementation Statement

This is the construction report describing the details of construction and instrumentation for the TWT research project on SH 121, from C470 to Park Hill Ave. The primary objectives of this research project are to revise or validate the current CDOT TWT pavement design procedures and to better understand the TWT pavement behavior and performance for highway applications. Final implementation for this research project will be addressed at the completion of the study.

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Construction Report

By Chung Wu[1] and Matthew Sheehan[2]

INTRODUCTION

Thin whitetopping (TWT) and ultra-thin whitetopping (UTW) are techniques for asphalt pavement rehabilitation that have gained considerable interest and greater acceptance in the last decade. Essentially, the TWT and UTW techniques involve placing a concrete overlay (typically 4 to 6 in. or 2 to 4 in., respectively) on deteriorated asphalt pavements. Unlike the conventional whitetopping approaches used previously, the TWT and UTW techniques recognize that certain bonding strength exists between the concrete overlay and the existing asphalt layer.(1,2,3) The TWT and UTW pavements, therefore, behave as composite pavements. Normally, short joint spacing, between 2 and 12 ft, depending on slab thickness, has been used for TWT and UTW pavements. The existence of interface bonding and the use of short joint spacings minimize slab bending, potential for shrinkage cracking, slab curling and warping, and reduces the required slab overlay thickness. Thin whitetopping pavements are often used for state and secondary highways subjected to moderate truck traffic while UTW pavements are intended for city streets or intersections with minimal truck traffic.

In 1996 the Colorado Department of Transportation (CDOT) sponsored a research project to develop a mechanistic design procedure for TWT pavements.(1, 5) This project involved construction of three TWT pavements containing many test sections with field instrumentation. Construction Technology Laboratories, Inc. (CTL) installed the instrumentation, conducted the load testing on the instrumented test sections, performed a theoretical analysis, and developed a

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TWT design procedure for CDOT. Many variables were considered in the construction of the test sections, including concrete overlay thickness, slab dimension, existing asphalt layer thickness, different asphalt surface preparation techniques, and the use of dowel and tie bars.

The developed design procedure has been regarded as a first-generation TWT pavement design procedure, and needs to be further calibrated, verified and/or modified as more performance data become available. As stated in the current design procedure, there are several observations and conclusions regarding using TWT pavements for rehabilitation that should be examined more extensively. During the 2001 construction season, CDOT planned to construct a new 4-mile long TWT pavement on SH 121 near Denver, Colorado. This provided an excellent opportunity to collect additional data that can be used for verification and modification of the current design procedure.

The TWT pavements were constructed in late July and early August 2001. This report presents information related to instrumentation and construction of the TWT test sections.

OBJECTIVES AND SCOPE

The overall objectives of this project are to revise the current CDOT TWT pavement design procedure and to study the TWT pavement behavior and performance for highway applications. These objectives will be accomplished by conducting the following scope of work.

- Literature and document review.
- Instrumentation, construction, and field load testing of the newly constructed test sections.
- Performance evaluation (condition survey) of these new and previously constructed TWT pavements in Colorado.
- Laboratory testing for material characterization and interface bonding strength determination.
- Verification and validation of the current design procedure using the obtained data.
- Assessment and revision of current CDOT TWT design procedure.

The observations and information documented during this project will contribute to the advancement of whitetopping technology through increased knowledge of techniques and considerations critical for constructing whitetopping pavements.

PROJECT DESCRIPTION

The test sections are located on a 4-mile long TWT pavement project on SH 121, between Colorado Route C 470 and Park Hill Avenue, south of Denver, Colorado. The general location of this TWT project is presented in Figure 1. This section of SH 121 is a four-lane divided secondary arterial with stoplights at the intersections. The general design of the TWT project included whitetopping overlay of 6 in. with 6-ft joint spacing. The TWT was designed to carry approximately 1.3 million 18-kip equivalent single axle loadings (ESALs) over a 10-year design period. The original asphalt concrete thickness for this pavement was 5-1/2 inches, but the existing asphalt surface will be milled to promote improved interface bonding between the existing asphalt and new concrete. The general design information for the TWT section is presented in Table 1.

Table 1. General Pavement Design Information

Roadway	Design Parameter	Value
SH 121 (C 470 to Park Hill)	Highway Category	Secondary
	Design Life (years)	10
	Design Traffic (18-kip ESAL)	1,272,000
	Joint Spacing (in.)	72
	Concrete Elastic Modulus (psi)	3,400,000
	Concrete Poison's Ratio	0.15
	Existing AC Thickness (in.)	5-1/2
	AC Elastic Modulus (psi)	266,000
	AC Poison's Ratio	0.35
	Modulus of Subgrade Reaction (psi/in.)	500
	Design Concrete Overlay Thickness (in.)	6

For the TWT test sections, two primary experimental variables, concrete slab thickness and joint spacing (or slab dimension), were considered. There were two levels of slab thickness and two levels of joint spacing for each thickness, resulting in four different experimental combinations, as presented in Table 2. All other design parameters and material properties were kept constant.

The test sections were located at the beginning of the southbound lanes (north end of the project) from approximately station 187+00 to station 197+00. Each test section was 200 feet long, with a 200-ft-long transition zone between the 4 inch and 6 inch sections. The 4-in.-thick sections were located at the northern end of the paving operation, and the 6-in.-thick sections were after the 4-in.-thick sections and the 200 ft transition zone.

Table 2. Thin Whitetopping Project Primary Experimental Variables

Test Section	Concrete Thickness, in.	Joint Spacing, ft
1	4	4-ft x 4-ft
2	4	6-ft x 6-ft
3	6	6-ft x 6-ft
4	6	6-ft x 9-ft

The order of test sections was slightly altered from the original plan outlined in the project proposal. The original test section layout involved constructing the two 6-in.-thick sections on the north end of the project before the 4-in.-thick sections. Since the remainder of the paving was designed to have 6-in.-thick slabs, the locations of the 4-in.-thick test sections were shifted so that the contractor would only have to adjust the paver once, and there would only be one thickness transition zone.

In general, the pavement had 10-ft-wide outside and 4-ft-wide inside concrete shoulders. All concrete shoulders were constructed monolithically with the main lane pavements. In addition, the entire lane was designed with a uniform cross slope across both shoulders and lanes.

The SH 121 rehabilitation project is representative of a typical situation when a TWT overlay could be considered. The traffic levels on this section of roadway are relatively high, but currently are limited to general vehicular and light truck traffic. The construction of a TWT overlay minimizes the amount of traffic interruption by expediting the construction and paving activities; using the existing asphalt as a base course facilitates the construction of a concrete pavement without requiring a more extensive and time consuming complete reconstruction project.

EXPERIMENTAL DESIGN AND FIELD TESTING PLAN

As discussed in the previous section, the two primary variables to be evaluated in the study were the slab thickness (two levels) and panel joint spacing (two levels for each thickness), resulting in four different combinations. In the original proposal, it was planned to instrument and test three replicate slabs for each test section for a total of 12 instrumented slabs. The multiple slab instrumentation installations would provide pavement response measurements more representative or indicative of actual pavement conditions. Replicated measurements could address some of the variability that is expected from the testing data. The measured pavement response in the replicate slabs could be averaged to more accurately represent the responses of the slabs in the test sections. The following tests on the test sections were planned:

- Static load testing with strain measurements.
- Surface profile measurements over daily temperature variations.
- Joint opening measurements.
- Temperature measurements.
- Pavement coring and laboratory testing.
- Ground penetrating radar testing for thickness estimation.
- FWD tests.

Two sets of field tests were performed. The first set was conducted about 28 days after pavement construction and the second will be one year after TWT pavement construction. Performing these 28-day and one-year load tests allows for the evaluation of the test section responses after being exposed to extended traffic repetitions and one full freeze/thaw cycle.

PRE-CONSTRUCTION PAVEMENT EVALUATION

A pre-construction investigation of the existing asphalt pavement was conducted on April 24 and 25, 2001. The evaluation was performed by the Colorado DOT and included a visual condition survey, rutting measurements, coring, and FWD testing.

Visual Condition Survey

Severe distresses in the form of fatigue cracking in both left and right wheel paths were detected in Test Sections 1 and 2 and also in the transition section. Presence of potholes was quite evident throughout these test sections. Figure 2 shows the typical fatigue cracking for these two test sections. However, as presented in Figure 3, distresses in Test Sections 3 and 4 were minor and were in the form of longitudinal cracking next to the centerline.

Severe cracking was observed for Test Sections 1 and 2. However, a large portion of the cracking was removed after the milling of about ½ in. of asphalt during construction.

Rutting Measurements

Rut-depth measurements were taken at 50-foot intervals in the left-wheel path (LWP) and in the right-wheel-path (RWP) for both inside and outside lanes (Figure 4). All the measured rutting was considered in the low range, with the average ranging from 1/8 in. to 3/8 in for the four test sections. Table 3 shows the average rut-depth for the four test sections.

Table 3. Average Rut Depth of the Existing Asphalt Pavement

Test Section	Measture Rut-Depth, in.			
	Traffic Lane		Passing Lane	
	RWP	LWP	RWP	LWP
1	3/8	3/8	1/8	3/8
2	3/8	3/8	1/8	3/8
3	1/8	3/8	1/8	2/8
4	3/8	3/8	3/8	1/8

Pavement Coring

As presented in Figure 5, pavement cores were drilled at 50-foot intervals, for each of the four test sections, resulting in a total of 12 cores. In each test section, the first and third cores were taken in the right wheel path and the second core taken in the middle of the lane. Cores were used to verify the thickness of the asphalt pavement in all four sections. As shown in Table 4, the thickness of the first and the second test sections ranged from 5 ½ to 6 inches, and the thickness of the third and the fourth test sections ranged from 6 ½ to 8 inches.

