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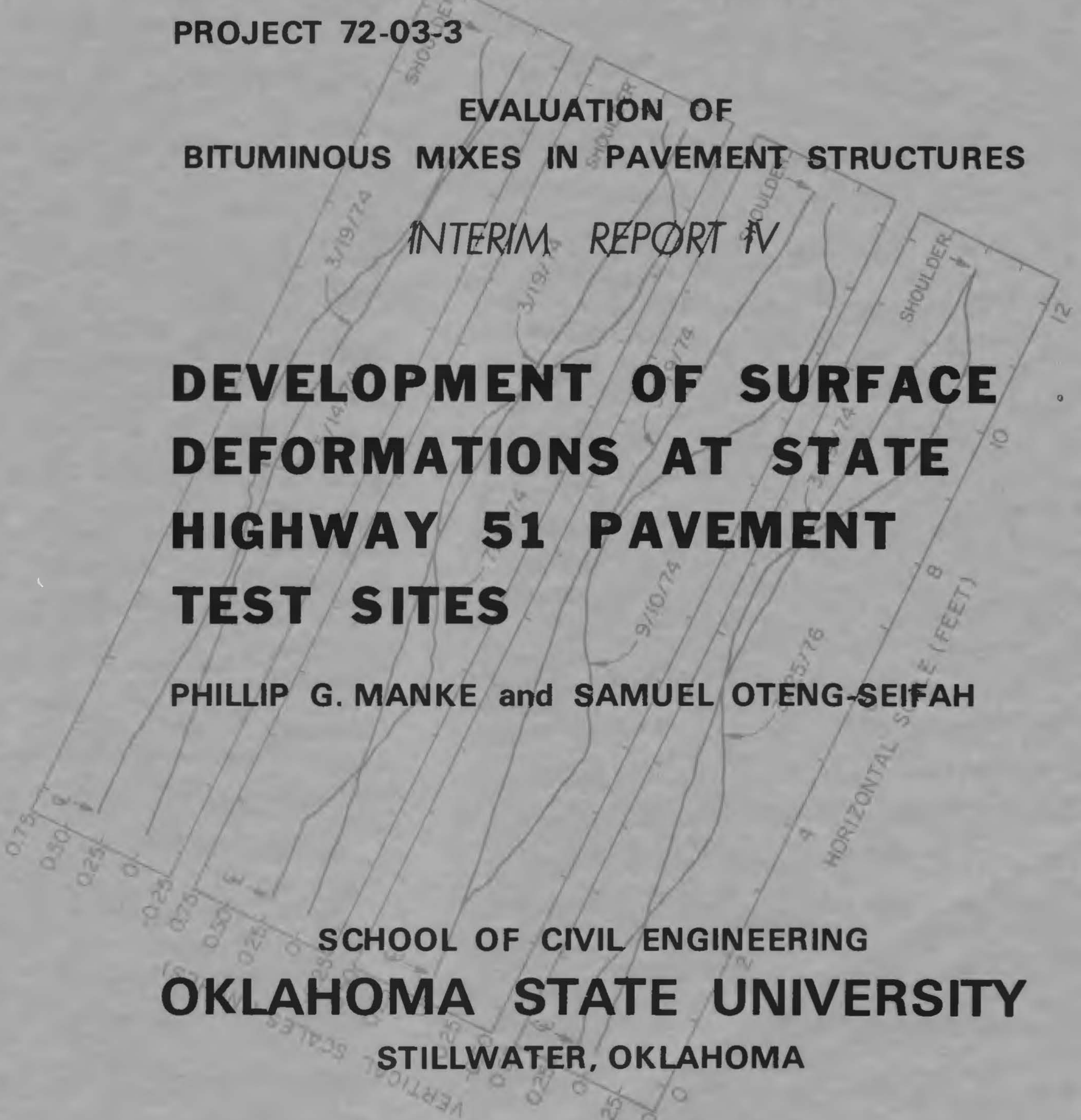
EVALUATION OF
BITUMINOUS MIXES IN PAVEMENT STRUCTURES

INTERIM REPORT IV



DEVELOPMENT OF SURFACE DEFORMATIONS AT STATE HIGHWAY 51 PAVEMENT TEST SITES

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16. Abstract

The development of rutting and other deformations in the pavement surface of two sites on a new section of State Highway 51 was monitored for a two year period. Transverse profile tracings, core samples, surface nuclear densities and stereophotographs were obtained at these sites.

Ruts developed rapidly during the first six months of traffic exposure with maximum rut depths occurring at approximately nine months. Subsequently a gradual reduction in rut depth was observed. Post-construction densification was found in each of the asphalt bound layers at the sites and occurred at both wheelpath and non-wheelpath locations. Increases of up to 12 percent in percent density values of the surface course were observed. Both heave and subsidence of surface elevation points were observed. Surface wear in the wheelpaths was minimal. Based on the analysis procedure used, approximately 50 percent of the maximum rut depth was attributed to densification and the remainder to lateral displacement or instability of the asphalt bound pavement layers.

17. Key Words
flexible pavements, rut depth measurement, transverse profile gage, post-construction densification, lateral creep, instability, surface wear

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Research Project 72-03-3
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CHAPTER I

INTRODUCTION

The characteristics of rutting deformations on high quality bituminous highway pavements have been reported previously (1). During this study on Interstate Highway pavements in Oklahoma, it became evident that an initial transverse profile of the pavement surface at a given location was needed to provide a "true" datum for comparing subsequent profiles and determining rut depths. This information was impossible to obtain since the pavements studied ranged from 36 to 169 months in age. However, an opportunity was presented in March, 1974, to obtain initial profiles and follow closely the development of ruts on a newly constructed pavement section.

At this time, a section of State Highway 51 on new alignment was completed northeast of Oilton, Oklahoma. Permission was obtained to select and monitor three study sites on this new section. Two of these locations were on natural or in situ subgrade materials and the third was to be located on a section having a lime modified subgrade. Due to time constraints, this latter location was not included in the performance study. Transverse profile tracings, core samples, nuclear densities and stereophotographs were obtained at the two selected study sites prior to opening this section of highway to traffic. Similar data was obtained at these sites periodically over a two year period.

A short preliminary report on the performance of the pavement at these two State Highway 51 test sites was submitted on June 24, 1974. This report presents and discusses the results of the investigations at these locations during the total study period from March 19, 1974 to March 25, 1976.

Test Site

The two test site locations on State Highway 51 were identified as SH 61-2 and SH 51-3 and were located approximately 3 miles east and 7 miles east, respectively, from the junction of State Highways 51 and 99, just north of Oilton. The pavement section at these locations consisted of a 9.0 in. thickness of hot-mix-sand-asphalt (HMSA) base, 3.0 in. of Type A binder mix and 1.5 in. of Type C surfacing mix. Both test sites were on fill sections with SH 51-3 on a sloped tangent and SH 51-2 on a horizontal curve.

The layout of each test site is shown in Figure 1. Points A, B and C were marked with nails driven through pop bottle caps and the line ABC represents the profile line or location for the initial and all subsequent profile traces of the pavement surface at a given test site. Bench marks with assumed elevations were established near each site and the elevations of points A, B and C, relative to the assumed bench mark elevations, were determined.

Points 1, 2, 3, and 4, 5, 6 in Figure 1 correspond to the actual wheelpaths and mid-wheelpath locations in the respective traffic lanes. These points were established initially from the profile traces along a line paralleling the profile line but offset approximately 2 ft in the direction of traffic flow in the lane. Cores of the pavement sections

