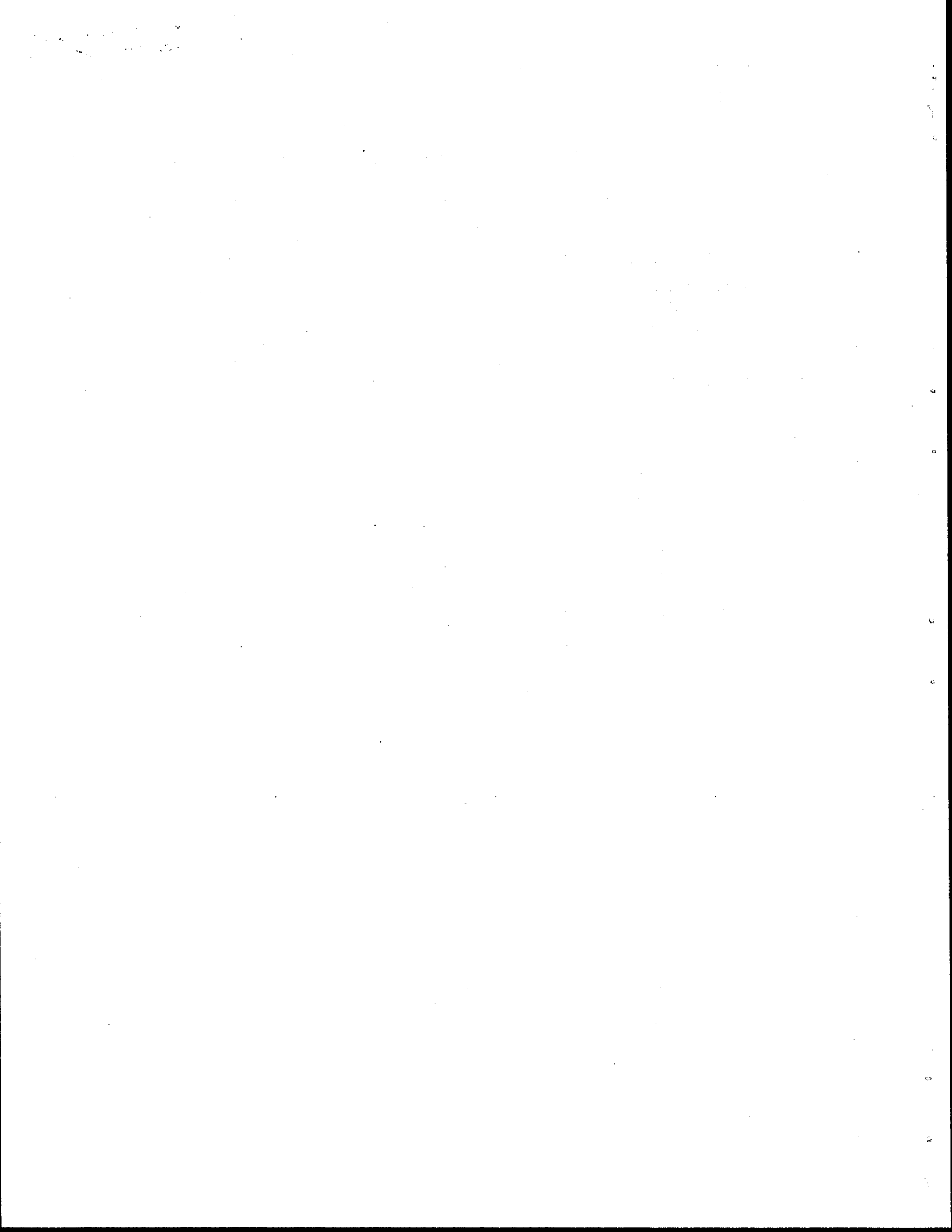


1. Report No. TTI-2-9-72-175-2F	2. Government Accession No.	3. Recipient's Catalog No.	
4. Title and Subtitle LOW TEMPERATURE TRANSVERSE CRACKING OF ASPHALT CONCRETE PAVEMENTS IN CENTRAL AND WEST TEXAS		5. Report Date September, 1976	
7. Author(s) Paul E. Benson		6. Performing Organization Code	
9. Performing Organization Name and Address Texas Transportation Institute Texas A&M University College Station, Texas 77843		8. Performing Organization Report No. Research Report No. 175-2F	
12. Sponsoring Agency Name and Address Texas State Department of Highways and Public Transportation; Transportation Planning Division P. O. Box 5051 Austin, Texas 78763		10. Work Unit No.	
15. Supplementary Notes Research performed in cooperation with DOT, FHWA. Study Title: "Hardening of Asphalt".		11. Contract or Grant No. Study No. 2-9-72-175	
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17. Key Words Asphalt hardening, thermal cracking, asphalt content, compactive effort, special additives.		18. Distribution Statement No Restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22151.	
19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages 62	22. Price



LOW TEMPERATURE TRANSVERSE CRACKING OF ASPHALT  
CONCRETE PAVEMENTS IN CENTRAL AND WEST TEXAS

by

Paul E. Benson

Research Report 175-2F

Research Study No. 2-9-72-175  
Hardening of Asphalt

Sponsored by

State Department of Highways and Public Transportation  
in cooperation with the  
U. S. Department of Transportation  
Federal Highway Administration

September, 1976

TEXAS TRANSPORTATION INSTITUTE  
Texas A&M University  
College Station, Texas

## ABSTRACT

An evaluation of the relation between asphalt hardening, thermal forces and pavement cracking in Texas is made. Thermally induced cracking of hardened pavements is found to be a potential failure mechanism in West and Central Texas. Factors affecting asphalt hardening susceptibility are studied. The form of a generalized model for predicting hardening of asphalt as a function of time is proposed. Recommendations regarding the type of asphalt to be used in West and Central Texas are made.

## PREFACE

An excessive amount of transverse pavement cracking of asphalt concrete highways has been observed in West Texas. Thermally induced cracking of hardened pavements is one of the possible causes of this costly problem. This report studies the potential for thermal cracking of pavements in Texas, and also makes suggestions on how to choose asphalts that will be resistant to age hardening. It constitutes the final report of the study entitled "Hardening of Asphalt." The study was sponsored by the Texas Department of Highways and Public Transportation in cooperation with the Federal Highway Administration.