Table 4. Measured Core Thickness of the Existing Asphalt Pavement

Test Location, ft	Asphalt Layer Thickness, in.			
	Test Section			
	1	2	3	4
50	6.0	5.8	7.5	7.5
100	6.0	6.0	8.0	7.0
150	5.8	5.5	7.3	6.5
Average	5.9	5.8	7.6	7.0

FWD Testing

FWD tests were conducted on the four test sections on April 24, 2001, from 7:00 pm to 7:45 pm (Figure 6). Air temperature was in the range of high 40's (°F) and the pavement temperature was about 47°F throughout the test. Tests were performed at 20-ft intervals, with three drops conducted at each location. The targeted FWD load was 9,000 lb, with the first drop at each location being a seating drop. The FWD deflection data for the four test sections are presented in Table 5. Please note that the deflections in the table have been normalized to a FWD load of 9,000 lb.

From this table, the average deflections under the loading center are 13.19, 15.14, 13.20, and 13.82 mils for Test Sections 1, 2, 3, and 4, respectively. The deflection data were also used to backcalculate the pavement layer moduli. From construction records, the existing asphalt pavement structure consisted of the AC layer, a CDOT Class 6 aggregate base of 4 in. and a Class 1 aggregate subbase of 10 in. The pavement was treated as a two-layer system, an AC layer and a foundation, in the backcalculation process. Summary of the backcalculated pavement layer moduli for the four test sections are shown in Table 6.

It can be observed from the table that all layer moduli are within reasonable ranges. The average estimated asphalt concrete moduli of elasticity are 398,700, 288,600, 334,600, and

Table 5. Normalized FWD Deflection Data

Test Section	Test Location	Drop Number	Normalized Load, lbf	Normalized Deflection for 9,000 lb FWD Load, mils						
				Distance from Loading Center, in.						
				0	8	12	18	24	36	60
1	0	2	9,000	13.08	10.42	8.12	5.84	4.02	2.68	1.44
		3	9,000	12.89	10.36	8.08	5.81	3.97	2.65	1.41
	20	2	9,000	13.71	10.54	8.59	6.25	4.31	2.84	1.63
		3	9,000	13.57	10.53	8.59	6.25	4.31	2.83	1.62
	40	2	9,000	14.89	10.69	8.47	6.12	4.17	2.77	1.50
		3	9,000	14.72	10.67	8.45	6.11	4.18	2.79	1.52
	60	2	9,000	13.43	10.61	8.53	6.16	4.23	2.81	1.64
		3	9,000	13.30	10.49	8.48	6.15	4.22	2.78	1.62
	80	2	9,000	12.05	9.90	8.31	6.38	4.51	2.88	1.51
		3	9,000	11.76	9.69	8.17	6.29	4.45	2.84	1.48
	100	2	9,000	12.33	10.02	8.22	6.04	4.08	2.55	1.32
		3	9,000	12.17	9.96	8.19	6.03	4.08	2.55	1.31
	120	2	9,000	13.02	10.54	8.58	6.23	4.01	2.30	1.09
		3	9,000	12.90	10.47	8.56	6.24	4.04	2.32	1.09
	140	2	9,000	14.34	11.66	9.54	7.04	4.74	2.94	1.22
		3	9,000	14.19	11.64	9.55	7.05	4.75	2.95	1.26
	160	2	9,000	13.13	10.91	9.06	6.77	4.61	2.81	1.36
		3	9,000	13.20	11.01	9.06	6.80	4.66	2.89	1.48
	180	2	9,000	12.60	10.50	8.64	6.45	4.43	2.81	1.58
		3	9,000	12.47	10.44	8.64	6.46	4.42	2.80	1.55
2	0	2	9,000	14.03	11.53	9.36	6.74	4.58	3.03	1.79
		3	9,000	13.94	11.51	9.40	6.75	4.60	3.04	1.79
	20	2	9,000	14.60	11.14	8.80	6.40	4.36	2.76	1.43
		3	9,000	15.01	11.55	9.14	6.67	4.58	2.90	1.49
	40	2	9,000	14.56	10.97	8.64	6.00	3.97	2.52	1.43
		3	9,000	14.49	11.00	8.65	6.03	4.00	2.56	1.46
	60	2	9,000	11.48	9.29	7.35	5.25	3.63	2.39	1.36
		3	9,000	11.41	9.25	7.34	5.26	3.64	2.37	1.35
	80	2	9,000	12.61	9.99	8.03	5.76	3.73	2.21	1.13
		3	9,000	12.48	9.97	8.00	5.76	3.74	2.22	1.15
	100	2	9,000	14.28	11.22	8.91	6.28	3.94	2.16	1.01
		3	9,000	14.20	11.24	8.96	6.34	3.98	2.16	1.02
	120	2	9,000	16.85	13.58	11.05	7.87	4.66	2.85	1.37
		3	9,000	16.70	13.50	11.02	7.89	4.67	2.87	1.34
	140	2	9,000	18.07	14.17	11.18	7.92	5.18	3.03	1.44
		3	9,000	17.68	13.96	11.05	7.82	5.13	3.02	1.46
	160	2	9,000	18.73	14.04	11.00	7.51	4.54	2.80	1.57
		3	9,000	17.72	13.39	10.51	7.19	4.35	2.68	1.53
	180	2	9,000	17.04	13.44	10.76	7.65	4.84	2.93	1.55
		3	9,000	16.94	13.45	10.79	7.68	4.88	2.94	1.53

Table 5. Normalized FWD Deflection Data (continued)

Test Section	Test Location	Drop Number	Normalized Load, lbf	Normalized Deflection for 9,000 lb FWD Load, mils						
				Distance from Loading Center, in.						
				0	8	12	18	24	36	60
3	0	2	9,000	10.02	8.41	7.11	5.54	4.05	2.68	1.38
		3	9,000	9.95	8.36	7.07	5.55	4.06	2.69	1.36
	20	2	9,000	11.92	10.23	8.76	6.84	4.91	3.15	1.51
		3	9,000	11.78	10.11	8.69	6.80	4.87	3.13	1.46
	40	2	9,000	14.36	11.82	10.12	7.94	5.80	3.79	1.82
		3	9,000	14.21	11.74	10.06	7.90	5.79	3.77	1.83
	60	2	9,000	13.99	11.94	10.08	7.75	5.49	3.54	1.64
		3	9,000	13.80	11.83	10.03	7.72	5.48	3.54	1.64
	80	2	9,000	13.19	11.16	9.54	7.46	5.50	3.63	1.78
		3	9,000	13.00	11.03	9.42	7.42	5.49	3.59	1.70
	100	2	9,000	15.82	12.72	10.55	8.13	5.81	3.71	1.59
		3	9,000	15.49	12.48	10.37	8.04	5.78	3.69	1.56
	120	2	9,000	13.57	10.97	9.20	7.13	5.13	3.30	1.46
		3	9,000	13.28	10.77	9.06	7.03	5.08	3.27	1.44
	140	2	9,000	13.71	12.75	11.11	7.71	5.32	3.12	1.47
		3	9,000	13.55	12.62	11.03	7.70	5.32	3.14	1.49
	160	2	9,000	13.69	11.80	10.09	7.96	5.81	3.78	1.72
		3	9,000	13.46	11.63	9.98	7.91	5.79	3.75	1.68
	180	2	9,000	12.73	10.80	9.13	7.07	4.96	3.04	1.33
		3	9,000	12.53	10.64	9.04	7.01	4.94	3.02	1.27
4	0	2	9,000	12.31	10.25	8.60	6.57	4.64	2.93	1.26
		3	9,000	12.16	10.16	8.56	6.57	4.65	2.92	1.30
	20	2	9,000	13.60	11.28	9.54	7.44	5.33	3.38	1.41
		3	9,000	13.42	11.13	9.43	7.38	5.33	3.39	1.45
	40	2	9,000	11.64	10.13	8.81	7.08	5.30	3.53	1.53
		3	9,000	11.44	9.97	8.71	7.02	5.28	3.52	1.52
	60	2	9,000	12.29	10.56	9.21	7.45	5.63	3.81	1.57
		3	9,000	12.18	10.47	9.18	7.44	5.67	3.85	1.58
	80	2	9,000	11.87	10.38	9.24	7.59	5.87	4.08	1.75
		3	9,000	11.80	10.36	9.22	7.62	5.90	4.12	1.76
	100	2	9,000	15.10	12.89	11.20	8.92	6.50	4.12	1.61
		3	9,000	14.89	12.76	11.11	8.89	6.49	4.13	1.60
	120	2	9,000	15.84	14.14	12.08	9.52	6.90	4.26	1.60
		3	9,000	15.56	13.83	11.83	9.37	6.80	4.23	1.51
	140	2	9,000	16.87	14.01	11.71	8.79	5.81	3.19	1.34
		3	9,000	16.54	13.76	11.52	8.68	5.77	3.18	1.34
	160	2	9,000	13.12	11.06	9.19	6.85	4.39	2.64	1.10
		3	9,000	13.03	11.03	9.20	6.90	4.44	2.66	1.10
	180	2	9,000	16.43	13.09	10.75	8.07	5.54	3.27	1.39
		3	9,000	16.24	13.00	10.70	8.07	5.54	3.32	1.43

394,200 psi for Test Section 1, 2, 3, and 4, respectively, and the average estimated elastic moduli of the composite foundation layer are 22,500, 21,800, 19,100, and 18,500 psi. The asphalt elastic modulus used in the design was 266,600 psi. It can also be observed that, within each test section, the estimated asphalt modulus varied widely. The coefficient of variation is as high as 38% (Section 3).