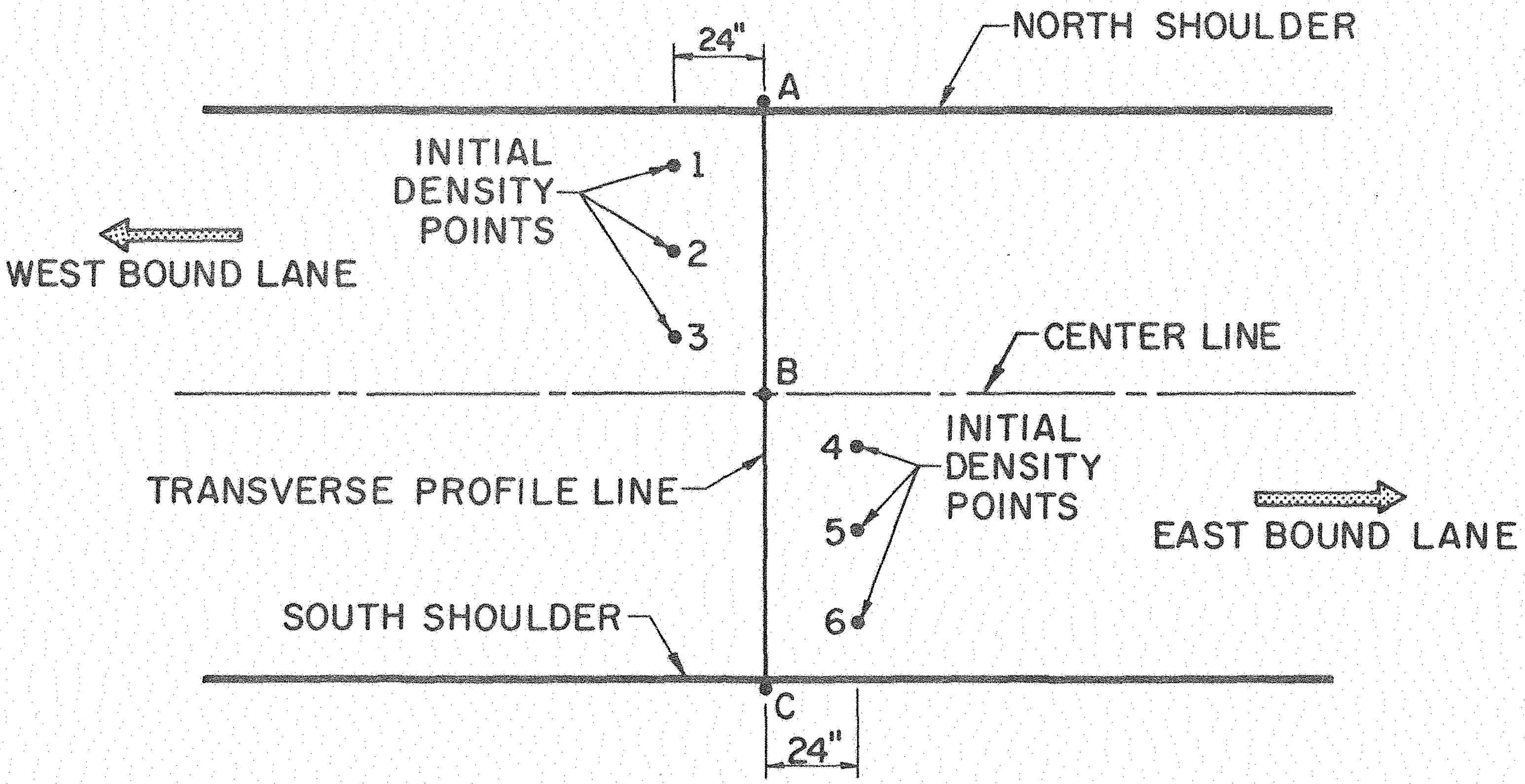

Figure 1. Test Site Layout

were obtained at these points on the offset line. Subsequently, the lateral placement of the wheelpaths and mid-wheelpath points was visually estimated and core obtained at these points along lines offset approximately 18.0 in. in the direction of traffic from the previous core offset line. Four sets of cores were obtained at each test site.

Field and Laboratory Testing

The procedures employed to monitor the performance of the pavement at the test sites included 1) transverse profile tracings, 2) nuclear density measurements, 3) stereo photography, 4) coring operations and laboratory density analyses of the core samples and 5) surface condition rating. These test procedures were identical to those described by Manke and Oteng-Seifah (1).

Nuclear density measurements made at the core points prior to coring were discontinued when poor correlation with the laboratory measured densities was found. The stereophotographs and surface condition rating information aided somewhat in the interpretation of the profile data but were not included in the presentation of results or the discussion.

As previously mentioned, relative elevations of the pavement shoulders and center line at each test site were established initially and checked periodically. Standard differential leveling techniques were employed using a Zeiss self-leveling level and a Philadelphia rod. Arbitrary elevations of 100.00 ft were assigned to each of the established bench marks.

CHAPTER II

TEST RESULTS AND DISCUSSION

Transverse Profile Measurements

Table I shows the maximum rut depths scaled directly from the transverse profile tracings made at various ages of the pavements. Rut depth was measured as the maximum vertical displacement of the surface in a wheelpath from a straight line tangent to the profile curve at points adjacent to the wheelpath. The listed rut depths were obtained for a given lane in the vicinity of the indicated "test point". That is, points 1 and 3 refer to the outer and inner wheelpath locations in the westbound lane and points 4 and 6 the inner and outer wheelpaths in the eastbound lane.

A plot of maximum rut depth versus time for the west bound pavement lane at SH 51-2 is shown in Figure 2. The curves indicate the rate of rutting was greatest during the first 4 to 6 months of the study period with maximum rut depths occurring between the 8th and 10th month. At this time, maximum rut depths of 0.57 in. and 0.41 in. were observed at the outer and inner wheelpaths, respectively. During the remainder of the study period, there was a reduction in rut depth in both wheelpaths of this pavement lane. At the 24th month, these respective rut depths were reduced to 0.46 in. and 0.36 in. Similar behavior was indicated by the rut depth versus time curves for the east bound pavement lane (Figure 3). Maximum rut depths were 0.29 in. and 0.08 in. for the inner and outer

TABLE I

TRANSVERSE PROFILE DATA SH 51 TEST SITES

TEST SITE	TEST PT.	Maximum Rut Depth (Inches)								
		Age (Mo.) 0	2	3	4	6	8	9	12	24
SH 51-2	1	0.0	.250	.321	.446	.500	.554	.571	.500	.464
	2									
	3	0.0	.125	.214	.321	.393	.411	.393	.361	.357
	4	0.0	.125	.156	.214	.275	.275	.286	.250	.250
	5									
	6	0.0	0.000	.000	.063	.063	.078	.078	.063	.063
SH 51-3	1	0.0	.250	.393	.500	.643	.714	.678	.643	.538
	2									
	3	0.0	.357	.482	.625	.696	.714	.714	.696	.696
	4	0.0	.214	.357	.429	.464	.482	.500	.500	.393
	5									
	6	0.0	.214	.286	.357	.411	.464	.482	.446	.357