DISCLAIMER

The contents of this report reflect the views of the author, who is responsible for the opinions, findings and conclusions presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

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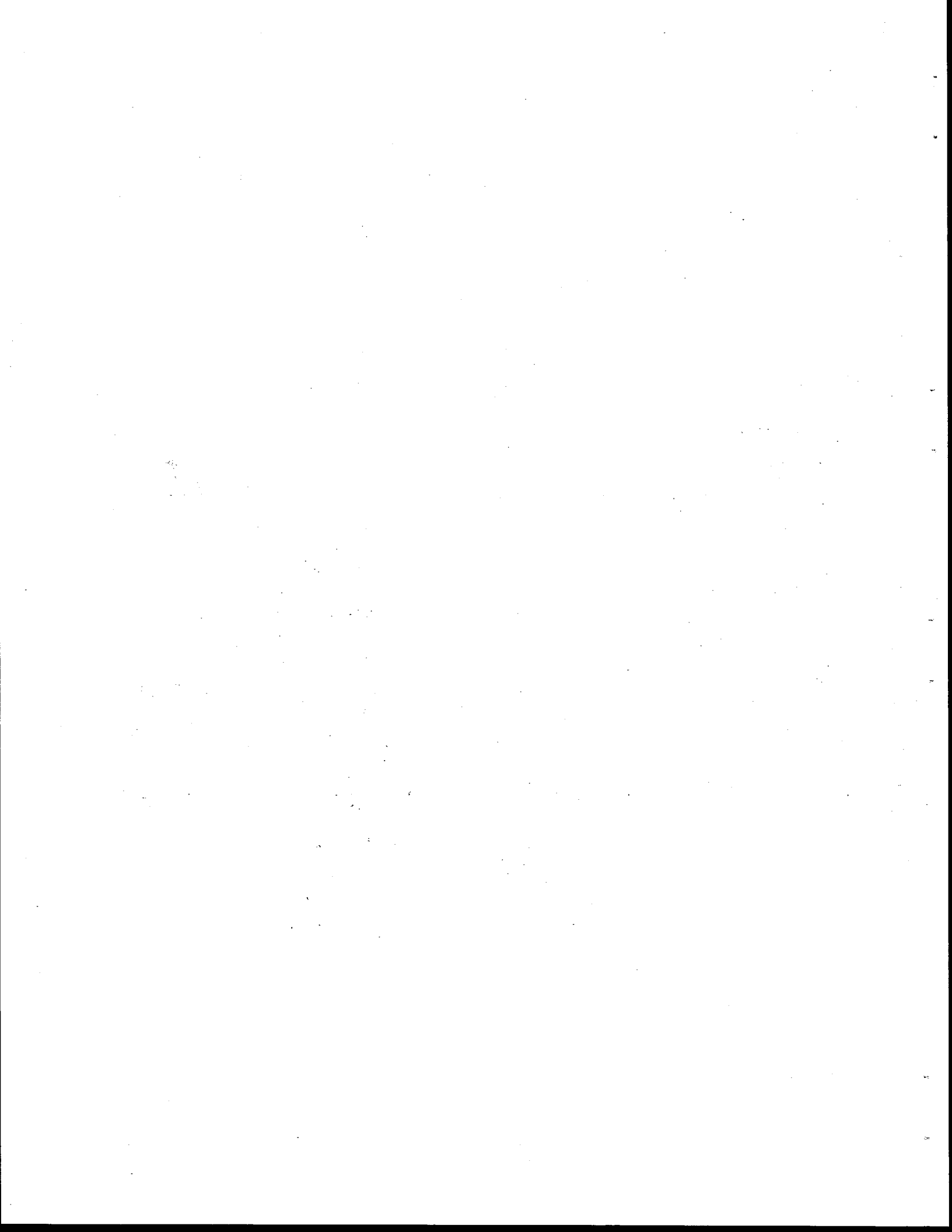
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## INTRODUCTION

The phenomenon of low-temperature transverse cracking of asphalt concrete pavements has received much attention in recent years. Most of the work has been carried out by individuals and agencies in Canada and the northern United States working with data from these areas (1, 5, 11, 13,). Based on the nature of the climate in these areas, it is quite natural that thermal cracking of bituminous pavement would be expected. However, the problem may not be limited to these areas. Excessive transverse cracking has been observed on highways in parts of Texas that are subject to severe weather conditions. These cracks are appearing much earlier and with a greater frequency than cracks caused by conventional load related mechanisms. The rising maintenance costs for these roads have prompted research into the causes and cures of transverse cracking in Texas.

The climate in the western half of Texas is potentially damaging to susceptible asphalt concrete pavements. Incoming storm systems bring dramatic temperature drops to the Panhandle Region while cloudless skies over the far western portions of the state subject pavements to extremely high solar radiation exposures. These areas undergo excessive day to night temperature cycling throughout the year. Such conditions place extremely broad demands on asphalt-aggregate mixtures. Paving mixtures must have sufficient mass viscosity to meet stability requirements, and yet have the capacity to effectively resist age hardening and fatigue failures.

Statewide pavement condition surveys begun in 1973 have confirmed suspicions that premature transverse cracking is a widespread problem in

West Texas. Two mechanisms have been suggested thus far as being the principal causes of the problem. The first deals with swelling clays found in many West Texas subgrades and is discussed in another report (3). This mechanism is manifested by deep cracks that reflect up from the subgrade. However, distress in many roads begins with shallow hairline cracks in the surface course and these cracks progress in a short time producing cracks through the entire bituminous layer. This distress pattern suggests then the possibility that thermal cycling may induce pavement failure. Thus, the second mechanism is one in which transverse cracks are caused by rapid, severe temperature loadings on excessively brittle asphalt concrete pavements. Single cracks soon regress into zones of multiple cracking with a resulting deterioration of ride quality, increased maintenance and overall shortening of pavement life. This type of behavior, called low-temperature cracking, has already been dramatically demonstrated on Canadian test roads (13). Climatic conditions in West Texas show a similar potential for this mechanism. Though minimum temperatures are not as extreme as in Canada, rates of temperature drop are nearly as severe. Solar radiation and thermal cycling are also predominant environmental factors in West Texas, and could be causing extensive age hardening.

This report investigates the significance of low-temperature cracking as it relates to pavements in Texas. The bulk of the data was obtained from cored samples taken periodically from over 50 highway projects throughout the state. From this base a hardening model was developed and pavement performance studied. In addition, controlled laboratory studies of asphalt specimens called "pizzas" were made. These studies were designed to assess the effect of factors such as

asphalt content, compactive effort, solar radiation and special additives on age hardening.

Throughout this report the emphasis is on studying the causes and significance of low-temperature transverse cracking in West Texas.