Table 6. Summary of the Estimated Layer Moduli of the Existing Asphalt Pavement

Back-Calculated Layer Moduli, psi	Test Section							
	1		2		3		4	
	AC	Base	AC	Base	AC	Base	AC	Base
Maximum	596,400	24,200	466,400	26,100	479,200	24,200	686,300	22,100
Minimum	260,500	20,900	164,000	18,600	237,500	17,200	205,800	15,100
Average	398,700	22,500	288,600	21,800	334,600	19,100	394,200	18,500
Standard Deviation, psi	88,900	1,100	91,300	2,700	64,900	2,100	151,200	2,100
Coefficient of Variation, %	22	5	32	12	19	11	38	11

INSTRUMENTATION

As proposed in the testing plan in the proposal, on each test slab, the instrumentation installation activities included the following.

- Embedded concrete strain gages.
- Surface concrete strain gages.
- Embedded thermocouples.
- Retrofitted temperature sensors.
- Reference rods.
- Whitmore plugs.

A portion of the instrumentation required for this project needed to be installed prior to construction of the TWT concrete overlay. This included the embedded concrete strain gages, reference rods and embedded thermocouples. Others would need to be installed just prior to load testing activities, such as the surface strain gages and temporary temperature sensors. The Whitmore plugs were installed just after the paving and contraction control joint sawing was completed. The instrumentation required prior to concrete paving was installed from June 17 to 19, 2001. The Whitmore plugs were initially installed on June 22 and 23, 2001, but supplemental installations were performed from July 25 to 29, 2001. The instrumentation activities performed just prior to the 28-day load test were performed from July 25 to 27, 2001.

The test slab locations were in the outside wheelpath of the traffic lane in all test sections. The specific slabs selected were near the center of each 200-ft-long test section. Each of the three test slabs per section was separated in the longitudinal direction from the following test slab by two concrete panels. Figure 7 presents the typical layout of test slabs within each test section.

Installation of Embedded Strain Gages

The typical layout of the strain gages for the four test sections is shown in Figure 8. In general, gages were placed at the slab center, along longitudinal joints adjacent to the concrete shoulder, along longitudinal joints on the concrete shoulder, and the transverse joint center. Also, as shown in Figure 9, multiple gages were used at designated locations. These multiple gages were installed on the concrete slab surface, 1 in. above the existing asphalt surface, and on the asphalt surface. There were six embedded strain gages on each test slab for a total of 72 for the entire project.

The embedded strain gages were fabricated and tested for stability in the CTL laboratory prior to arriving at the project site. They were made by epoxying ½-in.-long gages to the prepared, smooth surface of No. 3 steel bars. The gages installed at the concrete-asphalt interface were mounted on 12-in.-long bars and the embedded gages located at one inch above the asphalt-concrete interface gages were mounted on 16-in.-long bars. The embedded gages were installed from June 17 to 19, 2001, prior to pavement construction. The following is the sequence of the installation process:

- **Identification and Marking of Test Slab and Gage Locations** – The location of each test slab and gage was identified using the edge of the outside concrete shoulder and pavement centerline, which were provided by CDOT representatives (Figure 10). Also, as shown in Figure 11, after marking all test slabs and gage locations, the locations of all gages and joints were triangulated out to multiple reference points outside the roadway so the gage and joint locations could be accurately re-established following concrete paving.

- **Asphalt Surface Preparation** – To enhance bonding between the concrete overlay and the existing asphalt, the asphalt surface was milled, resulting in rough surfaces. The milled, rough asphalt surface needed to be prepared before gage installation. As shown in Figure 12, a diamond grinder was used to cut grooves in the asphalt surface for installing the interface gages. Two holes were then drilled into the asphalt layer that would be used to anchor the concrete embedment gages, also shown in Figure 12.
- **Gage Installation** – The bottoms of the grooves were cleaned with acetones and the interface gages were then epoxied into the prepared grooves. For installation of the concrete gages one inch above the interface, threaded rods were inserted into the drilled holes and the gages were tied to the rods as illustrated in Figure 13. The concrete gages were positioned directly above the interface gages and the one-inch spacing between the interface and embedded gages was maintained (Figure 14). Lead wires connected to the gages were recessed into the asphalt layer and were run to the edge of the pavements to protect them from the construction vehicles. The lead wires were individually labeled at the end for identification purpose and were buried at the pavement edge to further protect them during construction activities (Figure 15). All installed gages were then checked and all were functional.

Installation of Reference Rods

To serve as a basis for TWT pavement surface profile measurements, four 6-ft-long steel reference rods were installed, one at each test section. The reference rods were located on the concrete shoulder adjacent to the longitudinal joint between the traffic lane and the shoulder.

To install the reference rods, cores were drilled through the asphalt layer, and the rods were installed in the empty core hole locations by first pounding a steel pipe approximately 4 ft long into the ground (Figure 16). The pipe was intended to serve as a protective sleeve when inserting the steel reference rod into the ground approximately 4 ft below grade. Through the pipe, the steel reference rod was then driven into the ground about two feet beyond the depth of the protective pipe. This type of installation was utilized to prevent the reference rod from being affected by frost movement during the winter. A machined cap was screwed to the top of the reference rod to provide a consistent surface for the elevation measurement instrument to rest on when collecting slab deformation measurements. Figure 17 shows the continuation of the installation process.

A protective polyvinyl chloride (PVC) pipe assembly was used to protect the portion of the reference rod assembly above the asphalt grade from the TWT concrete and concrete paver. The top of this enclosed PVC assembly was set to an elevation just below the intended concrete surface. If the assembly was not set to the proper elevation, the paver might catch and tear it out during paving or the reference rod would be too low to be used as a reference point. Figure 18 shows the installed reference rod with the protective PVC assembly.

Installation of Embedded Thermocouples

As presented in a previous section of the report, thermocouples were installed at different depths in the concrete and asphalt layers (see Figure 9). Two test slabs were instrumented, one in the 4-in. and one in the 6-in. thick test sections. Prior to concrete construction, embedded thermocouples located five inches into the asphalt layer and at the asphalt-concrete interface were installed. Other thermocouples were installed later just before load testing. The thermocouples were used to monitor pavement temperature gradients during load testing activities.

Type K temperature sensors and thermocouple wires were prepared in the laboratory prior to arriving at the project site. A hole was drilled into the asphalt layer to the desired depth to install the asphalt layer embedded sensor. The ends of the wires were labeled for identification purposes. Grooves were cut on the asphalt surface to recess the thermocouple wires and to run the wires to the edge of the pavement protecting them from being damaged during construction.

Installation of Whitmore Plugs

Also included in the testing plan was the installation of Whitmore plugs at different locations across both transverse and longitudinal joints. These plugs were intended to measure slab movements and joint openings and might help determine if the contraction control joints were cracked as designed. It was proposed to install ten plugs at one test slab from each of the four experimental test sections. Typical locations for Whitmore plugs are shown in Figure 19.

The Whitmore plugs were installed on June 22 and 23, 2001, shortly after sawing the control joints. Holes were drilled in the concrete to provide recessed receptacles to receive the plugs and protect them from traffic. The plugs were anchored into position within the recessed holes with epoxy because the concrete was not strong enough to accept mechanical anchors. Because of concerns regarding the ability of the epoxy to withstand winter conditions, it was decided to install companion points at the time of the 28-day load testing. Measurements were collected between the plugs using a digital caliper. In the laboratory, conical holes were machined in the tops of the Whitmore plugs to provide stable reference points for the caliper to rest in when collecting measurements. Figure 20 shows the installed Whitmore plugs and the initial measurements used as baseline for future measurements.

Installation of Surface Strain Gages

Surface gages were installed just prior to the load testing activities, or approximately 28-days after the pavement construction. Tokyo Sokki PL-120-11 strain gages, with a 4-in. length, were used. The typical layout of the surface gages is presented in Figure 8. According to the plan, nine surface gages were to be installed on each test slab. Please note that it was originally proposed to load test three slabs for each experimental combination, resulting in twelve test slabs. Twelve slabs were instrumented during pavement construction. However, because of the heavy traffic in this section of the road and local regulation, the pavement could only be blocked between 8:30 am and 3:30 pm for the load testing. This time restriction would not allow for installation of surface gages and load testing of the 12 slabs. After discussion with and permission from Mr. Ahmad Ardani, the Colorado DOT project manager for this project, only two slabs were load tested for each combination (for a total of 8 slabs). It was felt that this would provide sufficient data for analysis purposes.

Before concrete construction, the locations of the embedded strain gages and control joints were triangulated out to multiple reference points. As shown in Figure 21, these reference points were used to accurately locate the surface gage locations so that they would match the locations of the embedded gages. The surface gages were placed directly over the embedded gages when appropriate. Gages near the joints were typically two inches from the contraction control joints as indicated in Figure 8.