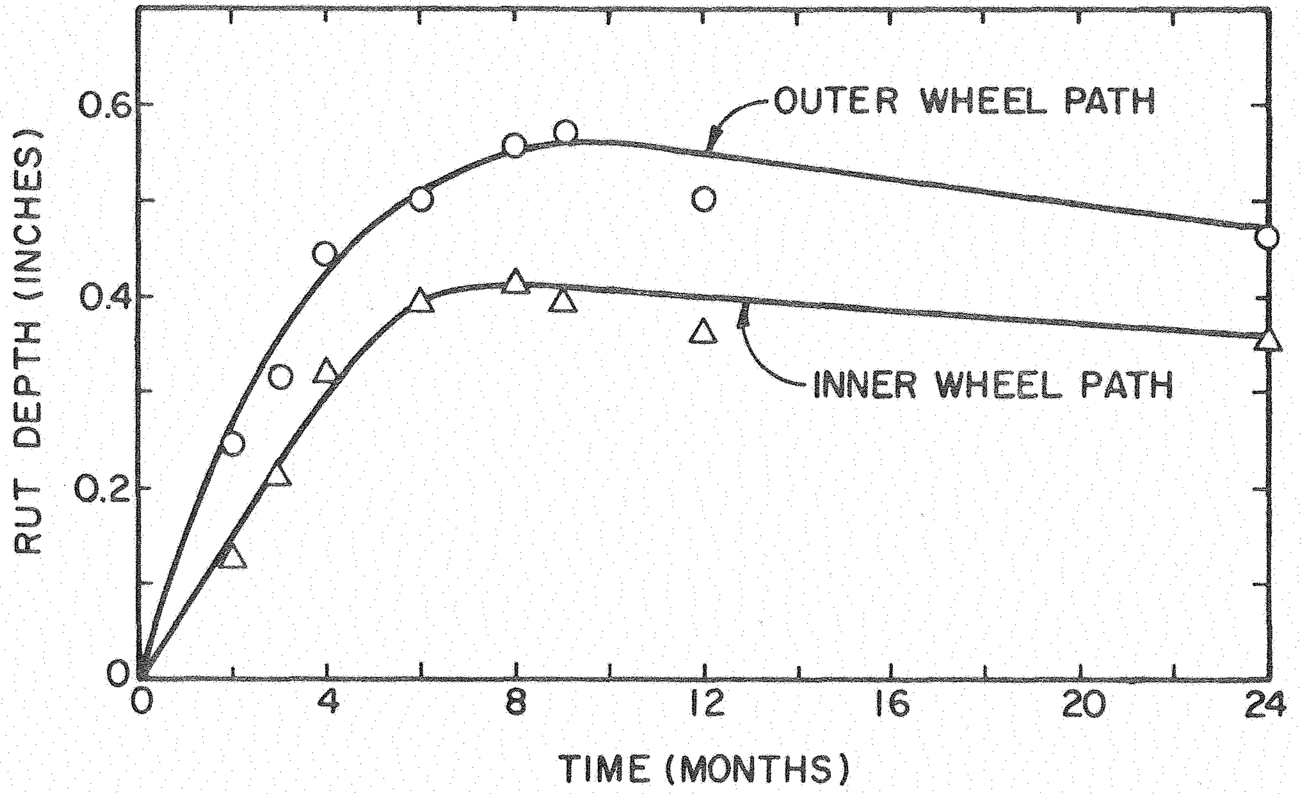

Figure 2. Maximum Rut Depth Vs. Time, SH 51-2, Westbound Lane

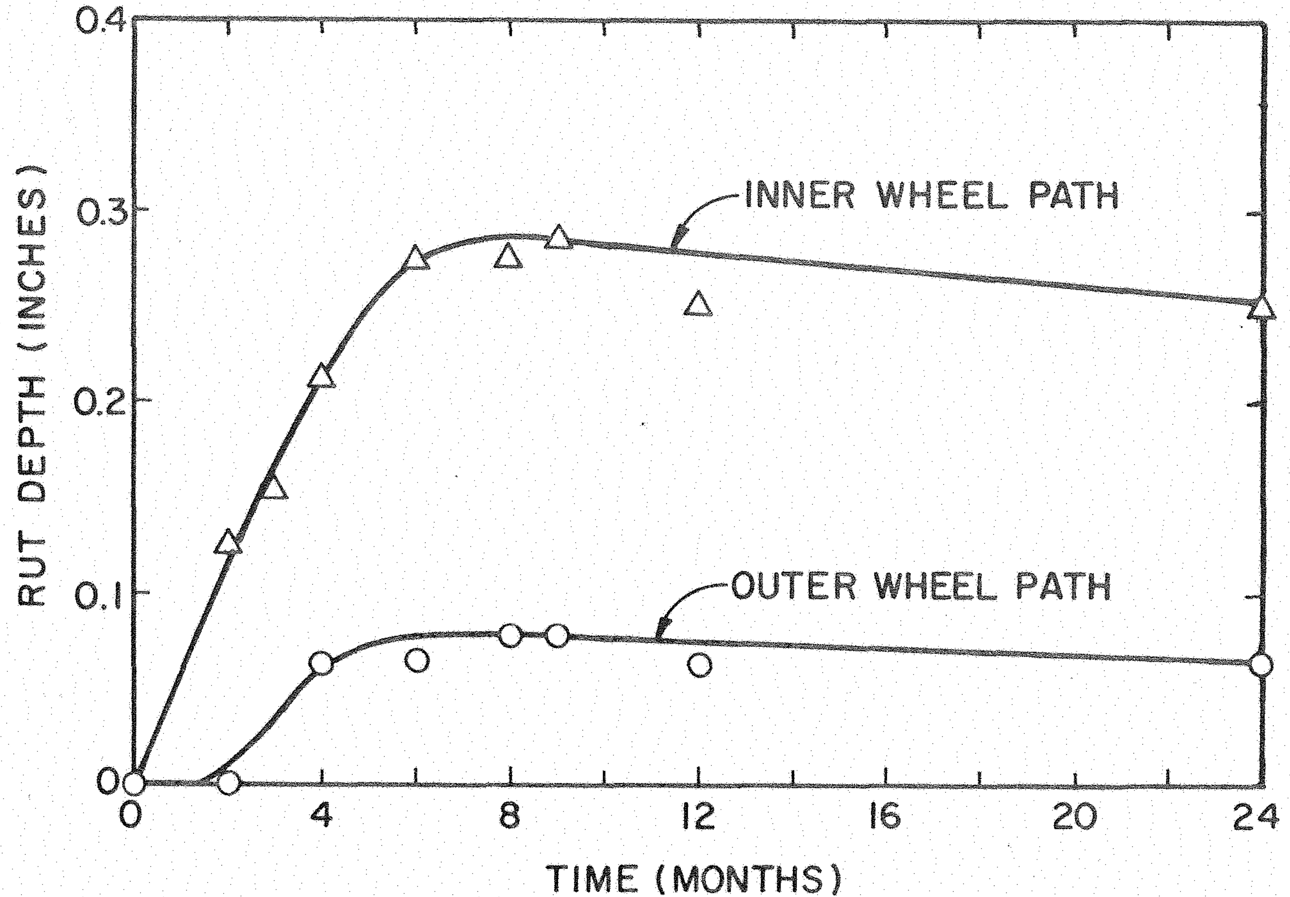

Figure 3. Maximum Rut Depth Vs. Time, SH 51-2, Eastbound Lane

wheelpaths, respectively, and occurred at about the 7th month. The maximum rut depths at these locations were reduced to 0.25 in. and 0.06 in. at the 24th month.

It is interesting to note that the deepest rut in the west bound lane occurred in the outer wheelpath. In the east bound lane and in each of the lanes at SH 51-3, deeper ruts occurred in the inner wheelpaths. Deeper rutting in the inner wheelpaths was also observed at a majority of the Interstate study sites. This reversal in tendency can be explained by considering that the SH 51-2 site was located on a horizontal curve that curved to the left in the west bound direction. Thus, centrifugal force acting on vehicles traversing the curve in this direction (at higher than design speed) increased the weight transmitted by the wheels on the right side of these vehicles. Similar reasoning for vehicles traveling east would explain the relatively shallow rut in the outer wheelpath of the east bound lane at this location.

Figures 4 and 5 indicate the development of rutting at the SH 51-3 test site. The rate of rutting at this location was also greatest during the first six months, with the maximum rut depths of 0.71 in. measured at the 8th month in the west bound lane (Figure 4). In this lane, the maximum measured rut depth was the same in the inner and outer wheelpaths. Again, the inner wheelpath rut remained deeper and at the 24th month the reduction in depth was much less than for the outer wheelpath.

The rut depth curves for the east bound lane (Figure 5) show indentical rut depths for the wheelpaths during the first 2 months. Thereafter, the rate of rutting increased slightly in the inner wheelpath and the depth of rut remained greater through the 24 month period. The