Some possible cures are reviewed: a) special additives to retard hardening, b) a new screening test for asphalts which simulates hardening by solar radiation, and c) modification of design and construction standards. The problem is a complex one and a costly one in terms of maintenance. In fact, a whole new maintenance strategy may have to be designed to cope with it. The ideal situation, however, is to design and construct new highway facilities so that transverse cracking will be minimized. It is hoped that the findings of this report will help promote design and construction standards that will make this a reality.

### FIELD PERFORMANCE ANALYSIS

Of the more than 50 test sections chosen for this project over half were established during two earlier projects. This permitted evaluation of six- and nine-year old pavements concurrently with this project's one and a half-year old pavements. One disadvantage to this scheme was the inability to obtain uniform data on original asphalt and mix properties for all three projects. Also, some of the older sections had received seal coats or overlays since their construction thus clouding the data. This made an evaluation of field performance questionable for some of the sections.

Visual ratings of the test sections were made by several independent observers during 1973 and 1974. These data, which included all types of pavement distress, were reported on standard rating forms used by the Texas Department of Highways and Public Transportation. Specific data on the amount of transverse, longitudinal and alligator cracking were extracted from these forms. Four degrees of cracking frequency were reported for each type of distress and these are delineated as follows:

<u>Code</u>	Transverse (No./Sta.)	Longitudinal (Lin.Ft. Sta.)	Alligator (% Area)
0	0	0	0
1	1 to 4	1 to 99	1 to 5
2	5 to 9	100 to 199	6 to 25
3	<u>&gt;</u> 10	<u>&gt;</u> 200	>25

Mays Meter readings were also taken at the test sites in 1974. From these readings serviceability indices were computed for the sections. The serviceability index (SI) is a number ranging from 0 to 5 that is a

measure of road smoothness or riding quality. From the data presented here and from past observations a strong correlation between cracking and smoothness is not evident; however, riding quality rates highly with the driving public and is therefore included. Riding quality is broken down as follows:

<u>SI</u>	<u>RIDING QUALITY</u>
4-5	Very Good
3-4	Good
2-3	Fair
1-2	Poor
0-1	Very Poor

Tables 1 thru 3 summarize the test section locations, construction type, crack frequency and S.I. Test sections that received subsequent treatment prior to visual surveys have the dates and nature of that treatment indicated. There are four types of construction noted: new construction, stage construction (resurfacing of newly built flexible pavements), resurfacing of older flexible pavements and resurfacing of concrete pavements.

Unfortunately, few of the test sections were new or stage construction. This made evaluation of field performance more difficult. For instance, test sections 2, 10, 17, and 20 in Table 1 show signs of transverse cracking and as may be noted these are overlays on concrete pavements. The cracks naturally occur at the expansion joints of the underlying slabs. Hence, pavements with rigid bases were excluded from the field performance analysis. Similarly, resurfaced flexible pavements were eliminated because of the possibility of underlying cracks influencing the development of surface cracking patterns. Therefore, only sections

TABLE 1. History and Performance of 1 1/2-Year Old Test Sections

Site No.	District	Highway	Construction Type	Subsequent Treatment	Crack Frequency			S. I.
					Transverse	Longitudinal	Alligator	
1	2	FM 730	Resurf. (Flex.)	None	0	0	0	3.2
2	3	US 82	Resurf. (Conc.)	None	1	1	0	4.1
3	4	US 60	Resurf. (Flex.)	None	0	0	0	4.5
4	4	SH 15	Resurf. (Flex.)	None	0	0	0	4.8
5	5	US 87	New	None	0	0	0	3.9
6	6	IH 10	New	None	0	0	0	3.8
7	8	US 87	Resurf. (Flex.)	None	-	-	-	-
8	8	US 83	Resurf. (Flex.)	None	0	0	0	4.1
9	11	FM 58	New	None	0	0	0	3.6
10	13	SH 60	Resurf. (Conc.)	None	2	1	0	4.4
11	15	US 281	Resurf. (Flex.)	None	0	1	0	4.5
12	16	IH 37	Resurf. (Flex.)	None	0	0	0	3.2
13	16	SH 358	New	None	0	0	0	3.4
14	17	SH 6	New	None	0	0	0	4.5
15	17	US 79	New	None	0	1	0	4.3
16	18	SH 205	Resurf. (Flex.)	None	0	0	0	3.7
17	19	US 79	Resurf. (Conc.)	None	3	2	0	3.9
18	20	US 96	Resurf. (Flex.)	None	0	0	0	4.0
19	20	SH 87	Resurf. (Flex.)	None	0	0	0	3.3
20	20	SH 146	Resurf. (Conc.)	None	1	1	0	3.9
21	21	FM 493	Resurf. (Flex.)	None	-	-	-	-
22	24	US 54	Resurf. (Flex.)	None	0	0	0	-



TABLE 2. History and Performance of 6- to 7-Year Old Test Sections

Site No.	District	Highway	Construction Type	Subsequent Treatment	Crack Frequency			S. I.
					Transverse	Longitudinal	Alligator	
23	1	SH 24	Resurf. (Conc.)	None	1	1	0	4.2
24	1	IH 30	Resurf. (Conc.)	None	2	0	0	3.8
25	1	SH 5	Resurf. (Conc.)	None	2	1	3	3.6
26	9	SH 6	New	None	3	3	0	3.7
27	9	US 77	Resurf. (Flex.)	None	-	-	-	4.1
28	9	US 84	New	None	0	0	1	3.9
29	12	SP 330	Resurf. (Conc.)	Resurf. 1971	1	1	0	4.3
30	12	IH 45	Resurf. (Conc.)	Resurf. 1972	1	1	0	4.2
31	12	FM 1485	Resurf. (Flex.)	None	3	2	0	4.2
32	17	SH 308	Resurf. (Conc.)	None	2	2	0	2.5
33	17	SH 36	Resurf. (Flex.)	Seal Coat 1969	2	3	0	3.6
34	20	SH 87	Resurf. (Flex.)	None	0	0	0	3.3
35	20	SH 12	Resurf. (Conc.)	Resurf. 1971	2	2	0	3.4
36	25	US 287	New	None	1	2	0	4.5
37	25	US 70	Stage	None	3	2	0	3.5

TABLE 3. History and Performance of 9- to 10-Year Old Test Sections

Site No.	District	Highway	Construction Type	Subsequent Treatment	Crack Frequency			S. I.
					Transverse	Longitudinal	Alligator	
38	4	US 54	New	None	0	0	0	4.7
39	5	US 87	New	Seal Coat 1969	3	2	0	3.9
40	8	US 83	New	None	2	3	1	2.4
41	9	US 84	New	None	0	0	0	3.9
42	9	SH 31	Resurf. (Conc.)	None	2	1	0	3.7
43	12	FM 1314	Resurf. (Flex.)	None	1	1	0	1.9
44	13	US 59	Resurf. (Conc.)	Resurf. 1973	1	1	0	4.4
45	16	IH 37	Resurf. (Flex.)	None	1	0	0	3.2
46	18	SH 34	Resurf. (Flex.)	None	1	0	0	3.8
47	19	US 259	Stage	None	1	1	0	4.4
48	20	SH 105	Resurf. (Conc.)	None	1	1	0	-
49	21	US 281	Resurf. (Flex.)	Seal Coat 1973	-	-	-	3.8
50	21	US 83	New	Reconstruct. 1968	-	-	-	3.8
51	24	US 62	Resurf. (Flex.)	None	3	2	0	3.7

that were new or stage construction, and subsequently received no major treatment, could be used for the field performance analysis. This limited the number of test sections available for the analysis.