The installation of the surface gages included the following:

- Cutting recessed slots into the concrete surface at each strain gage location (Figure 22).
- Cleaning the recessed slots using Acetones.
- Attaching gages to the recessed slots using fast-setting epoxy (Figure 23).
- Cutting grooves to the control joint locations for running the lead wires to the pavement edge (Figure 24).
- Soldering leads to the installed gages (Figure 25).
- Recessing the leads and running the leads to the pavement edge where the embedded gages were located (Figure 26).
- Checking installed gages (Figure 27).
- Applying hot wax over the gage and solder connections to protect them from moisture intrusion during the testing period (Figure 28).

In this project, the embedded and surface strain gages were used to measure strains induced by static truckloads placed on the pavement surface in selected locations. A typical test slab with installed surface gages is shown in Figure 29.

Installation of Additional Thermocouples

As mentioned previously, thermocouples in the concrete were not installed during construction. These temperature sensors were installed just prior to load testing activities. Holes were drilled to pre-determined depths in the concrete near the embedded temperature sensor locations (refer to Figure 9). The first hole was drilled to the mid-depth of the concrete and the second hole was drilled ½ inch into the concrete. A small amount of mineral oil was placed in the bottom of each of these drilled holes. A Type K thermocouple wire was placed in the mineral oil and the drilled hole was sealed to keep out debris. The thermocouple wires were then labeled, and were taped down and run out to the shoulder location where the previously installed sensor leads were located.

The temperature data were collected with automatic data loggers during load testing activities. These data will be used to monitor the temperature in the pavement system during load testing.

CONSTRUCTION

The SH 121 TWT pavement was constructed in the summer of 2001. The test sections for this project were located at the beginning of the southbound lane and were constructed on June 22, 2001. Interstate Highway Construction, Inc. (IHC) from Denver, Colorado was the paving contractor.

Asphalt Milling and Surface Preparation

The existing asphalt surface was cold milled by IHC on June 15 and 16, 2001. The asphalt milling removed ½ in. of the asphalt concrete to create a surface, which would promote enhanced interface bonding between the concrete and the asphalt layers. A 15-ft-long area of asphalt across the entire pavement width at the very beginning of the 4-in.-thick test sections (the north end of the paving operation) was milled 2 inches deeper than the remaining areas of the pavement. This additional milling was made to provide a thicker (6 in. thick) area at the beginning of the TWT where the new pavement transitions from asphalt to concrete. Past experiences have indicated that this is often an area susceptible to increased amounts of panel cracking and deterioration, and that constructing a thickened area at this location would help to eliminate the occurrence of cracking and distress. Figure 30 shows the milling operation and the rough asphalt surface after milling.

Previous studies by CTL and others have indicated that cold milling the existing asphalt surface promotes a stronger mechanical interface bond between the two layers and results in a composite pavement section to carry load induced stresses. In addition to milling the asphalt surface, the milled asphalt was swept multiple times, air blasted to remove any remaining debris or dust, and wetted prior to concrete placement. Figure 31 shows an example of IHC personnel air blasting the asphalt surface on June 22, 2001 just prior to concrete overlay placement in the test section locations. Each of these tasks was performed to provide a clean asphalt surface that would promote mechanical bond at the interface between the asphalt and new concrete overlay.

Concrete Mix Design

The concrete mixture used for the TWT overlay was typical for a slip-form paving mixture used in Colorado, with the exception that it included fiber reinforcement. The specified compressive strength for the mixture was 4,200 psi at 28-days. The concrete supplier was also Interstate

Highway Construction located in Denver, Colorado. Table 7 presents the concrete mixture proportions provided to CTL:

Table 7. Concrete Mix Design

Cement	585 lb
Fly Ash (Class F)	113 lb
Coarse Aggregate	1,614 lb
Sand	1,320
AEA	2.5 oz
Water	264 lb
Polypropylene Fiber	3 lb

Note: Based on one cubic yard SSD Batch Weight

Concrete Paving

The TWT test sections were paved on June 22, 2001. String lines and elevations were set on June 18 and 19, 2001 with the intention of paving on June 20, 2001. However, because the concrete trucks were unavailable, the paving operation was delayed by two days. The 4-in.-thick test sections, located at the north end of the project, were constructed first. Following a 200-ft transition area, all remaining pavement was designed to be nominally 6 in. thick.

Paving started at approximately 6:30 am on June 22, 2001. The paver started at approximately Station 196+25 and paved the southbound lanes in the direction of traffic. The first and second test sections (4-in.-thick test slabs) were paved at approximately 6:45 am and 7:15 am, respectively. Test Sections 3 and 4 (6-in.-thick test sections) were paved starting at approximately 8:30 am and were finished by 9:15 am. A photograph of the paving train is presenting in Figure 32.

Dowel bars were not used in transverse control joints in the TWT pavement construction. However, tie bars were placed at 30 inches on-center along all longitudinal contraction joints. The paver was equipped with an Automatic Tie Bar Inserter and placed all tie bars automatically, except for Test Section 1, where tie bars were placed manually using tie bar chairs. Section 1 had 4 ft by 4 ft joint spacing, while all other test sections had a 6-ft spacing between longitudinal joints, and the Automatic Tie Bar Inserter was set for 6-ft spacing. For Test Section 1, the tie bars were placed on chairs at the joint locations, and the chairs were fastened to the asphalt. The chairs were set so the tie bars would be at the mid-depth of the 4-in.-thick concrete slabs. The tie bar assemblies for Test Section 1 are shown in Figure 33.

The test slab locations in each test section were marked to prevent concrete trucks from damaging the instrumentation as they were backing in to deliver concrete. As the paver approached each set of test slabs, concrete was placed by hand around the embedded strain gages and thermocouples to ensure proper consolidation around the instrumentation and to reduce the possibility that the gages would be damaged by the paver passing over the test slab locations. The instrumentation could be damaged if the paver was set low enough to reach the gages or if a large amount of concrete was being pushed ahead of the paver as it passed the instrumentation locations. The placement of concrete around the strain gages is shown in Figure 34.

Since this stretch of pavement is located in densely populated areas, traffic noise is a major concern. To minimize traffic noise, final surface texture was provided by Astroturf drag, as shown in Figure 35.

Transverse and Longitudinal Control Joint Sawing

Once the concrete gained sufficient strength to support people walking on the surface, the locations of the gages and test slab joints were identified and marked using reference points established prior to paving (Figure 21). The joint sawing subcontractor marked out the remaining control joints prior to initiate sawing activities. Slight adjustments to the longitudinal sawcut locations were necessary to ensure that the embedded strain gage locations properly corresponded to the joint locations and matched the locations of the embedded gages. Test Section 2 required the only significant adjustment of the joint location, where the joint location was moved approximately 6 inches toward the shoulder. The joint location was adjusted because the original baseline marks provided to install the gages were inaccurate.

The transverse joints were sawed prior to the longitudinal joints. The joint sawcutting subcontractor performed trial sawcuts at the beginning of the paving to determine when the concrete had gained sufficient strength to allow for sawcutting without raveling of the sawcut edges. The transverse sawing started at approximately 2:00 pm on June 22, 2001, about 7½ hrs after paving. Two self-propelled saws were used to perform the transverse sawcuts, as shown in Figure 36. Soffcut saws were on site but only used for the 4-ft by 4-fy test sections.

A train of walk-behind and self-propelled diamond blade concrete saws were used to cut the 6-ft longitudinal joints. This approach was utilized because the assembly could be set up to maintain the proper spacing between saws and make straight and evenly spaced longitudinal sawcuts, and because it would make more efficient joint sawing. The assembly used to maintain the proper saw spacing consisted of a bar placed across the front of the saws and used as a guide. A photograph of the longitudinal sawing operation is presented in Figure 37, and the finished pavement surface is shown in Figure 38.

Instrumentation Installation After Concrete Construction

Following the completion of the control joint sawing for the test sections to establish the test slab joint locations, the locations of the reference rods were identified and the PVC protective sleeves were exposed, and the Whitmore gages were installed. The reference rod PVC pipe sleeves were covered by approximately 3/8 in. of concrete following paving. Concrete over the PVC sleeve covers was chipped away, the recessed plastic covers unscrewed, and the reference rod checked for damage and the proper elevation. A photograph of the reference rods and PVC protective assembly is presented in Figure 39.

As presented in the previous sections, Whitmore gages were installed on concrete surface (recessed just below the surfaces) for measuring joint movements (Figure 20).

Construction Concerns

One of the primary concerns regarding TWT construction and load testing involves the cracking or lack of cracking at the control joints. Since the concrete overlay is bonded to the underlying asphalt layer, the concrete slabs often do not tend to exhibit as much movement at the contraction control joints as a conventional pavement. Therefore, cracks do not always form at all of the control joint sawcut locations, at least during the early stages of the TWT service life. This is not typically a concern that affects the performance of the TWT pavement, but it can have an effect on load test results if the joints of the test slab are not cracked. The loading conditions being evaluated include edge or joint load conditions, but if the joint at the strain gage location is not cracked, the measurements collected at such a location will not be indicative of edge loadings.

An additional issue involving the contraction control joints is that the joints may not be precisely where they were intended with respect to the embedded strain gage locations. The reference points established prior to paving should have addressed this issue, but impulse radar was also used to verify the locations of the embedded gages. This is an important concern because the strain measurements and subsequent modeling of load stresses can be affected by slight misplacements of strain gage positions.