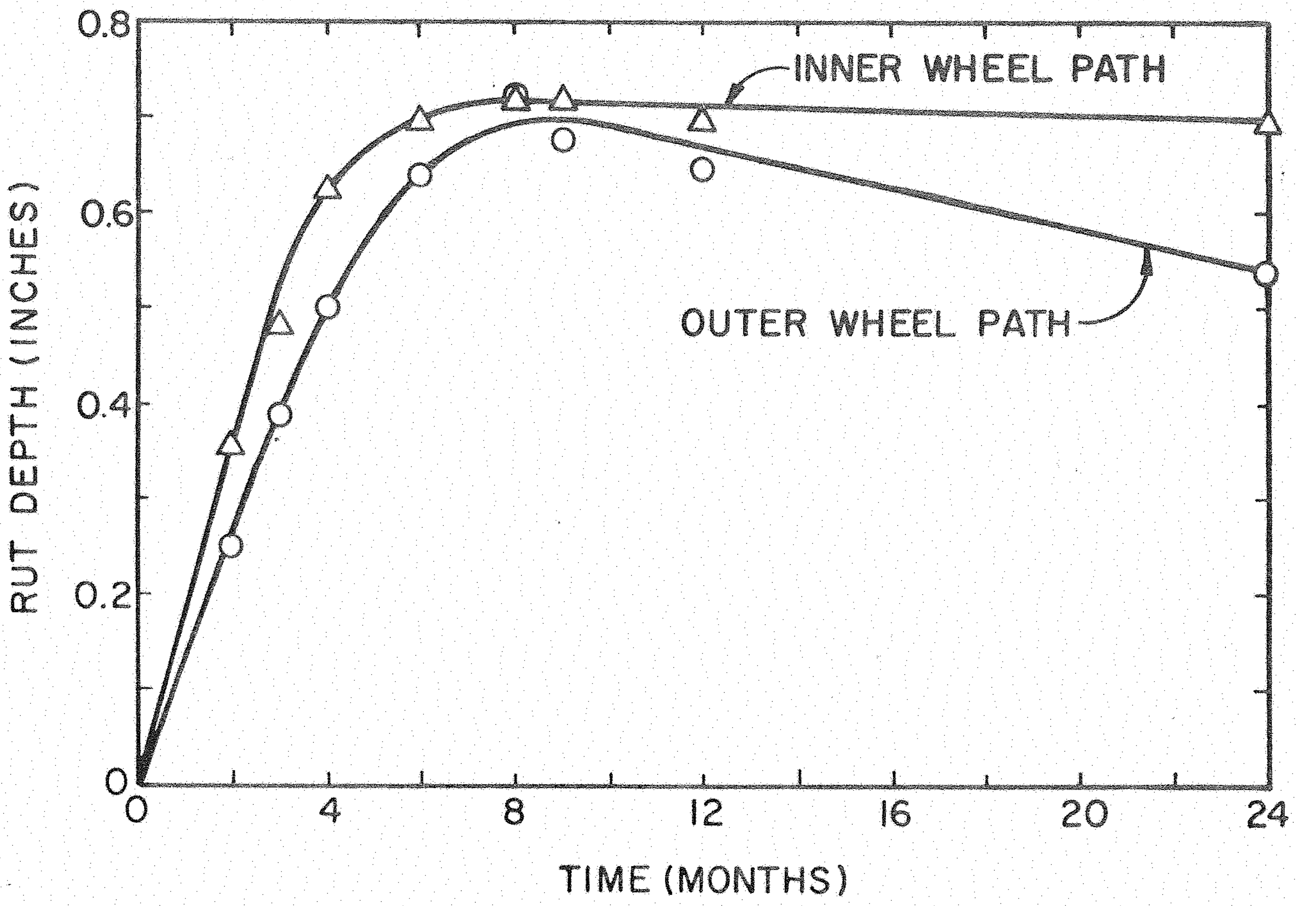

Figure 4. Maximum Rut Depth Vs. Time, SH 51-3, Westbound Lane

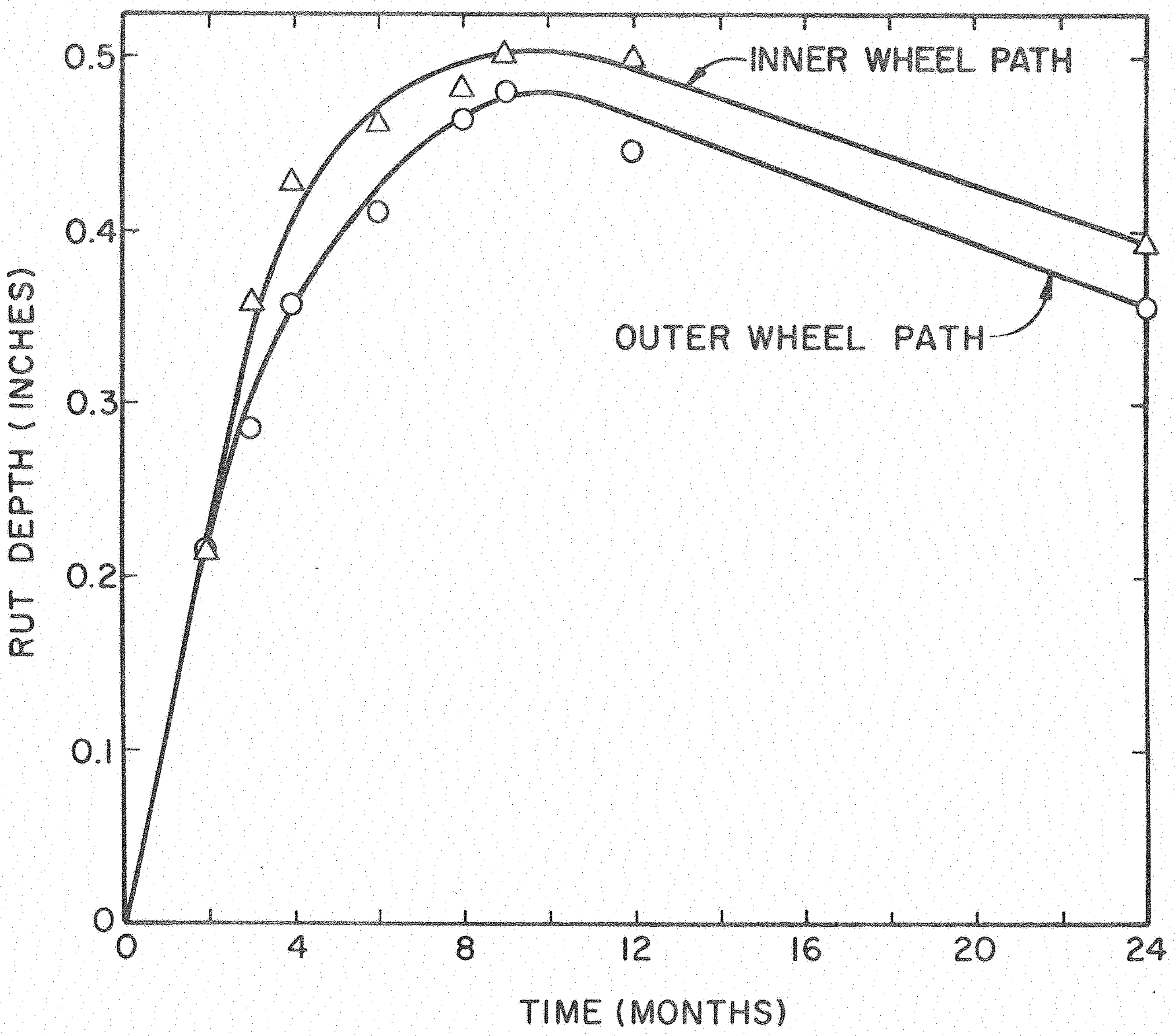

Figure 5. Maximum Rut Depth Vs. Time, SH 51-3, Eastbound Lane

maximum rut depths were measured at the 9th month and these were 0.50 in. and 0.48 in. in the inner and outer wheelpaths, respectively.

These curves for both sites indicate 1) deepest rutting occurred in the west bound lane, 2) highest rate of rutting during the first six months, 3) maximum rut depth achieved between the 8th and 10th month, and 4) a gradual reduction in rut depth after about the 9th month.

The deeper ruts in the west bound lane are indicative of higher traffic volumes and heavier wheel loads in this direction. No traffic counts or load studies were conducted but it is believed that this observation could be easily verified.

The indicated initial high rate of rutting is reasonable, since at time zero both the subgrade and the bituminous bound paving materials are most susceptible to traffic densification and to lateral displacement under traffic loads. As time progresses, post-construction subgrade consolidation and/or densification increases the resistance of the materials to further densification or lateral movement in the wheelpaths. Thus, if the initial density and stability of the pavement layers are low enough, ruts develop rapidly during the first few months of exposure to traffic loading until a maximum depth is attained.

Subsequently, a gradual reduction in rut depth takes place. Relative vertical translation of the pavement can contribute to this reduction. That is, increases in moisture content may result in swelling of clay subgrade materials and consequently an uplift of the pavement surface. Such an uplift or heave and the flexibility of the bituminous materials (ability to conform to variations in subgrade elevations), tends to subdue the depth of rut in the wheelpaths. The major cause of this reduction, however, is attributed to the tendency of the motorists to shift the path of their vehicles transversely to avoid driving in deep ruts.

This lateral translation of the vehicle wheels creates a widening of the wheelpath depressions in the pavement surface as additional densification or displacement occurs adjacent to the location of maximum rut depth. The resulting subjection or flattening of the transverse curvature in the pavement surface reduces the measured depth of rut. Due to higher densities, increased confining pressures, and asphalt hardening, the maximum rut depth versus time curves should reach a minimum value at some future time in the service life of the pavement and remain fairly constant thereafter.

Relative Elevation Measurements

Table II presents the relative elevation data determined at the two test sites. Elevation point A correspond to the north shoulder, point B the center and point C the south shoulder on the profile line established at each test site (see Figure 1). Bench marks were established in the vicinity of the respective test sites and assigned arbitrary elevations of 100.00 ft. Standard differential leveling procedures were used to determine the initial or control elevations (to the nearest 0.01 of a foot) of these points and for subsequent elevation checks that were made.

Elevation checks were made at intervals of 1, 2, 4 and 6 months. Unfortunately, these checks were discontinued after the 6th month and not made again until the end of the second year of the study period. Additional checks during the intervening 1½ years might have confirmed the possibility of cyclic changes in pavement surface elevation.

TABLE II

RELATIVE ELEVATION DATA SH 51 TEST SITES

TEST SITE	SH 51-2 BM=100.00'(Assumed)			SH 51-3 BM=100.00'(Assumed)		
Elevation Point	A	B	C	A	B	C
Control Elevation 3/19/74	102.96	102.49	102.04	104.40	104.55	104.41
Elevation Check 5/14/74	102.96	102.49	102.04	104.42	104.58	104.43
Difference (ft)[1]	0.00	0.00	0.00	- 0.02	- 0.03	- 0.02
Elevation Check 7/11/74	102.96	102.49	102.04	104.40	104.57	104.42
Difference (ft)	0.00	0.00	0.00	0.00	- 0.02	- 0.01
Elevation Check 9/10/74	102.95	102.46	102.00	104.38	104.55	104.40
Difference (ft)[1]	+ 0.01	+ 0.03	+ 0.04	+ 0.02	0.00	+ 0.01
Elevation Check 3/25/76	102.91	102.45	101.99	104.38	104.56	104.41
Difference (ft)	+ 0.05	+ 0.04	+ 0.05	+ 0.02	- 0.01	0.00

[1] (+) values indicate subsidence and (-) values indicate heave.

SH 51-2 Test Site. The data in Table II have been plotted to give a visual conception of the surface elevation changes at this site (Figure 6). Between the 6th and 24th month there may have been some changes greater or smaller than what is indicated by these plots but the trend has been a general and relatively uniform subsidence of the pavement surface. At the end of the 2 year study period, this subsidence amounted to 0.60 in. at the north and south shoulder points and 0.48 in. at the center line point.

The control elevations of the respective points indicate the superelevation on the horizontal curve with the direction of drainage from point A to point C, or towards the inside of the curve. The differential in total subsidence between the shoulders and the center point has created a slight crown. However, this crown is not enough to change the direction of flow or greatly impede the drainage. Since the site is located on a slight fill with a sandy subgrade soil the general subsidence of the surface is attributed to consolidation or settlement of the subgrade.

SH 51-3 Test Site. A plot of the surface elevation changes versus time for this study site are shown in Figure 7. Unlike the other site, the elevation points indicated a surface heave or elevation increase during the first 2 months of pavement life. The greatest amount of heave occurred at the centerline and amounted to 0.36 in. During the next 4 months the center elevation point had returned to the control elevation and the shoulder points showed a change from heave to subsidence. The maximum amount of subsidence occurred at the north shoulder and amounted