It is apparent from Table 1 that newly constructed asphalt concrete pavements suffer no transverse cracking problems under Texas environmental conditions in their first year and a half of service. In addition, flexible pavement overlays show no signs of transverse cracking whether reflective or otherwise in the same time period. McLeod (13) has stated that four to seven years are needed before an 85-100 asphalt hardens to the point where low temperature cracking begins to occur. This is based on environmental conditions in Ontario, Canada, where much lower pavement temperatures are reached. Thus, while the 1½-year test sites were characteristic of short term asphalt hardening behavior, the six- and nine-year sites gave the best opportunity for studying low temperature cracking of hardened asphalts.

Nine test sections were therefore selected for the field performance analysis after elimination of resurfaced existing pavements, subsequently treated pavements, and 1½-year old pavements. The results, however, are convincingly consistent with the hypothesis that transverse cracking and asphalt hardening are related.

Table 4 summarizes the pertinent material properties for the nine selected sections. The entries are ranked in order of increasing transverse crack frequency. There are obviously many important forces interacting to determine the crack frequency for a given pavement among which are: age, environment, traffic volume and asphalt quality. However, all of these forces, except traffic and the thermal loading potentials of the environment, should be reflected in a measurement of the asphalt hardness.

TABLE 4. Spearman's Rank Correlation Coefficient Between Transverse Crack Frequency and Various Physical Parameters.

Site No.	Age (Years)	Trans. Crack Freq.	Reological Properties					Strength Properties @ 32°				% Air Voids
			Visc. @ 77° (MP)	Hard. Index @ 77°	Visc. @ 275° (Stokes)	Pen. @ 32° (MM)	Pen. @ 77° (MM)	Asphalt Stiffness (KG/CM <sup>2</sup> )	Schmidt M <sub>R</sub> (PSI x 10 <sup>6</sup> )	Direct Tension (PSI x 10 <sup>6</sup> )	Indirect Tension (PSI x 10 <sup>6</sup> )	
28	7	0	5.9	6.8	7.7	22	45	5	4.3		0.46	1.8
38	10	0	12.4	12.7	14.4	13	33	10	2.0		0.45	3.0
41	9	0	18.0	15.7	10.7	14	26	25	2.9		0.41	1.5
47	9	1	16.4	13.9	16.5	15	25	25	2.8		0.42	2.3
36	6 1/2	1	35.6	16.2	14.0	10	12	120	3.2		0.40	3.4
40	9	2	54.0	62.8	8.3	7	13	100	3.4		0.43	5.4
37	6	3	54.0	20.0	22.5	9	15	80	2.5		0.63	5.1
26	6 1/2	3	70.0	26.9	18.5	7	16	70	—†	—†	—†	6.4
39	10	3	76.0	79.2	14.1	5	10	230	4.1		0.39	5.8
		r	0.91*	0.85*	0.58	-0.75*	-0.68	0.74*	0.12		-0.10	0.89*

\* Significant at the 95% Confidence Level.

† No cores available.

Temperatures are given in °F.

Therefore, if one accepts the hypothesis that thermal cracking of pavement occurs in parts of Texas, one can reasonably expect a strong individual correlation between crack frequency and asphalt hardness.

The Spearman's Rank Correlation Coefficient ( $\rho$ ) is used in Table 4 to test for the significance of this correlation. This is a non-parametric statistical technique that does not require normally distributed data, and is therefore convenient to use with typically non-linear measurements such as viscosity and stiffness.

Viscosity at 77<sup>o</sup>, Hardening Index at 77<sup>o</sup> and Penetration at 77<sup>o</sup> correlate significantly at the 95% level with transverse crack frequency. For a given type of test (i.e. viscosity or penetration) the values at a lower test temperature give better correlation. This suggests that low temperature asphalt behavior is more important in determining transverse cracking susceptibility than high temperature behavior. Numerous other studies attest to this observation (2) (4) (13). This is exactly the type of relationship that could be expected from thermally induced transverse cracking failures.

The hardening index (H.I.) @ 77<sup>o</sup>F is also included in Table 4. This index, which has been used in the literature for some years, is simply the recovered viscosity divided by the original viscosity. Its correlation coefficient, 0.85, is lower, though not significantly so, than that of the actual recovered viscosity. This is probably due to the fact that this index is sensitive to both the measurement of the original viscosity and the final viscosity. The final viscosity is, of course, more directly related to the actual cracking potential. The intent of the index has been to classify asphalts by characteristic hardening susceptibilities, thus enabling the consumer to predict age hardening with

original asphalt properties. This is a valid goal, but requires a more complete model that will give weight to the numerous factors governing asphalt hardening. Such a model is discussed in a later section of this report.

Estimated stiffness at 32<sup>o</sup>F of the recovered asphalts [from Heudelom and Klomp's nomograph (10)] and strength tests at the same temperature on pavement cores are also shown in Table 4. It is interesting to note that the stiffness of the binder is well correlated to transverse cracking while the resilient modulus of the binder-aggregate mixture is not. This would indicate that attempts to control thermal cracking should be focused on the properties of the binder, not the mixture. However, mixture design and asphalt properties are too strongly interrelated to the mixture stability and asphalt hardening susceptibility of the final product to consider separately. Control of thermal cracking and stability problems will only come through the coordinated control of the mixture and its constituents.

Included at the extreme right of Table 4 are the "final" percent air voids for each of the test sections. These values correlate significantly with the transverse crack frequency. They also correlate nearly as well ( $\rho = 0.86$ ) with viscosity at 77<sup>o</sup>. It has been stated that less dense mixtures are prone to excessive oxidation hardening in early pavement life, from construction to sealing of the surface by traffic (7). This would lead to excessive asphalt hardening early in pavement life, and hence greater potential for transverse cracking. These field results indicate that air voids do play an important role in determining susceptibility of pavements to thermally induced transverse cracking.