Another issue related to the embedded gages involves damage incurred during paving. Based on previous experience, a portion of the embedded gages may be damaged and non-functional following paving operations. The replicate test slab instrumentation installations in each test section were incorporated into the instrumentation program to address this issue. If some embedded gages were damaged during paving, the replicate-instrumented slabs could be used for testing.

Concrete overlay thicknesses are also a concern. Slight variations in concrete thickness have substantial impacts on the strains measured in the test slabs. To address this issue, impulse radar was used to nondestructively determine the thicknesses of the slabs in each test section. The impulse radar data can be used in conjunction with a limited number of cores to accurately estimate the thickness of each test slab with a high degree of reliability.

SUMMARY

In 1996 the Colorado Department of Transportation (CDOT) sponsored a research project that developed the current CDOT mechanistic design procedure for TWT pavements. Many variables were considered in construction of the test sections, including concrete overlay thickness, slab dimension, existing asphalt layer thickness, different asphalt surface preparation techniques, and the use of dowel bars and tie bars. Based on the original design procedure development, there are several observations and conclusions regarding using TWT pavements for rehabilitation that should be examined more extensively with supplemental investigations. The items include the subgrade support conditions, the required thickness of asphalt beneath the concrete layer, and the effects of variable joint spacings. Also, this design procedure was regarded as the first generation procedure and need to be verified, calibrated, and maybe revised, as more performance data become available.

The construction of new TWT pavements on SH 121 near Denver, Colorado during summer of 2001 provided an excellent opportunity for accomplishing these objectives. The instrumentation and construction phases of the research project have been completed successfully and are described in this construction report.

ACKNOWLEDGEMENTS

This project was made possible by the sponsorship of the Innovative Pavement Research Foundation (IPRF) and the Colorado Department of Transportation (CDOT). Construction Technology Laboratories, Inc. (CTL) would also like to extend our appreciation to Mr. Ahmad Ardani, Mr. Skip Outcalt, Mr. Kevin Sullivan and many other CDOT personnel for their contributions during the project planning and preparation, instrumentation, construction and load testing activities. We would also like to thank the concrete paving contractor, Interstate Highway Construction, Inc. (IHC) for their cooperation and assistance during the construction and testing activities.

REFERENCES

1. Wu, C.L., Tarr, S.M. Refai, T.M., Nagi, M.N., and Sheehan, M.J., *Development of Ultra-thin Whitetopping Design Procedure*, Report Prepared by Construction Technology Laboratories for the Portland Cement Association (PCA Serial No. 2124), January 1997.

2. Mack, J.W., Wu, C.L., Tarr, S.M., and Refai, T.M., *Model Development and Interim Design Procedure Guidelines for Ultra-thin Whitetopping Pavements*, <u>Proceedings of the sixth International Purdue Conference on Concrete Pavement – Design and Materials for High Performance</u>, Vol. 1, Indianapolis, Indiana, November, 1997, pp. 231-256.

3. *Whitetopping – State of the Practice*, Engineering Bulletin, EB210P, American Concrete Pavement Association, Skokie, Illinois, 1998.

4. Crawley, A.B., and Pepper, J.M., *Application of Fiber Reinforced Concrete for Thin and Ultra-Thin Whitetopping on I-20 in Mississippi*, Paper Presented at the 1999 Transportation Research Board Annual Meeting, Washington, D.C., January 1999.

5. Tarr, S.M., Sheehan, M.J., and Okamoto, P.A., *Guidelines for the Thickness Design of Bonded Whitetopping Pavement in the State of Colorado*, Report No. CDOT-DTD-R-98-10, Colorado Department of Transportation, Denver, Colorado, December 1998.