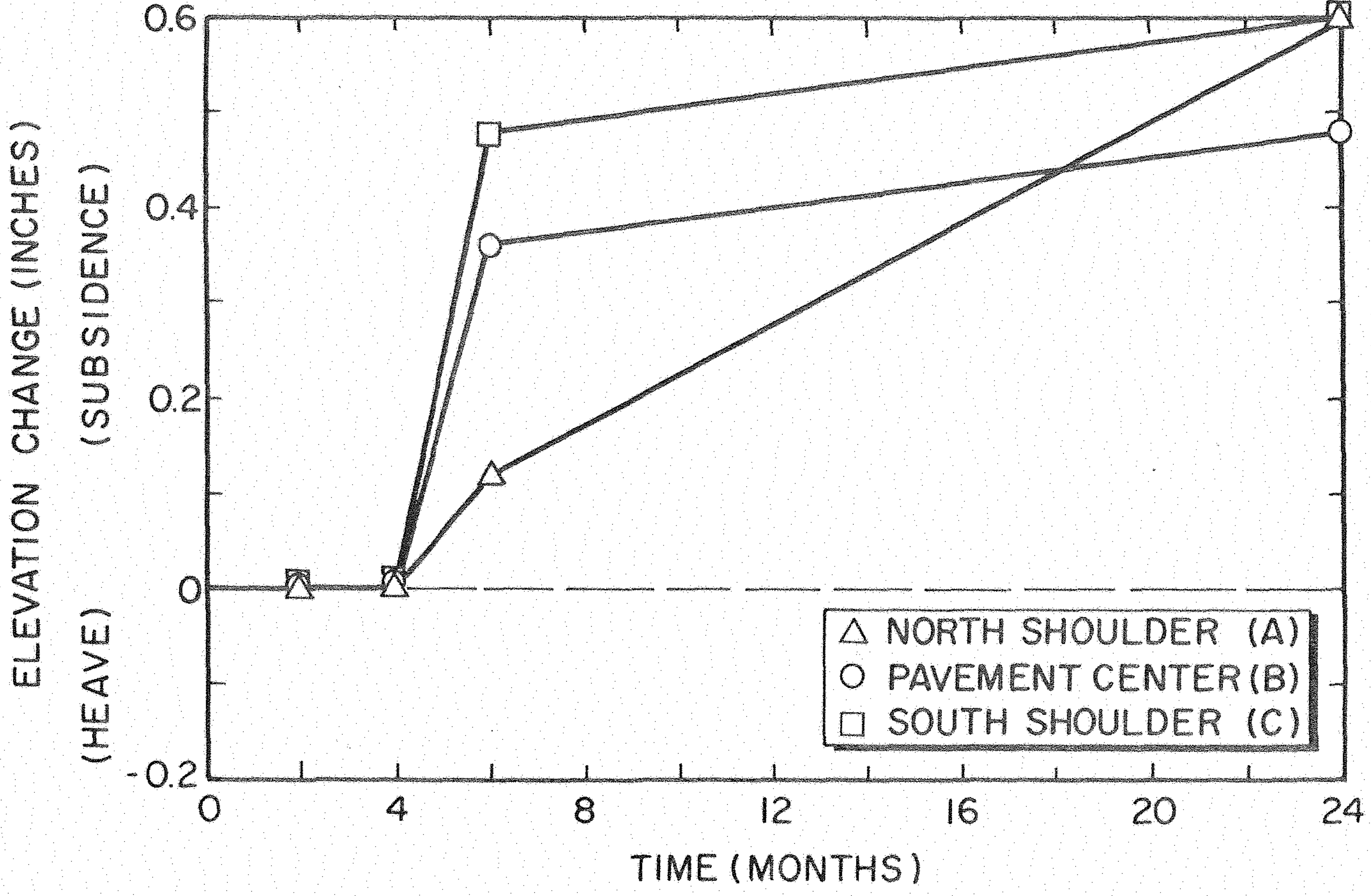

Figure 6. Surface Elevation Change Vs. Time, SH 51-2, Fill Section

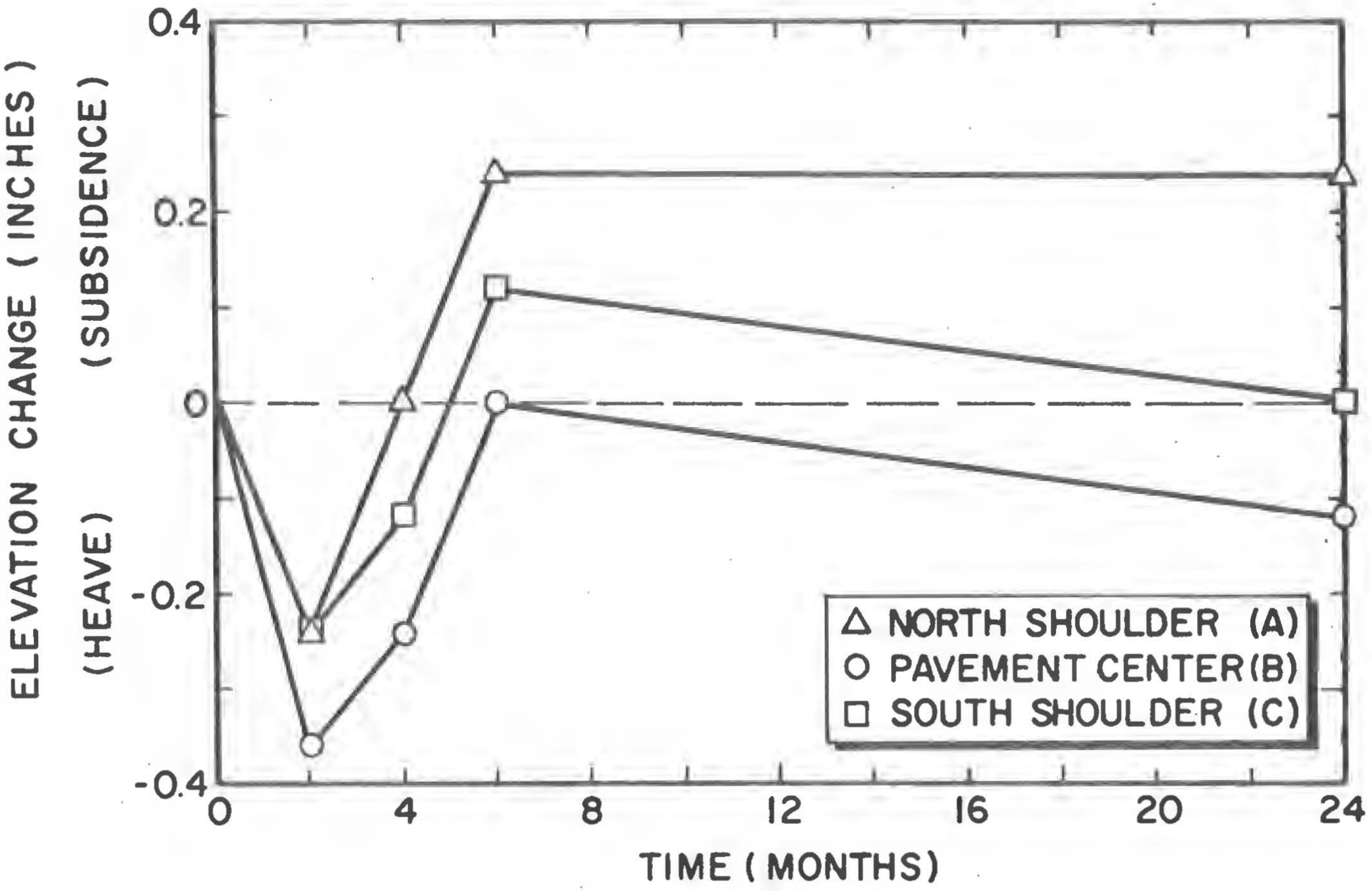

Figure 7. Surface Elevation Change Vs. Time, SH 51-3, Fill Section

to 0.24 in. At the end of the study period, the vertical translations of these points had caused an increase in cross slope on both sides of the centerline.

This site was also located on a fill section. However, in this case, the subgrade fill material appeared to have a high clay content and was probably obtained from cut sections to the east or west of the site location. Between the 6th and 24th month, there may have been cyclic changes in the elevations of the surface points due to seasonal moisture variations and the resultant shrink and swell of this subgrade soil.

These changes in elevation of the pavement surface complicated the study and determination of the cumulative surface deformations under traffic. For example, uniform or differential settlement and heave of the pavement completely obscured any surface indications of lateral displacement of the bituminous bound materials.

Deviations from Original Profile. To study the effects of surface elevation changes, a comparison of the transverse profile tracings, i.e., the initial and subsequent tracings, in each lane of the two study sites was made. The profile tracing of a lane on a given date was superimposed on the initial tracing for that lane and the shoulder and centerline points adjusted to the indicated vertical translations. The mid-point of the lane and points 3 ft either side of this mid-point (according to the tracing scale) were selected and vertical deviations of these points from the initial tracing were scaled. This information is presented in tabular form in Table III. Since elevation data was not available for all the test dates, reasonable assumptions of the translations of the surface elevation points at these times were made.

TABLE III

DEVIATIONS FROM INITIAL PROFILE SH 51 TEST SITES

TEST SITE	TRANSVERSE POINT	VERTICAL DEVIATIONS IN INCHES (+ = Subsidence; - = Heave)								
		Test Date: 3/19/74	5/14/74	6/11/74	7/11/74	9/10/74	11/12/74	12/12/74	2/27/75	3/25/76
		Age(Mo.): 0	2	3[1]	4	6	8[1]	9[1]	12[1]	24
SH 51-2	Shldr. -A	0.000	0.000	0.000	0.000	+0.120	+0.120	+0.360	+0.540	+0.600
	(a)	0.000	+0.321	+0.321	+0.500	+0.750	+0.892	+0.964	+0.982	+1.125
	(b)[2]	0.000	+0.125	0.000	+0.062	+0.321	+0.536	+0.375	+0.392	+0.696
	(c)	0.000	+0.250	+0.268	+0.393	+0.750	+0.892	+0.839	+0.785	+0.982
	C.L. -B	0.000	0.000	0.000	0.000	+0.360	+0.360	+0.360	+0.420	+0.480
	(d)	0.000	+0.250	+0.187	+0.140	+0.625	+0.607	+0.536	+0.446	+0.625
	(e)[2]	0.000	+0.321	+0.286	+0.125	+0.625	+0.661	+0.482	+0.571	+0.607
	(f)	0.000	+0.187	+0.156	+0.046	+0.571	+0.589	+0.464	+0.446	+0.607
	Shldr. -C	0.000	0.000	0.000	0.000	+0.480	+0.480	+0.480	+0.540	+0.600
SH 51-3	Shldr. -A	0.000	-0.240	-0.240	0.000	+0.240	+0.240	+0.240	+0.240	+0.240
	(a)	0.000	+0.078	+0.125	+0.410	+0.857	+0.946	+0.732	+0.750	+0.643
	(b)	0.000	-0.078	-0.250	-0.063	+0.141	+0.156	-0.031	-0.031	+0.047
	(c)	0.000	+0.063	+0.109	+0.375	+0.714	+0.732	+0.607	+0.643	+0.500
	C.L. B	0.000	-0.360	-0.360	-0.240	0.000	0.000	0.000	-0.060	-0.120
	(d)	0.000	-0.125	-0.031	+0.125	+0.339	+0.500	+0.411	+0.250	+0.156
	(e)	0.000	-0.187	-0.156	-0.063	+0.063	+0.203	0.000	-0.250	-0.063
	(f)	0.000	-0.031	+0.047	+0.286	+0.464	+0.625	+0.500	+0.357	+0.268
	Shldr. C	0.000	-0.240	-0.240	-0.120	+0.120	+0.120	+0.120	+0.060	0.000

[1] Vertical translations of elevation points A, B, and C are assumed on these dates.

[2] Points (b) and (e) are assumed centers of traffic lanes. Points (a), (c), (d), and (f) are 3.0 ft either side of the lane centers. Locations of these points were held constant in determining deviations from the initial profile.