TABLE 5. Pertinent Environmental Data from Test Sections Chosen for Field Performance Analysis.

Site No.	Transverse Crack Frequency	ADT/Lane	ENVIRONMENTAL FACTORS (1966 to 1973)						1966-1973 Minimum Temp. (°F)	Estimated Crack Temp. (°F)
			AVERAGE ANNUAL							
			Solar Radiation (Langleys/ Day)	Precip. (in.)	Freeze- Thaw Cycles	Maximum Monthly Degrees- Days	Minimum Temp. (°F)	1966-1973 Minimum Temp. (°F)		
28	0	1650	430	32	27	586	17	4	-23	
38	0	950	485	18	128	985	-1	-8	-33	
41	0	1720	430	32	27	586	17	4	-12	
47	1	1510	420	48	58	686	13	9	-22	
36	1	1000	470	19	75	838	5	-3	5	
40	2	870	460	25	52	687	9	-1	-3	
37	3	410	475	21	75	794	6	0	-1	
26	3	1330	425	34	44	642	14	9	*	
39	3	1450	485	18	92	879	5	-2	12	

\* Insufficient Data

Table 5 gives a summary of environmental factors for each of the test sections used in the cracking study. None of these factors correlate directly with transverse crack frequency. This is not surprising since different materials are used at each of the test sites and have varying tolerances to environmental conditions. It does indicate, however, that within the areas studied (primarily Northwest and Central Texas) materials properties have a far more significant effect on cracking susceptibility than environmental factors. If this were not so, the correlations in Table 4 would be confounded by different site conditions, and would therefore not have proven to be significant.

Some insight into the influence of environmental factors was sought by studying pairs of test sites with similar binder properties. By using the viscosity results at 77°F shown in Table 4, the following pairs (by site number) can be arranged: (41, 47), (40, 37), (39, 26). The second site listed in each pair except the last had a greater transverse crack frequency than the first. For the last pair Site 39 had an eight-year rating of two, so that Site 26 actually exhibited earlier distress. By isolating environmental factors for these pairs it was thought that a pattern might emerge. The only pattern that did emerge involved traffic volume. Though it could hardly be called significant, it indicated that the less heavily travelled sections incurred more transverse cracking. If anything, this suggests that transverse cracking in West Texas is not a load-related phenomenon.

Another way to evaluate at least one of the environmental factors was to estimate the crack formation temperature of each of the pavements from laboratory data, and then compare this to the minimum recorded temperature at the test site. Crack formation temperatures were estimated



by using all incremental computation method which sums thermal stress build-up and compares it to mixture tensile strength at each incremented temperature (11). A cooling rate of 9°/hr. (a reasonable maximum for Central and Northwest Texas) was assumed along with a coefficient of expansion of  $1.5 \times 10^{-5}/F^{\circ}$  (12) and an initial pavement temperature of 50°F. Bitumen stiffnesses were estimated using Huekolem and Klomp's nomograph (10), and mixture stiffnesses were then computed by techniques developed in their reference. Huekolem's work (9) was also used to calculate tensile strength at each incremented temperature. Mixture factors for this calculation were determined at 32°F and 0.1 second loading rate. For a set of results from typical calculations, see Table 6.

At those sites where no transverse cracking was observed, minimum temperatures never approached the estimated crack formation temperature of the pavement (see Table 5). However, this was also true of Site 47 at which minor cracking occurred. The anomaly could be due to the fact that Site 47 was the only site located in East Texas, and as such far outstripped the others in average annual precipitation, suggesting a possible moisture related mode of failure. The remaining four test sections (Sites nos. 36, 37, 39, and 70) where transverse cracking occurred and sufficient data were available for computations, show minimum temperatures approaching or surpassing estimated crack formation temperatures. In all likelihood then, these sections experienced thermally induced stresses which exceeded their tensile strengths, thereby resulting in the observed transverse cracks.

An additional reason for alarm is the frequent association between transverse and longitudinal cracks. Longitudinal cracking is commonly

TABLE 6. Incremental Computation of Thermally Induced Stresses.

Site No. 37 Cooling Rate 9°F/Hr.  
 $\alpha$   $1.5 \times 10^{-5}/^{\circ}\text{F}$   $C_v$  0.87 % Voids 3.4 Mix Factor 0.59

T (°F)	$\Delta T$	$S_{BIT_2}$ (KG/CM <sup>2</sup> )	$S_{MIX_2}$ (KG/CM <sup>2</sup> )	Thermal Strain $\epsilon_{TH}$	Thermal Stress $\sigma_{TH}$ (KG/CM <sup>2</sup> )	$\Sigma \sigma_{TH}$ (KG/CM <sup>2</sup> )	Tensile Strength <sub>2</sub> $\sigma_{BR}$ (KG/CM <sup>2</sup> )
50							
	9	20	$8.0 \times 10^3$	$1.4 \times 10^{-4}$	1.1		
41	"	50	$1.5 \times 10^4$	"	2.1	1.1	11.8
32	"	100	$2.3 \times 10^4$	"	3.2	3.2	18.6
23	"	220	$3.8 \times 10^4$	"	5.3	6.4	29.0
14	"	450	$5.9 \times 10^4$	"	8.3	11.7	29.8
5	"	900	$8.9 \times 10^4$	"	12.5	20.0	29.1
-4						32.5	27.7