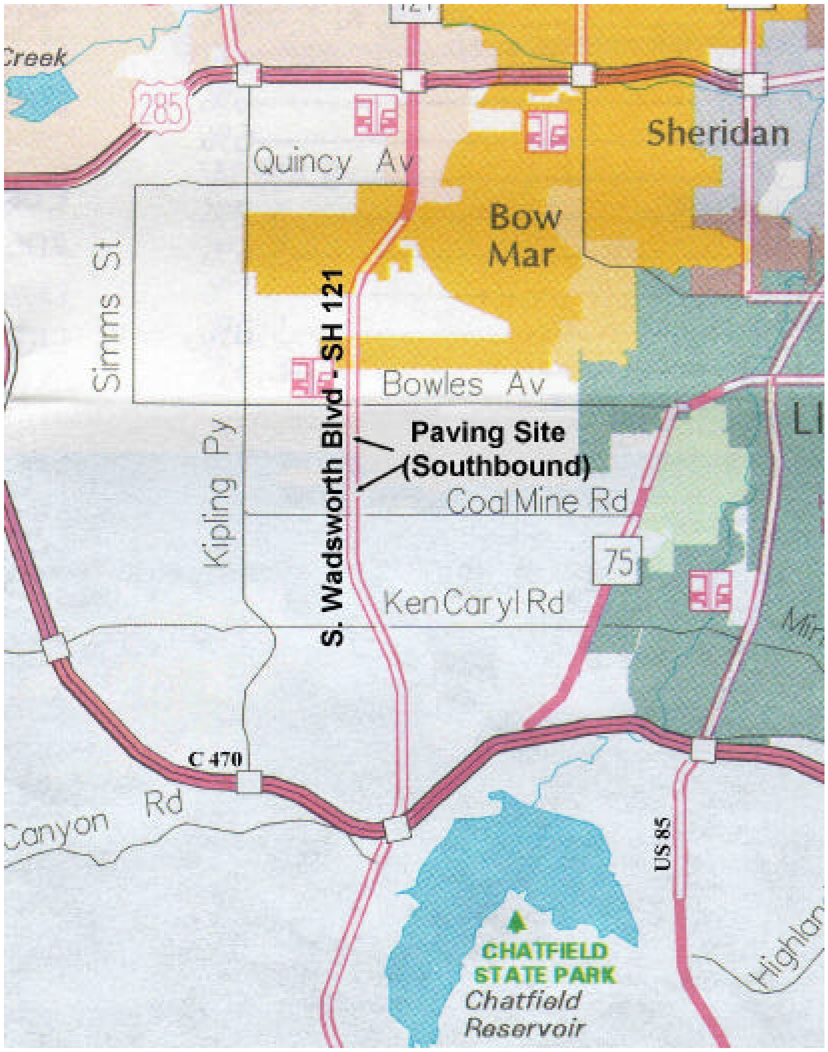

Figure 1 Thin Whitetopping Project Location

FIGURE 2. FATIGUE CRACKING OF TEST SECTIONS 1 AND 2

FIGURE 3. TYPICAL CONDITION OF TEST SECTIONS 3 AND 4

FIGURE 4. RUTTING MEASUREMENT ON EXISTING ASPHALT PAVEMENT

FIGURE 5. CORING ON EXISTING ASPHALT PAVEMENT

Figure 6. FWD Testing on the Existing Asphalt Pavement

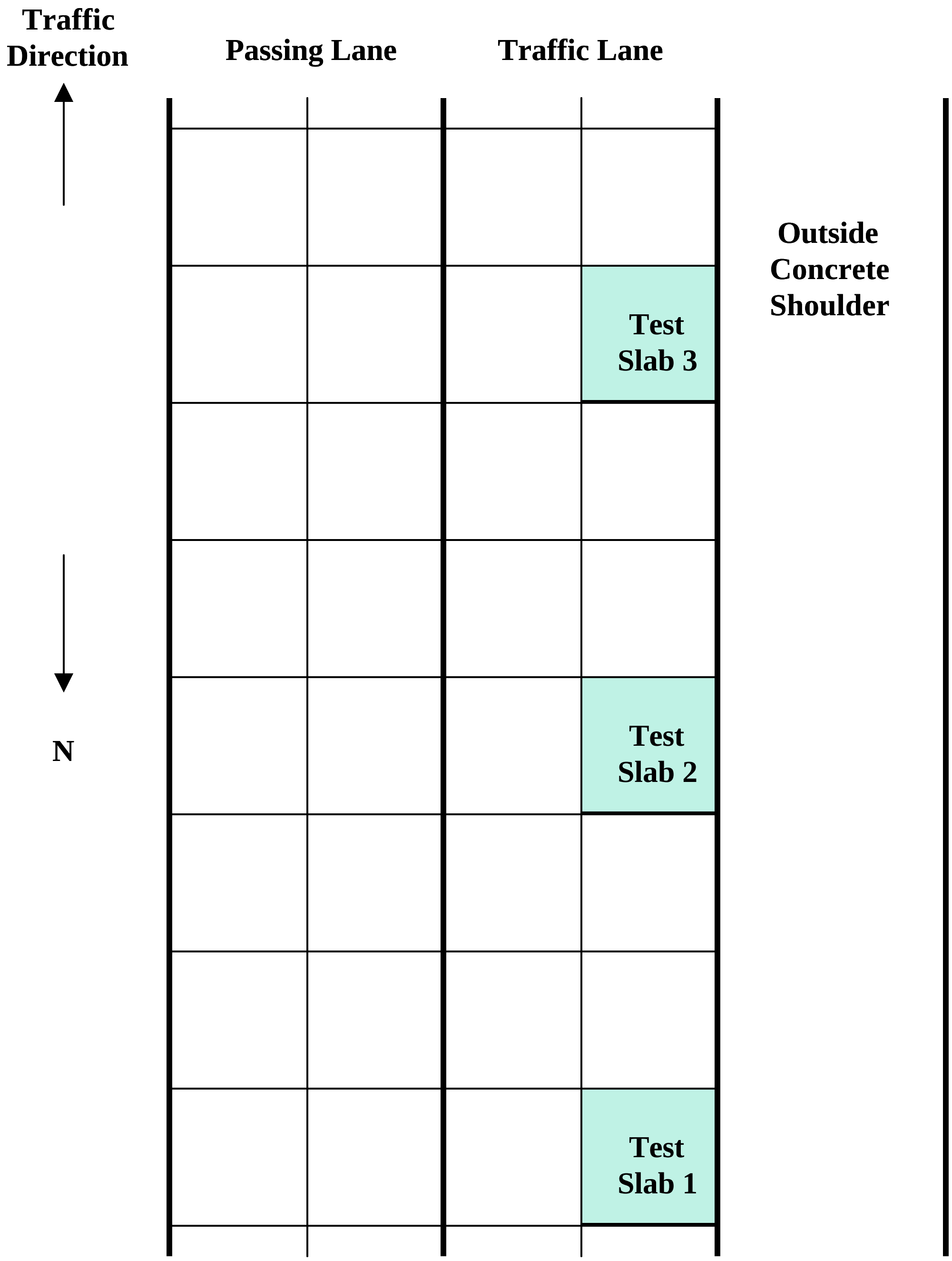

Figure 7. Typical Layout of Test Slabs Within Each Test Section

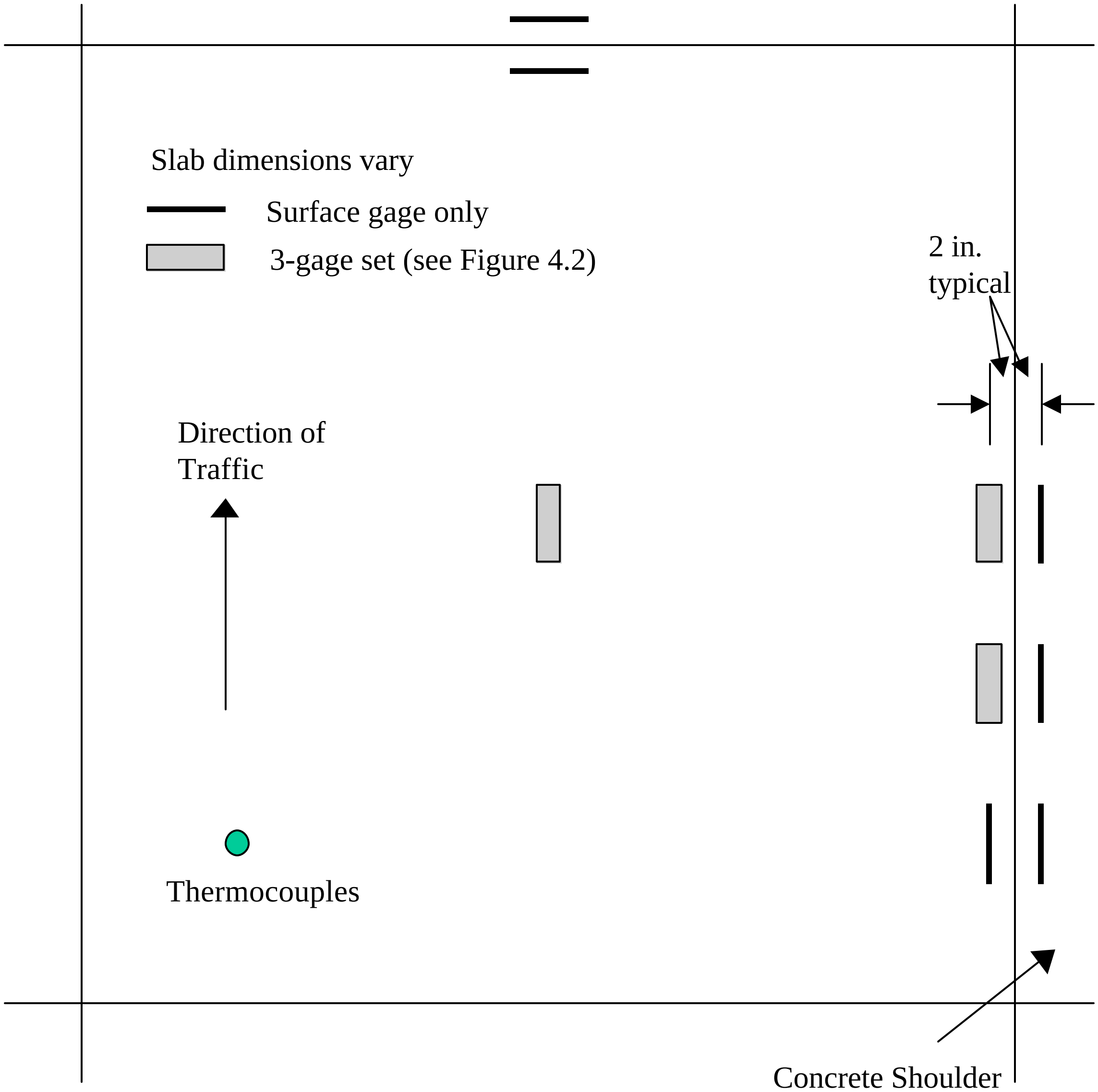

Figure 8 Typical Test Slab Strain Gage Layout -- Plan View

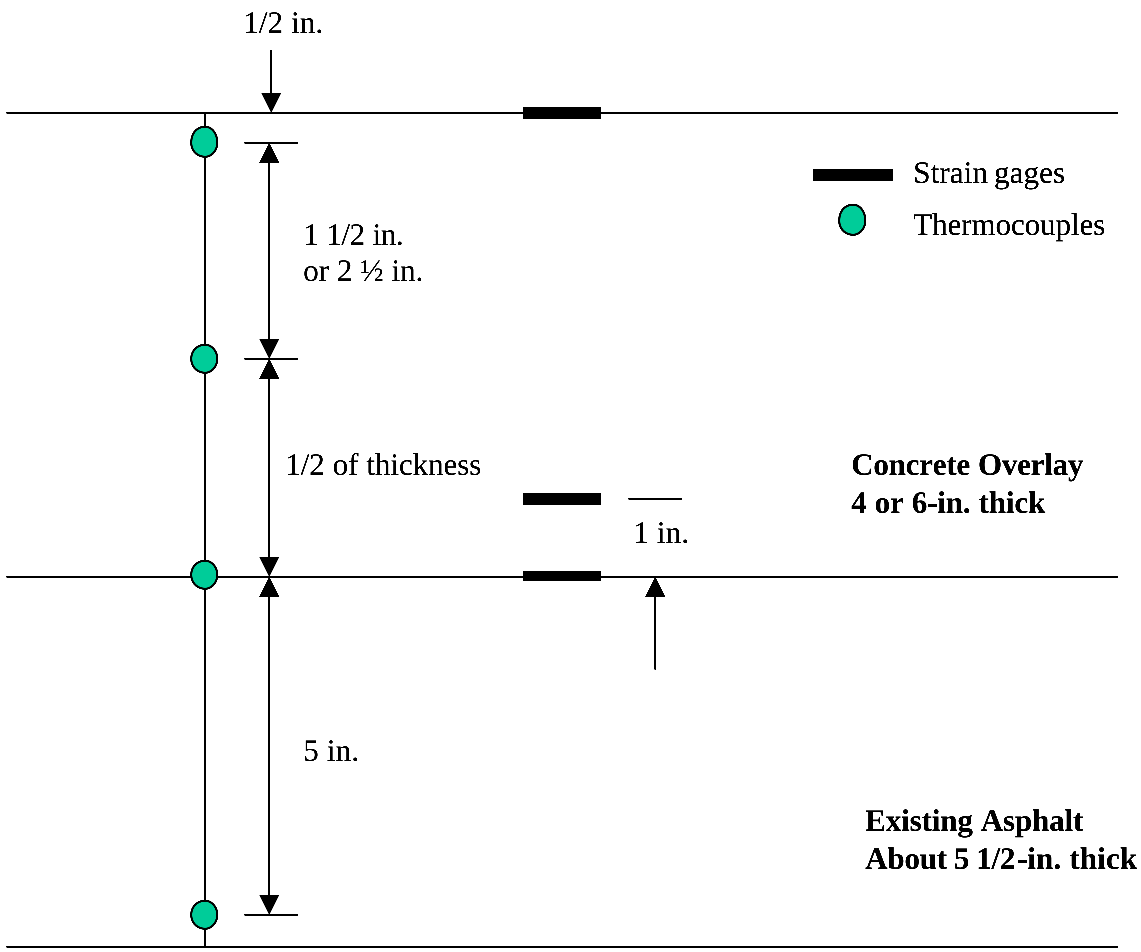

Figure 9 Typical Test Slab Layout -- Section View

Figure 10 Identification and Marking of Test Slab and Gage Locations

Figure 11 Setting Up Reference Points for Slab and Gage Locations

(a) Cutting groove using a diamond grinder