Graphical presentations of this information were prepared for the west bound lane at the SH 51-2 site (Figure 8) and the east bound lane at the SH 51-3 site (Figure 9). The surface profiles shown in these figures were traced from the X-Y recorder plots made on the indicated dates. The profiles at four dates subsequent to March 19, 1974, are shown in these figures along with the initial profile. For clarity, the ends or elevation points at the shoulder and centerline of the initial profile were plotted on the horizontal and the corresponding points on the subsequent profiles moved up and down by the amount of vertical translation determined at the respective test date. These plots show the combined effects of the developing ruts and the vertical translations of the pavement surface. They also indicate the dynamic nature of surface deformations on flexible pavements and some of the complications involved in interpretation of profile data.

Density Measurements

When the study of the test sites on State Highway 51 began, it was decided that core samples of the pavement would be taken at intervals of 3, 6, 12, 18 and 24 months in order to determine the effect of traffic loading on layer density. This schedule was maintained through the first year of the study but the exigencies of other project work conducted during the second year restricted the taking and processing of additional core samples. Thus, only four sets of cores were taken at the respective sites and the data for one of these sets was lost as a result of the tornado that severely damaged the Civil Engineering Laboratory building on June 13, 1975. However, it is believed that sufficient data was taken

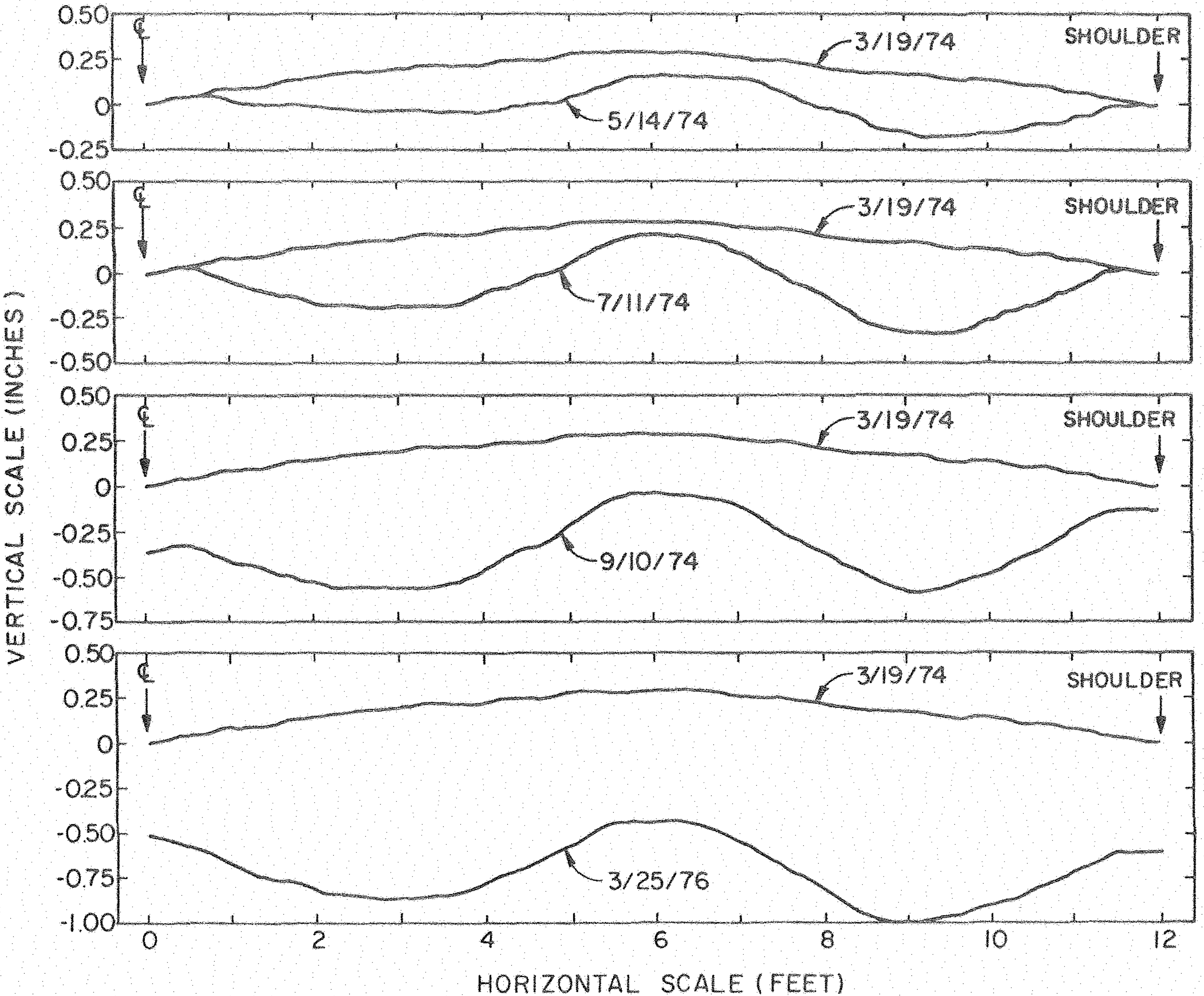

Figure 8. Deviations From Initial Profile, SH 51-2, Westbound Lane

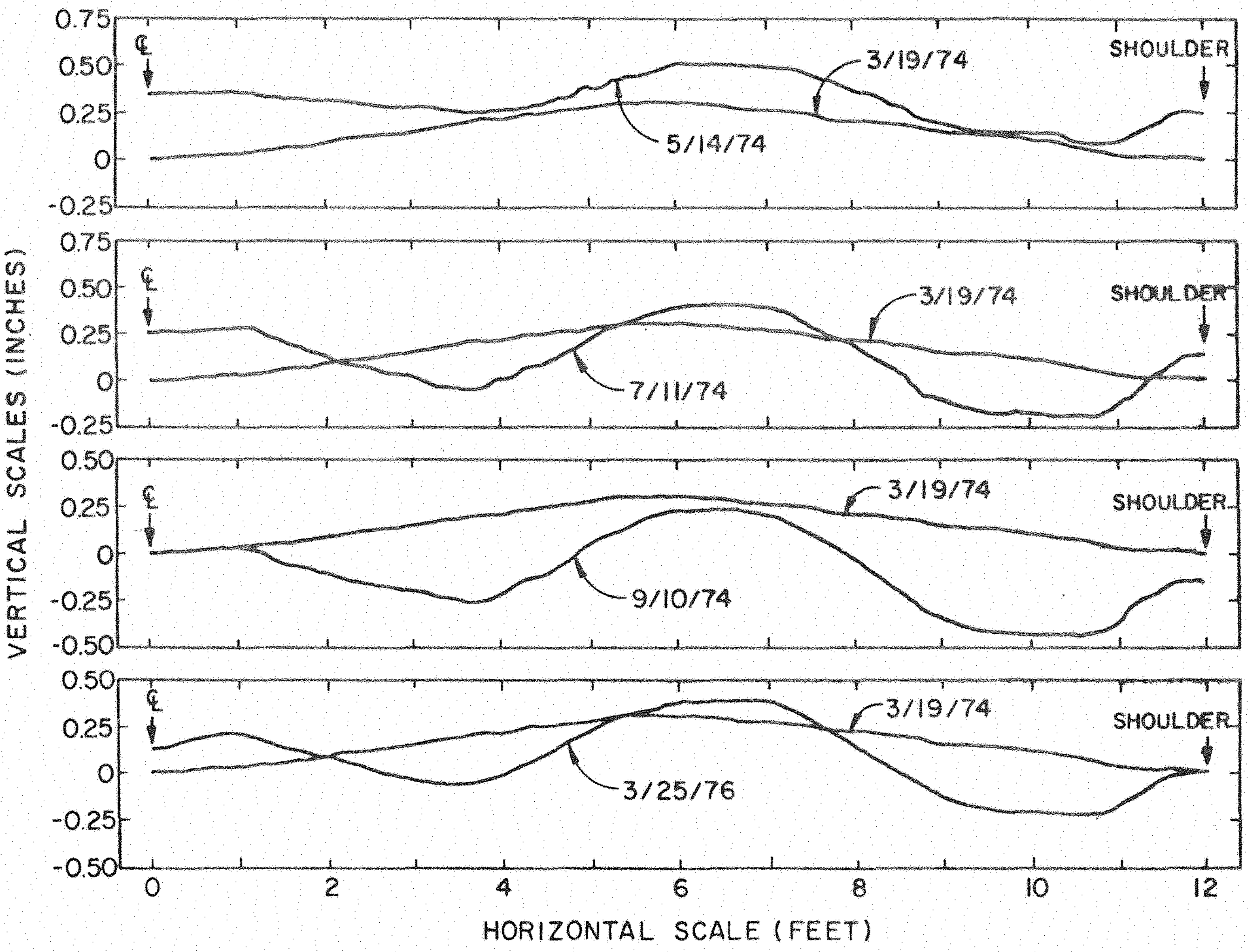

Figure 9. Deviations From Initial Profile, SH 51-3, Eastbound Lane

to show generalized trends and to substantiate the findings on Interstate Highway pavements previously reported (1).

Pavement core samples of the full depth of asphalt-bound materials were obtained from the wheelpath and mid-wheelpath locations in each lane at the test sites. The core or test points were numbered as indicated in Figure 1. These 4.0 in. diameter cores were cut into five segments corresponding to the respective layers, i.e., surface course, leveling course, and top, middle and bottom thirds of the base course. These layers were designated A, B, C, D and E, respectively. Specific gravities of the individual layers were then determined and differences in density noted. The bulk specific gravities of these core segments were obtained using the ASTM method of test, D-1188, and the maximum specific gravities of the mixtures determined using the ASTM method of test, D-2041 (2).