Failure Temperature by Linear  
Interpolation = -1°F

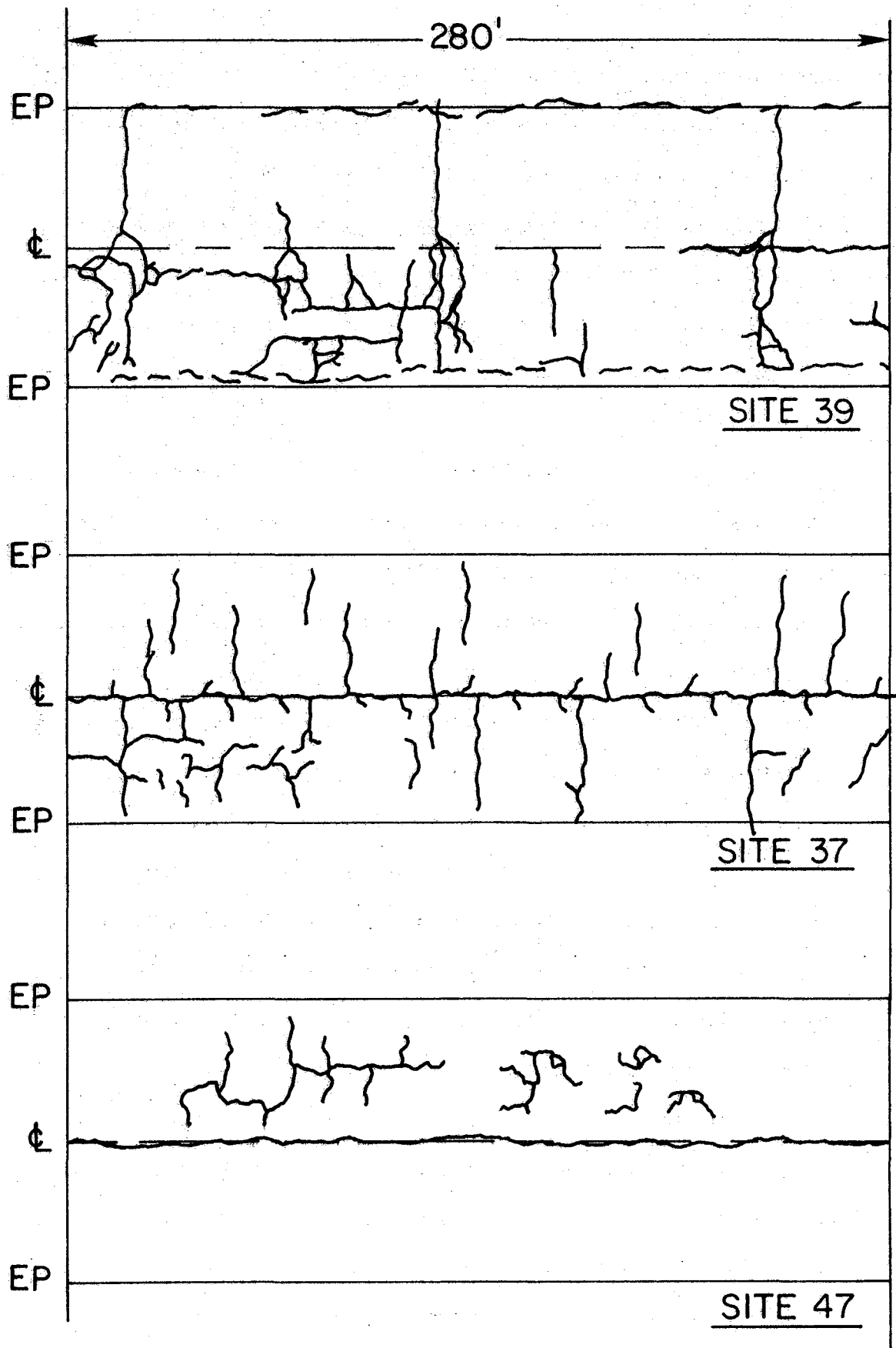


Figure I. Crack Diagrams

associated with load related fatigue failures, not thermal stresses. It is apparent from field observations, however, that the two types of cracks interact. Sketches of representative cracking patterns of three of the nine test sections in the field performance analysis are shown in Figure 1. Except for shoulder and centerline cracking, all longitudinal cracks tie into transverse cracks. In fact, longitudinal cracking correlates well with transverse cracking for all nine test sections (Spearman's  $\rho = 0.86$ ).

There is no clear cut picture, however, of which type of crack is occurring first. The sketches in Figure 1 show transverse cracks occurring alone in some places while longitudinal cracks are always accompanied by transverse cracks (except for shoulder and centerline cracking). This suggests that the transverse cracks are occurring first and then propagating into longitudinal cracks by providing zones of weakness and water access to foundation materials. If this mechanism is, in fact, occurring then transverse cracking followed by excessive longitudinal cracking may reduce the Serviceability Index to a very low value in a relatively short period of time. Resurfacing and maintenance procedures can then only delay the inevitable roadway disintegration. The problem may be minimized by selecting the correct materials and by effecting good construction techniques.

It appears from the following analysis that thermally induced transverse cracking is a problem to be dealt with, at least in Central and Northwest Texas. It is also apparent that the crucial factor in controlling this problem is the susceptibility to age hardening of the asphaltic binder. Specifications are needed that will clearly spell out the lowest acceptable unit for this property and promote construction methods that will deter age hardening.

## LABORATORY SPECIMENS

Complete evaluation of the factors governing asphalt hardening would be impractical from field data alone. Field conditions do not normally permit precise control of all influencing factors for a discrete analysis. A more acceptable degree of control is achieved by studying specimens fabricated under laboratory conditions. In this manner, several important factors can be held constant while one factor is varied. The degree to which that factor affects asphalt hardening can then be assessed without interference. Such information can be extremely helpful when trying to develop an overall model for asphalt hardening from limited amounts of field data.

The laboratory specimens used for this study were 17 inches in diameter and 2 inches thick, and were compacted by a large gyratory compactor in two lifts. These specimens, called pizzas, were fabricated in three different ways. Most were mixed and compacted in the laboratory from different asphalts and to different void and asphalt contents. These were called Group A pizzas. Group B pizzas were composed of a job site asphalt-aggregate mixture from Site 14 which was compacted in the laboratory. The only variable for these pizzas was the degree of compaction. Group C pizzas were the counterparts of several Group A pizzas, but with a hardening inhibitor added. All the pizzas were made using the same aggregate and, except for some different asphalt contents, the same mixture design.

Pizzas were placed in specially prepared outdoor sandbeds in El Paso and in Bryan, Texas. The prime climatic differences between these two locations are solar radiation temperature and precipitation. A control set of pizzas was kept in darkness and at room temperature in a basement

on the Texas A&M University Campus.

The pizzas were cored after three years, and a battery of tests were run on the cores. Of primary interest to this study were the properties of the recovered asphalts. Several analyses were performed with the following objectives:

1. To measure the importance of compactive effort as a means to deter asphalt hardening,
2. To assess the effectiveness of an anti-hardening additive for asphalts,
3. To evaluate the importance with regard to hardening susceptibility of maintaining optimum asphalt content,
4. To relate actinic light tests to asphalt hardening susceptibility,
5. To assess the rate of asphalt hardening as a function of exposure to solar radiation.

Four indicators were studied in each of the analyses. They were: penetration at 77°F, viscosity at 77°F, bitumen stiffness at 32°F as predicted by Heukelom and Klomp's nomograph and resilient modulus at 32°F as measured by the Schmidt Testing Apparatus. Non-parametric statistical techniques or transformations were used when treating the first three indicators because of their non-linear natures.