(b) Drilling holes into asphalt layer for embedded concrete gage installation

Figure 12. Asphalt Surface Preparation for Gage Installation

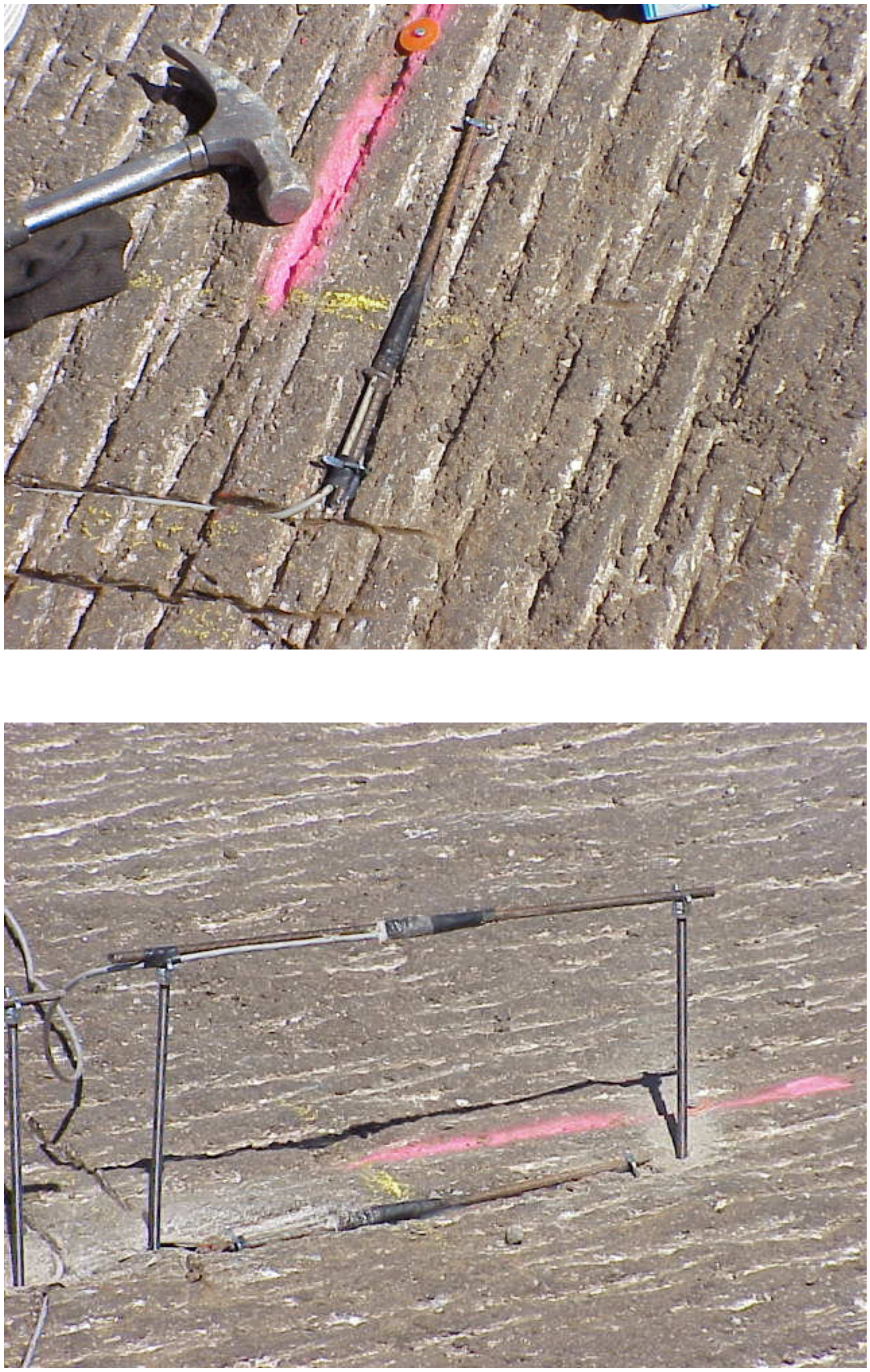

Figure 13. Installation of Embedded Strain Gages

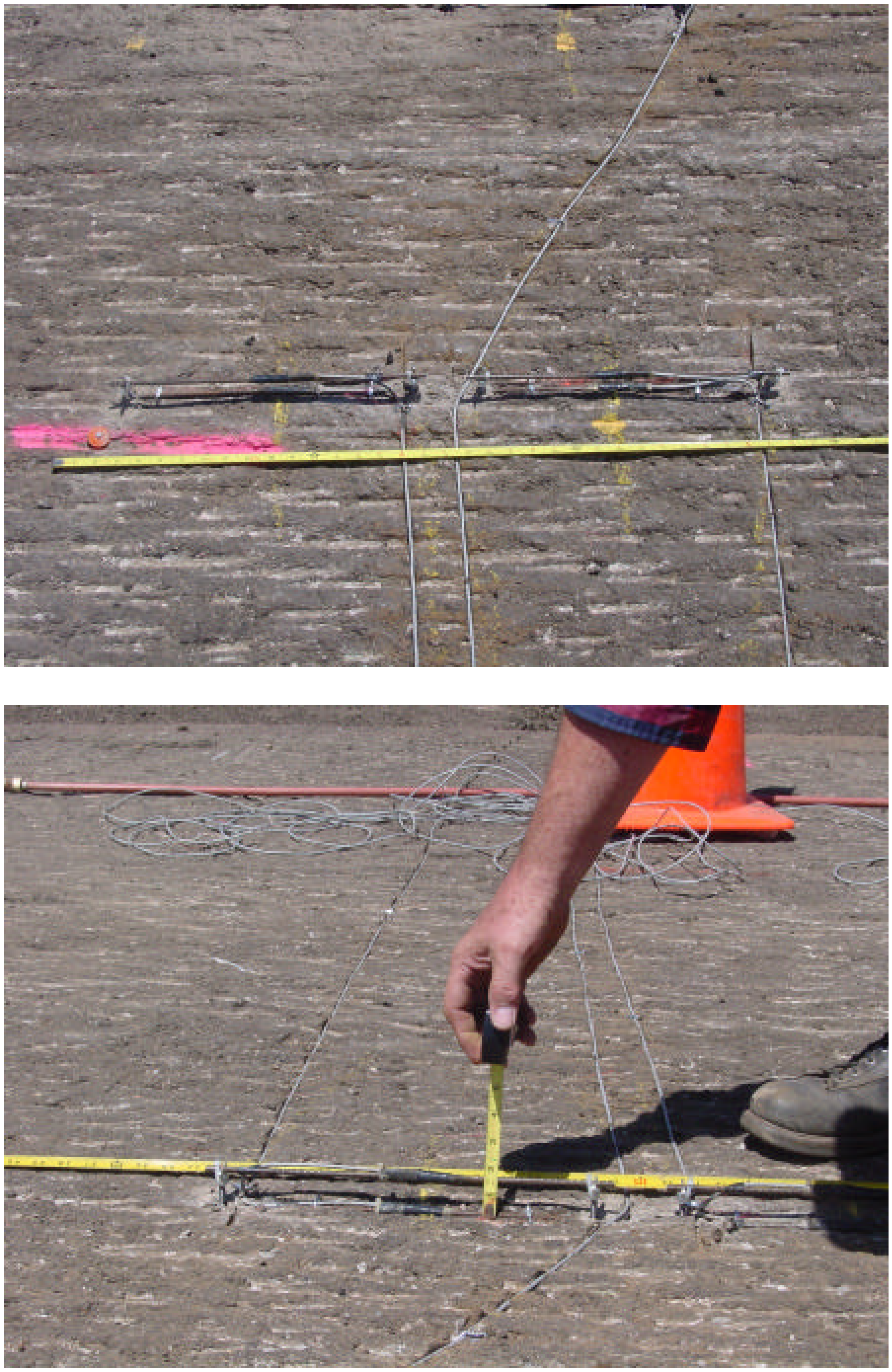

Figure 14. Embedded Strain Gages

Figure 15. Recess and Protection of Lead Wires

Figure 16. Installation of Reference Rods

Figure 17. Installation of Reference Rods, continued

Figure 18. Completed Reference Rod Installation

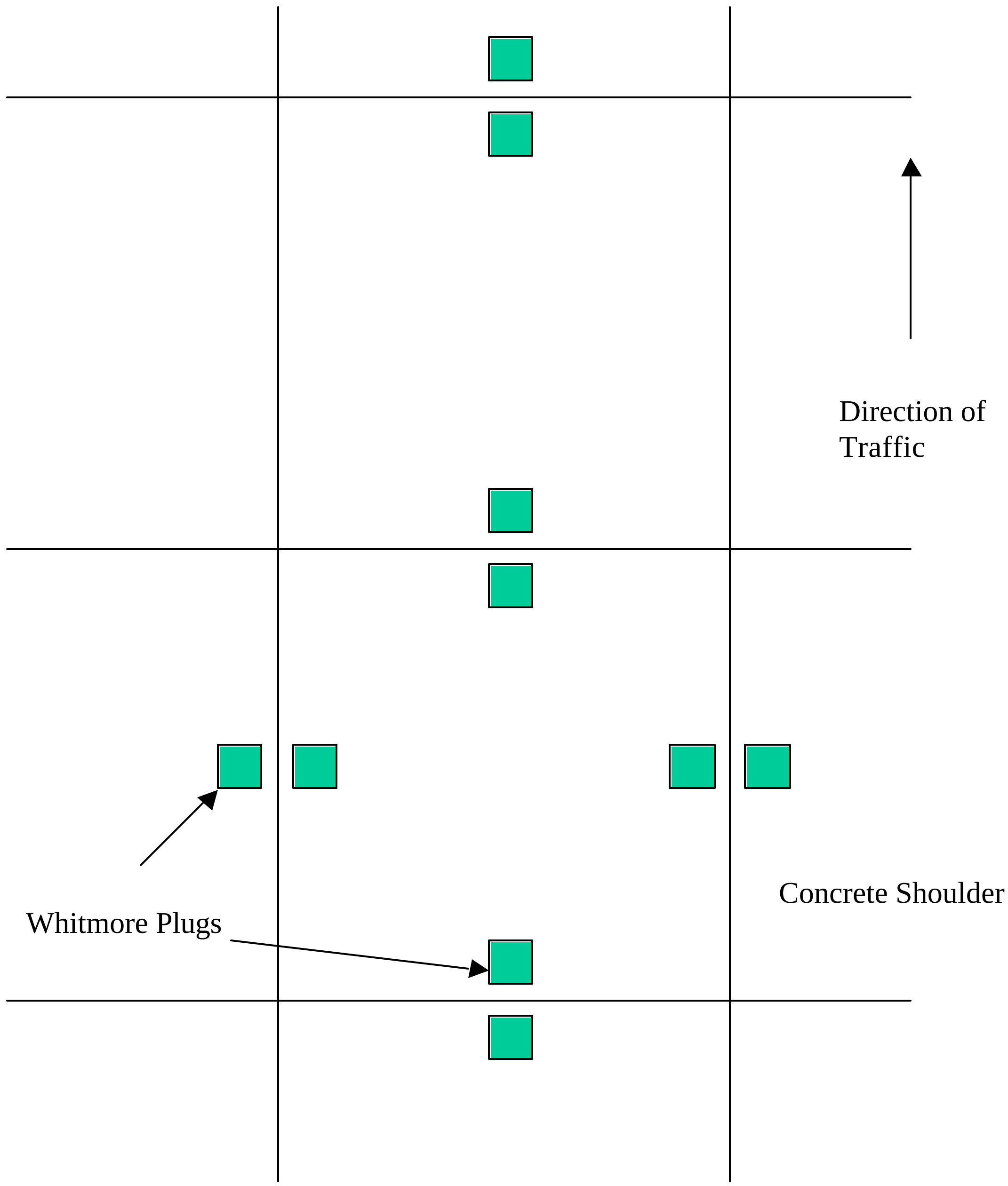

Figure 19 Typical Whitmore Plug Positions for Each Experimental Combination

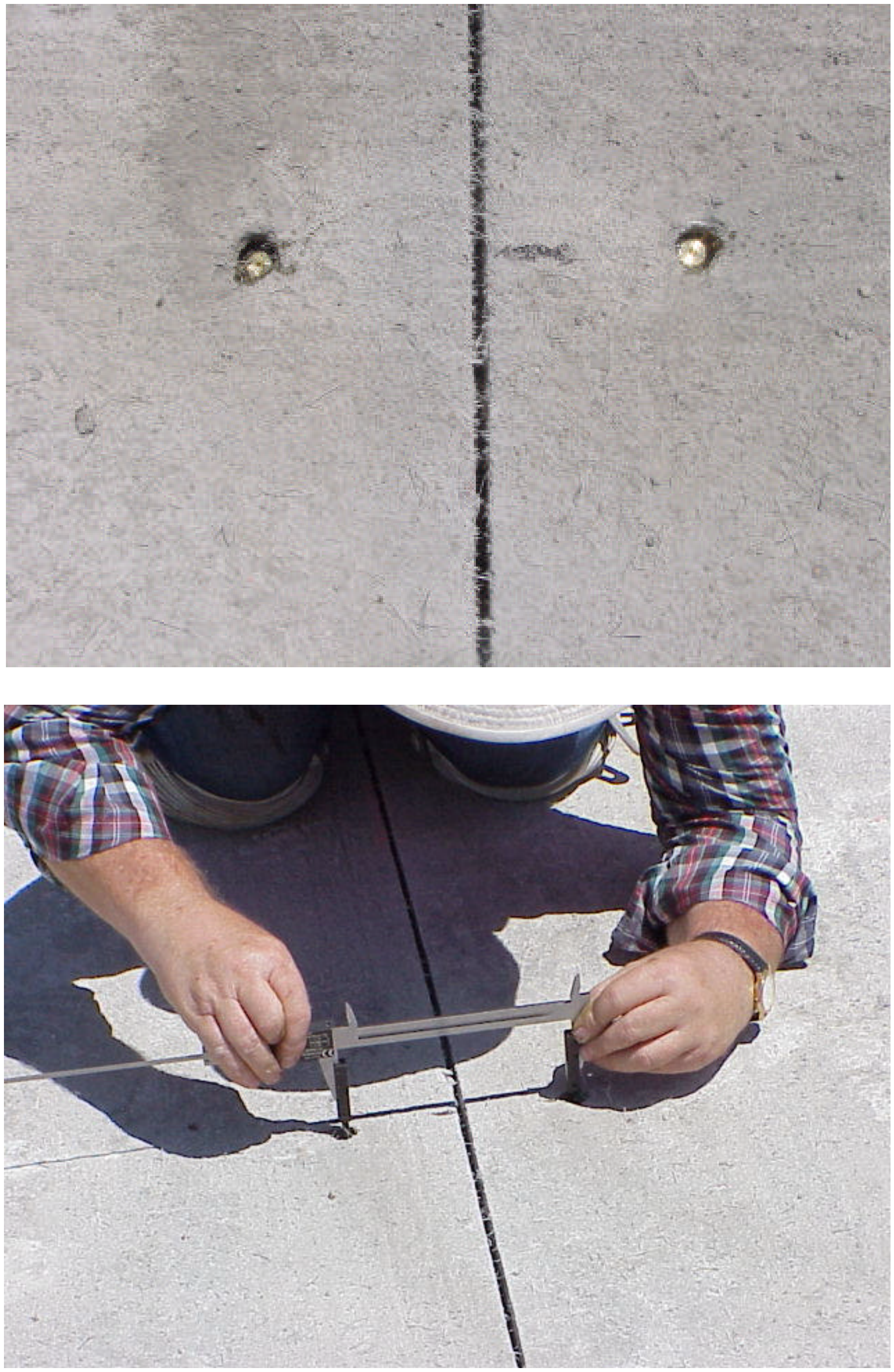

FIGURE 20 WHITMORE PLUGS AND INITIAL MEASUREMENT

Figure 21. Determination of Surface Gage Locations from the Reference Points

Figure 22. Recessed Slots on Concrete Surface for Gage Installation

Figure 23. Attaching Strain Gages using Fast-Setting Epoxy