The percent density values for these core samples are listed in Table IV. An analysis of variance (AOV) using the Statistical Analysis System computer program (3) was applied to check for differences in this percent density data regarding test points (wheelpath versus-mid wheelpath locations), layers and time.

<u>SH 51-2 Test Site</u>. The AOV indicated strong evidence of layer, time and "layer-time" differences in the percent density data for this pavement section. The observed significance level was 0.0001 in each case (4). There was a considerable spread in the initial densities of the layers with the leveling course having the highest and the hot-sand base the lowest values. The densities of the respective asphalt bound layers were time-dependent, i.e., the densities increased with time, indicating post-construction densification. For this section, the AOV showed no significant point differences and this indicated relatively uniform

TABLE IV

DENSITY DATA

TEST POINT	LAYER[1]	PERCENT DENSITY VALUES					
		Test Site: SH 51-2			Test Site: SH 51-3		
		3/19/74[2]	9/10/74	2/27/75	3/19/74	9/10/74	2/27/75
1	A	90.98	93.77	94.55	89.80	94.53	94.55
	B	95.33	96.89	98.82	90.76	94.96	96.50
	C	83.19	83.33	85.06	78.62	83.06	83.19
	D	82.38	83.33	83.54	77.41	81.78	82.49
	E	82.04	83.05	82.30	77.86	77.01	78.54
2	A	88.24	93.35	95.73	90.62	92.99	93.39
	B	95.33	95.75	98.82	91.15	93.10	95.72
	C	83.40	83.40	83.27	80.24	82.66	83.52
	D	82.04	84.48	83.95	80.65	82.25	82.84
	E	82.10	82.25	82.86	80.73	79.43	79.44
3	A	88.14	94.53	95.74	88.23	92.66	93.08
	B	95.70	96.15	98.05	90.38	93.51	94.98
	C	81.55	83.40	83.73	80.81	82.73	83.33
	D	81.90	84.08	83.89	78.45	82.52	82.51
	E	81.72	83.87	82.30	79.09	79.01	79.95
4	A	91.63	94.16	95.31	87.05	94.16	94.96
	B	96.14	96.92	98.45	93.06	96.51	96.53
	C	82.37	83.46	83.61	81.14	82.99	83.00
	D	83.60	84.21	84.02	81.14	84.14	83.61
	E	81.30	82.59	82.11	80.73	80.56	80.50
5	A	89.80	93.75	93.82	87.10	91.47	92.25
	B	96.52	98.83	98.05	91.95	93.48	94.02
	C	83.19	83.80	84.08	81.55	82.99	82.52
	D	82.37	83.95	84.08	79.09	81.37	81.22
	E	81.30	82.32	83.27	75.00	80.97	81.12
6	A	89.32	94.53	93.39	86.27	93.79	96.47
	B	95.36	98.06	98.05	93.41	95.78	96.53
	C	81.96	82.66	82.72	81.55	82.99	83.00
	D	82.01	84.14	84.79	81.55	83.19	83.61
	E	81.30	81.45	82.50	80.73	82.52	82.99

[1] A = Surface Course; B = Leveling Course; C, D, E = Base Course Subdivisions.

[2] Field test date

densification of the paving materials across the width of the pavement. The horizontal curve at this location would encourage random lateral shifting of the vehicle paths and result in a more even distribution of wheel coverages over the pavement surface.

The density data for this location has been plotted in Figure 10. Each of the data points on the graph for the surface and leveling courses is the average of the values obtained at the six test points. The data points for the base course correspond to the average of eighteen values. Each of the layer curves showed a high rate of densification during the first 3 months of pavement life and a relatively slower rate thereafter. This decreasing rate of densification is due to increased internal friction in the layer mixtures resulting from increased confinement.

As could be expected, the surface course exhibited the highest rate of densification due to its initial low density and the greater vertical stresses in the upper pavement layers. It should be noted that the initial average percent density in this layer was only 89.69 percent. If it is assumed that the minimum laboratory density of 94 percent of maximum theoretical density for this mixture was used for compaction control, the field compacted density barely met the specifications. In addition, the initial air void content in this layer of over 10 percent greatly exceeded the desirable range of 2-6 percent air voids. The increase in percent density of this layer over the first year was approximately 5.0 percent density units.

The density of the leveling course was the highest of the respective layers and it increased about 2.6 percent density units during the first year. The hot-mix-sand-asphalt showed an increase of only 1.24 in percent density for the same time period. While the initial density of this

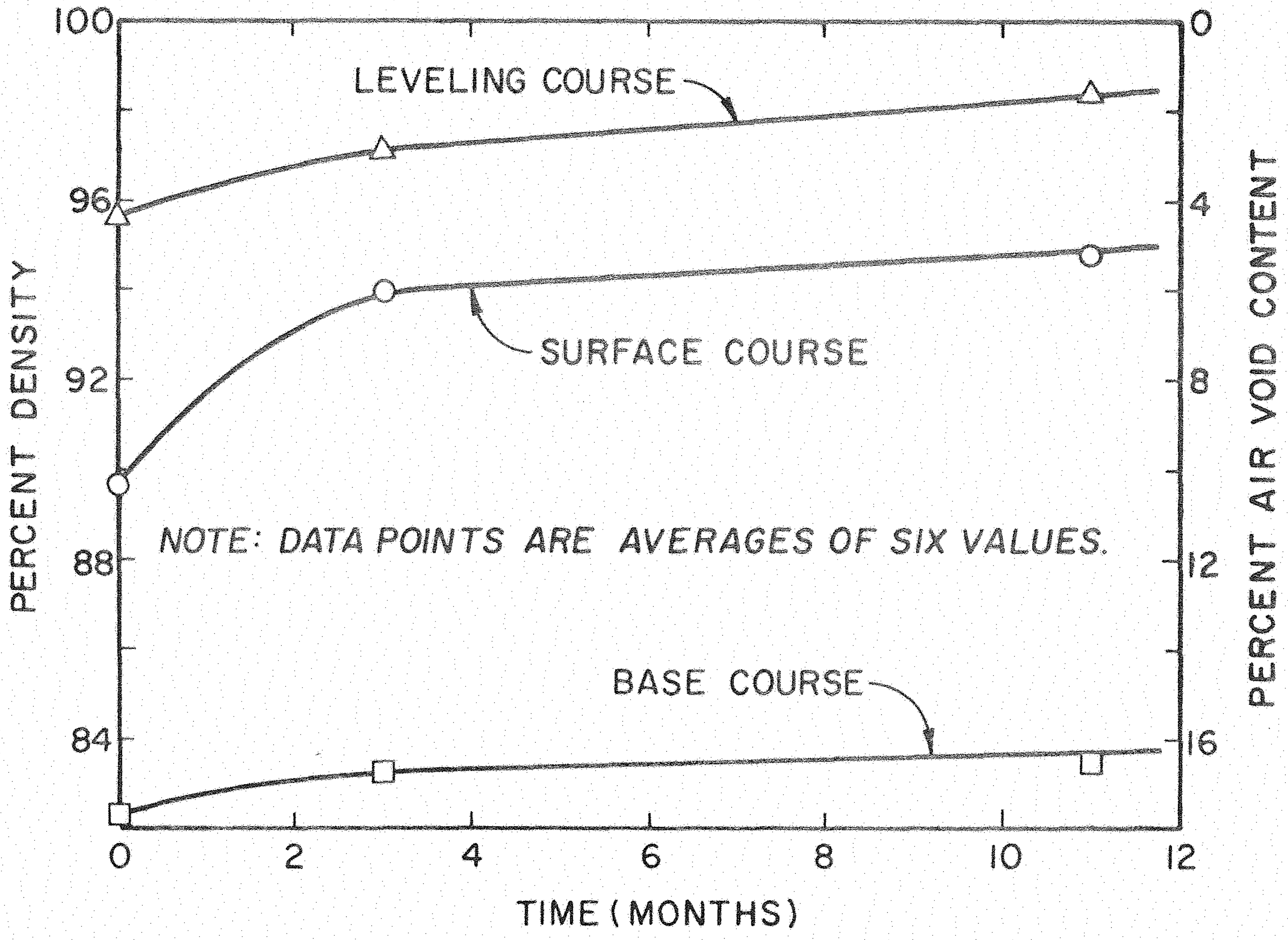

Figure 10. Percent Density Vs. Time, SH 51-2 Test Site

hot-sand mix was low in comparison with those of the upper layers, it was within the specification range for this type of base material.

SH 51-3 Test Site. The AOV also indicated evidence of layer, point, time and "layer-time" differences in the percent density values of the paving materials at this test site. The observed significance level was 0.0001 in the case of the layer and time differences. For the point and "layer-time" differences, the significance levels were 0.0003 and 0.0013, respectively.

Layer differences indicate relatively large differences in the percent density values for the various layers. The time differences indicate increasing density with time. Point differences indicate variability in the density values for the materials at different transverse points across the pavement. However, the AOV did not indicate strong evidence of "point-time" differences so the point differences cannot necessarily be interpreted to mean higher densities of the materials in the wheelpaths relative to those at the mid-wheelpath locations during the first 11 months of the study period. The indicated variability in density was probably due to lack of uniformity in the degree of compaction obtained during construction.