Table 7 summarizes results for the Group B pizzas used in studying the effects of compaction on asphalt hardening. All these pizzas had nominal 5% asphalt contents. Spearman's rank correlation coefficients were computed for percent voids versus each of the four indicators. Naturally, separate correlations were run for each of the three storage locations. No clear pattern emerged from the results. Three of the twelve coefficients were significant at the 95% confidence level, but

TABLE 7. Spearman's Rank Correlation Coefficient Between Percent Voids and Various Parameters for Group B Pizzas.

Location	Pizza No.	Percent Voids	Penetration @ 77°F (mm)	Viscosity @ 77°F (MP)	S <sub>BIT</sub> @ 32°F 1 Hr. Loading (KG/CM <sup>2</sup> )	M <sub>R</sub> @ 32°F (0.1 Sec. Loading) (PSI x 10 <sup>6</sup> )
El Paso	17B	6.2	21	30	40	3.0
	26B	7.0	24	12	30	4.2
	29B	8.1	31	18	17	3.0
	11B	11.5	68	3.8	2	3.3
	9B	14.4	123	0.5	0.5	2.8
	10B	15.7	47	3.8	4	2.5
	ρ		0.83*	-0.81	-0.83*	-0.64
Bryan	5B	4.9	28	18	20	5.6
	6B	5.8	12	58	130	5.5
	4B	7.2	28	28	20	4.3
	12B	11.2	43	8.4	6	2.9
	2B	11.9	24	19	25	2.9
	14B	12.4	62	1.6	2	2.8
	ρ		0.50	-0.54	-0.47	-0.96*
Control	8B	5.9	14	28	100	4.1
	3B	6.4	33	11	11	3.4
	20B	6.5	22	40	40	2.1
	19B	9.2	32	14	12	3.9
	15B	15.3	110	6.2	0.5	1.5
	16B	16.4	36	11	8	1.5
	ρ		0.77	-0.53	-0.77	-0.79

\* Significant at 95% Confidence Level.

each for a different indicator. Except for resilient modulus, the best correlations were obtained for the El Paso pizzas. This could be interpreted as meaning that compactive effort is critical to hardening susceptibility of pavements in hot, dry environments. Corollary to this would be the idea that hardened, water soluble asphaltic oxidants would be washed away in the moist Bryan environment, and would never form under the stable control conditions. However, extraction results showed that Bryan pizzas did not lose significantly more asphalt than El Paso pizzas during their three years of exposure. Thus, the higher correlations for El Paso pizzas must be attributed to random fluctuations in the results.

A set of fifteen pairs of replicate pizzas were fabricated to evaluate the effect of an anti-hardening additive made up of a combination of paraphenylene diamine antiozonates and ultra-violet light inhibitors. One percent by weight of the additive was added to one pizza of each pair. The asphalts to receive the additive were first heated to 200°F, then poured into a mixing bowl where the additive was introduced. They were mixed for approximately five minutes. Some of the pairs were kept at the Bryan location and some at the control location. In this way, a paired statistical analysis would point out the effectiveness of the additive.

The differences (d) for the four indicators studied between the additive and non-additive pizzas are shown in Table 8. A rank order analysis called the Wilcoxon Matched Pairs Test was performed on the first three indicators. A T-Paired Test was used on the resilient modulus differences. Both these tests can show whether the average of a series of differences varies significantly from zero. If the additive



TABLE 8. Paired Analyses (Wilcoxon and T-Paired) for Pizzas Used in Additive Evaluation.

Pizza Nos.	DIFFERENCES d = Additive - (Non-Additive) Samples			
	Penetration @ 77°F	Viscosity @ 77°F	S <sub>BIT</sub> @ 32°F	M <sub>R</sub> @ 32°F
21C-237	-2	-2.4	1	-0.7
17C-170	-2	-2.4	2	-0.2
7C-121	9	-1.4	-23	0
11C-132	1	15	-15	-0.9
12C-9	8	0	-90	-1.2
8C-136	-33	19.8	56	-0.1
23C-74	7	-3.4	-17	-1.9
2C-84	-16	5.2	3	0.1
4C-104	6	4.0	-160	0.2
10C-160	-23	20.4	90	-0.7
1C-115	-70	24.4	18.5	1.4
3C-98	-13	23.2	30	0
9C-184	-8	8.2	18	0.1
14C-231	12	-4.6	-11	0.2
16C-230	28	-3.6	-7.5	-1.0
	W+ = 46.5  W-  = 73.5	W+ = 83*  W-  = 22	W+ = 59.5  W-  = 60.5	$\bar{d}$ = -0.31 t = -1.56

\* Significant at 94.2% Confidence Level

is effective the differences should average out to indicate softer asphalt in the additive pizza of each pair. However, none of the test statistics showed any significant difference from zero except the Wilcoxon W for viscosity at 77°F. This is significant at the 94.2% level of confidence, but indicates that the additive pizzas have a higher viscosity than the non-additive pizzas. This means that the effectiveness of the particular additive used was not evident. An alternative explanation would be that exposure time was not sufficient to develop a difference between additive and non-additive samples. Since hardening rates follow fairly uniform patterns, however, this seems unlikely.

The same sort of analytical approach was used to evaluate the importance of maintaining optimum asphalt content. In this case nine pairs of pizzas were available for analysis. One member of each pair was  $\frac{1}{2}\%$  below design asphalt content, and one  $\frac{1}{2}\%$  above. All other factors were held constant for each pair (i.e. aggregate, asphalt type, storage location and % voids). It was felt that the thinner film thicknesses of the low asphalt content material would permit a greater proportion of asphalt to be hardened.

However, results showed no significant difference between the low and high asphalt content pizzas. Table 9 summarizes differences for the four indicators and the resulting statistics. Apparently, variations in asphalt content of  $\pm \frac{1}{2}\%$  are not large enough to measurably affect the hardening susceptibility of the mix.

While asphalt content within the  $\pm \frac{1}{2}\%$  limits may not have proved to be important in determining the hardening susceptibility of a mix, asphalt type did. Five different asphalts were chosen to make up a

TABLE 9. Paired Analyses (Wilcoxon and T-Paired) for Pizzas Used in Asphalt Content Evaluation.

Pizza Nos.	DIFFERENCES d = (-1/2%) - (1/2%) Samples			
	Penetration @ 77°F	Viscosity @ 77°F	S <sub>BIT</sub> @ 32°F	M <sub>R</sub> @32°F
134-125	-35	112	66	2.1
137-157	2	-7	-10	1.0
120-243	-4	11	40	1.2
187-164	20	-160	-206	0.1
119-159	-31	10	95	1.6
136-160	13	-3	-6	-0.1
144-29	-23	18	293	0.6
20-25	11	-3	-15	-0.8
21-30	1	5	-4	-0.5
	W+ = 18  W-  = 27	W+ = 29  W-  = 16	W+ = 27  W-  = 18	$\bar{d}$ = 0.58 t = 1.77

special set of fifteen pizzas (three replicates for each asphalt). The first objective was to see whether the different asphalts hardened at significantly different rates. If this were true, then various test results presumably indicating hardening susceptibility would be correlated with actual hardening. The test method that had the best significant correlation with the actual hardening could possibly be used as a specification for governing asphalt hardening susceptibility.