Figure 24. Cutting Grooves Along Joints for Gage Cables

Figure 25. Soldering Leads to Installed Surface Gages

Figure 26. Recessing Leads

Figure 27. Checking Installed Gages

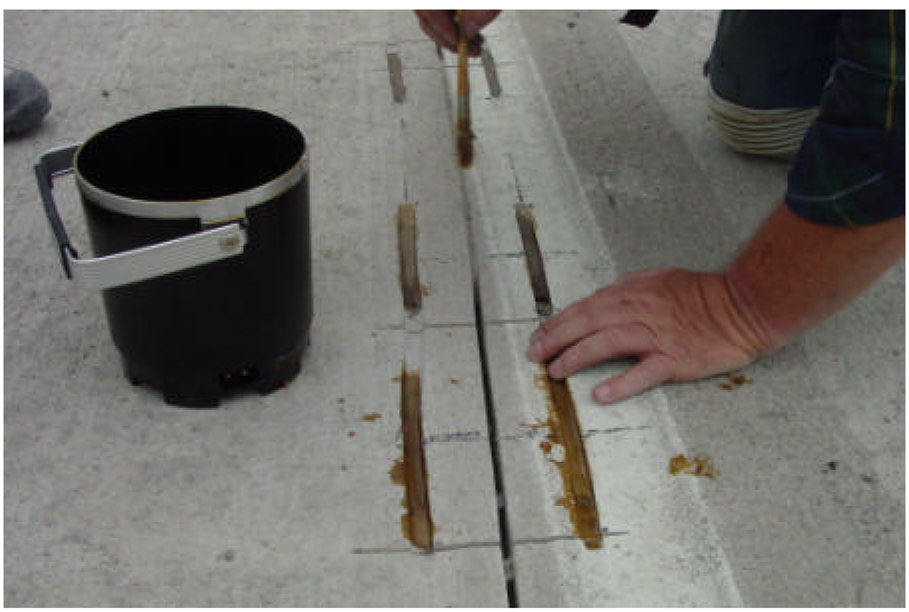

Figure 28. Applying Wax for Protection

Figure 29. A typical Test Slab with Installed Surface Gages

Figure 30. Milling of Existing Asphalt Pavement

Figure 31. Air Blasting Asphalt Surface Prior to Concrete Placement

Figure 32. General View of the Paving Operation

Figure 33. Tie Bar Assembly for Test Section 1

Figure 34. Protection of Instrumentation Ahead of Concrete Paving

Figure 35. Surface Texture Provided by Astroturf Drag

Figure 36. Sawing Transverse Control Joints

Figure 37. Sawing Longitudinal Control Joints

Figure 38. Pavement Surface After Joint Sawing

Figure 39. A Reference Rod After Concrete Paving