Figure 11 illustrates the densification trends in the respective layers at this site. Here, the initial density of the leveling course was again the greatest (average = 91.78 percent), followed by that of the surface course (average = 88.18 percent) and that of the hot-sand base (average = 79.80 percent). In each case, these values were lower than those determined at the SH 51-2 test site. Each of the layers experienced a high rate of densification during the early stages of traffic exposure and then a subsequent reduction in rate. During the study

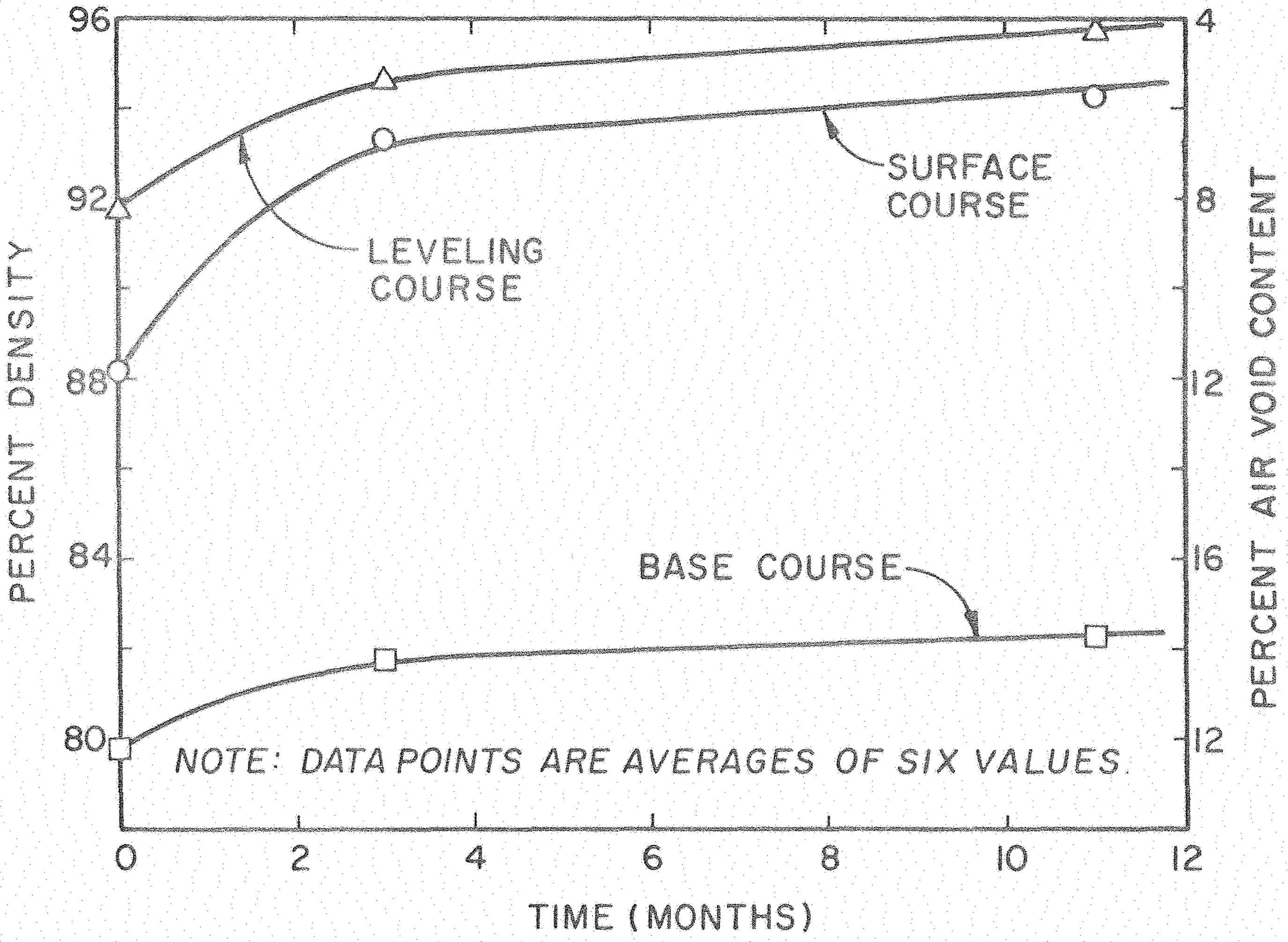

Figure 11. Percent Density Vs. Time, SH 51-3 Test Site

period, the total amount of densification in each of these layers was slightly greater than in the corresponding layers at the SH 51-2 site.

Although the pavement at this site is on the westerly slope of the hill, it lies on a straight tangent section which encourages the concentration of wheel coverages in rather well-defined transverse locations or wheelpaths. Densification was, therefore, expected to be significantly greater at the wheelpath locations, as was found in the study of Interstate highway pavements. The raw data seems to show this tendency, particularly in the east bound lane, but the statistical analysis of the density data revealed no significant differences between the wheelpath and mid-wheelpath points.

Since the total amount of densification was rather small at this early stage in the pavement life, it is possible that relative densification at the wheelpath locations was not large enough to be indicated by the methods employed in collecting and analyzing the density test data. Thermal densification (5) resulting from temperature cycling may also be a factor at this particular location. On the west slope of the hill, the sun's rays would impinge more directly on the pavement surface. At high temperatures the asphalt expands in volume into the void spaces of the mixture and, upon cooling, it contracts pulling the aggregate particles closer together through the action of surface tension. This temperature cycling would be quite uniform at all transverse locations and if the density of the surface materials was initially low the consequent increase in density would be uniform across the pavement.

From the slope of the curves in Figures 10 and 11, it is expected that the densities of the respective material layers will continue to increase during a subsequent two or three year period until a maximum value for the layer is attained. Core samples could be obtained in March of 1976 and 1977 to substantiate this, if a continuation of the study is warranted.

An attempt was made to determine the amount of surface heave (adjacent to the wheelpaths) from lateral creep that may have occurred at these test sites. This couldn't be done due to the fact that the pavement surfaces underwent substantial vertical translation during the study period. Thus, no datum for such measurements could be established even though an initial transverse profile of the surface had been made. An estimate of the contribution of lateral creep or instability of the asphalt-bound materials to rutting was made, however.

The increase in percent density or decrease in percent air void content of a material layer is directly proportional to the percent decrease in thickness of the layer. Based on the 8th month rut depths in Table I and interpolated density values at this point of time from Figures 10 and 11, 45.8 percent of the maximum rut depth at the SH 51-2 site can be attributed to densification. At the SH 51-3 site, densification was responsible for 56.3 percent of the rut depth. Since surface wear was minimal for this period of time, the balance of the rut depth at both sites was considered due to lateral creep or instability in the respective material layers. The direct relationship between the density of a mix and its stability (6) and the low initial densities of the material layers at these sites lends credence to this analysis.

CHAPTER III

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

Based on the test procedures and the pavement sections included in this study, the following conclusions are made:

1. Although highly desirable, a transverse profile of a pavement surface made prior to opening the facility to traffic does not necessarily provide a true datum for the determination of subsequent surface deformations. Subsidence and/or heave in the subgrade and consequent vertical translations of the pavement complicate the measurements and require that surface elevations be established.

2. Elevation points at the shoulders and centerline of the pavement at the two test sites were found to undergo changes in relative elevation with time. At the SH 51-2 site, these changes were all subsidence and amounted to a maximum of 0.6 in. at the north and south shoulder lines. Both heave and subsidence of the surface elevation points were observed at the SH 51-3 site during the first six months of the study. In this case, these elevation changes may have been cyclic in nature and resulted from subgrade moisture variations.

3. At both test sites, ruts developed rapidly during the first four to six months of the study with maximum rut depths occurring approximately nine months after the pavement was opened to traffic. Subsequently, a gradual reduction in rut depth was observed.

4. Relatively low initial percent density values were determined for the respective asphalt bound material layers at each test site. Void contents in these layers were as high as 12 percent in the surface course, 8 percent in the leveling course and 20 percent in the hot-sand base.

5. Post-construction densification was found in each of the asphalt layers at the two test sites. Densification occurred at both the wheelpath and non-wheelpath locations. Generally, the rate of densification was greatest during the first three months of traffic exposure. Increases of up to 10 percent density units in the surface course were observed at wheelpath locations.

6. Based on the maximum rut depth at the end of the 8th month and interpolated average layer densities at this point in time, 45.9 percent and 56.3 percent of the rut depth were attributed to densification of the respective asphalt bound layers at the two respective test sites.

7. Since surface wear was minimal and there were no surface indications of general subgrade failure, the remainder of the measured rut depth at these locations was ascribed to lateral displacement or instability of the respective pavement layers. Evidence of lateral creep at the surface was obscured by the relative elevation changes in the pavement.

8. Some surface wear or attrition in the wheelpaths was indicated by stereophoto analysis but was not considered large enough, at this point in time, to appreciably influence the total rut depth.

Recommendations

It is believed that the results of this study add to and substantiate the findings reported in Interim Report II, "Characteristics of Rutting on

High Quality Bituminous Highway Pavements". Specific recommendations in this previous report concerning field inspection procedures, construction specifications and laboratory mix design methods to minimize the contributions of densification and lateral creep to rutting are applicable here and are not repeated.

REFERENCES

1. Manke, P.G. and S. Oteng-Seifah, "Characteristics of Rutting on High Quality Bituminous Highway Pavements", Interim Report II, Research Project 72-03-3, Oklahoma Department of Highways, December, 1975.

2. American Society for Testing and Materials, 1975 Annual Book of ASTM Standards, Part 15.

3. Barr, A.J., et al, "A User's Guide to the Statistical Analysis System". North Carolina State University Press, August, 1972.

4. Snedecor, W.J., and W.G. Cochran, "Statistical Methods", Iowa State Press, 6th Ed., 1972.

5. Ellis, D.S., et al, "Thermally Induced Densification of Asphalt Concrete", Proceedings, Association of Asphalt Paving Technologists, Vol. 38, February, 1969, p 660.

6. McLeod, N.W., "Discussion on Session VI - Design and Construction Influence on Structural Behavior of Asphalt Pavements", Proceedings International Conference on the Structural Design of Asphalt Pavements, University of Michigan, August 1962, p 767.