Each of the fifteen pizzas used in this study had approximately 11.5% voids and 5% asphalt content. They were all stored at the El Paso location. Table 10 shows a summary of results for the four indicators studied. A one-way analysis of variance was run on the resilient modulus data and its non-parametric equivalent, the Kruskal-Wallis Statistic (H), was run on the other three parameters. Except for viscosity results, which had some rather large within group differences, the analyses showed significant differences between at least one asphalt and the others. The H statistics and F-Ratio are shown at the bottom of Table 10.

In Table 11 the asphalts are listed in order of descending average recovered penetration. Accompanying these are the corresponding original penetrations and results from several potential indicators of hardening susceptibility. The first of these is the Actinic Light Test. In this test 10-micron film samples of the original asphalt are subjected in air to 1,000 microwatts/ $\text{CM}^2$  of 3,660 Angstrom wave length radiation for 18 hours at  $95 \pm 2^\circ\text{F}$  (14). Before and after viscosity at  $77^\circ\text{F}$  are obtained. The hardening index (H.I.) is defined as the ratio of the before to after viscosities. Results from the Thin Film Oven Test and Relative Viscosity or Aging Index Tests were also examined. Finally, vanadium content, seen

TABLE 10. Summary of Results for Pizzas Used in Producer Analysis.

Producer No.	Pizza No.	Penetration @ 77°F (MM)	Viscosity @ 77°F (MP)	S <sub>BIT</sub> @ 32°F 1 Hr. Loading (KG/CM <sup>2</sup> )	M <sub>R</sub> @ 32°F 0.1 Sec. Loading (PSI x 10 <sup>6</sup> )
1	45	27	17.2	20	3.5
	46	38	18.0	7	4.4
	60	33	28.0	11	4.3
3	48	16	19.2	70	3.1
	49	12	55.0	120	2.5
	50	17	26.0	60	3.5
6	8	29	13.2	18	3.2
	10	23	29.0	40	2.3
	18	45	4.8	5	2.6
8	35	77	2.8	1.5	1.5
	36	75	3.2	1.5	1.7
	37	119	2.1	0.4	1.6
11	39	37	10.4	7.0	2.6
	40	84	8.4	1.0	2.7
	41	27	34.0	20	2.7
	H	10.1*	8.2	10.2*	---
	F	----	---	----	16.3*

\* Significant at 95% Confidence Level.

TABLE 11. Spearman's Rank Correlation Between Average Recovered Penetration and Various Hardening Susceptibility Indicators.

Producer	Average Penetration	Original Penetration	After Actinic Light		After TFOT		Relative Viscosity	Vanadium Content (PPM)
			Viscosity @ 77°F (MP)	Hardening Index	Viscosity @ 140°F	Hardening Index		
8	90.3	132	39	81	1674	1.55	1.69	19
11	49.3	67	90	38	4815	2.49	2.83	31
1	32.7	55	56	25	2853	1.64	4.94	74
6	32.3	55	88	37	3726	1.81	4.02	55
3	15.0	68	84	50	4356	2.00	4.31	51
		-0.33	0.30	-0.30	0.40	0.40	0.70	0.60

as a possible catalyst for hardening reactions, was studied.

Spearman's Rank Correlation Coefficients between each of these tests and the average recovered penetrations were determined and are given at the bottom of Table 11. None of them is significant. It must be concluded that none of these tests accurately predicts hardening susceptibility. However, with further refinement, the Relative Viscosity Test could prove valuable.

The final objective of the pizza study was to assess the effect of solar radiation on asphalt hardening. This was accomplished by locating replicate pizzas at each of the three exposure locations. Thirteen sets or blocks of pizzas were distributed in this manner. Each block consisted of three pizzas made from the same asphalt and aggregate and having approximately the same void and asphalt contents. This type of arrangement lent itself well to a statistical technique known as Randomized Block Analysis (RBA). The purpose of RBA is to show whether differences in treatment means are more than can be explained by random fluctuations in the data. In this case the treatment means were the three different exposure locations: control (inside, covered), Bryan and El Paso. The average annual solar radiation for these locations is as follows (4):

<u>Location</u>	<u>Average Annual Radiation (Langleys/Day)</u>
Control	0
Bryan	425
El Paso	535

If solar radiation significantly affected asphalt hardening, it was expected that pizzas stored at the El Paso location would certainly

TABLE 12. Results Used in Solar Radiation Analysis.

Block	CONTROL				BRYAN				EL PASO			
	Penetration	Viscosity	S <sub>BIT</sub>	M <sub>R</sub>	Penetration	Viscosity	S <sub>BIT</sub>	M <sub>R</sub>	Penetration	Viscosity	S <sub>BIT</sub>	M <sub>R</sub>
1	24	16.0	30	2.8	46	10.8	5	2.6	45	4.8	5	2.6
2	56	12.4	3	2.1	98	2.6	1.5	1.3	29	13.2	18	3.2
3	16	27.0	80	2.1	19	32.0	50	2.0	27	14.4	20	3.0
4	14	25.0	200	1.5	16	40.0	70	3.6	19	30.0	50	2.4
5	56	5.0	3	1.3	52	5.4	3	1.7	75	3.2	1.5	1.7
6	50	10.2	4	2.5	23	18.6	35	2.8	29	17.2	18	2.6
7	14	26.0	300	2.6	31	16.4	11	3.3	38	10.8	6.5	1.5
8	37	13.6	10	2.6	19	17.0	50	2.6	28	12.4	22	3.1
9	16	31.0	90	3.8	14	68.0	100	2.4	27	17.2	20	3.5
10	34	14.8	9	2.0	27	22.0	20	2.6	17	26.0	60	3.5
11	28	13.6	17	1.6	28	16.6	18	2.3	32	19.6	10	2.7
12	39	21.0	7	4.1	15	27.0	100	4.4	28	18.8	20	3.3
13	37	7.6	7	2.0	16	34.0	70	2.6	27	14.0	21	2.3



be in a more hardened condition than those stored at the control location. A summary of the data in Table 12 shows that this was not so. Results for the four indicators showed no overall pattern between the storage locations. This was confirmed by the RBA which yielded insignificant F-Ratios.

It was decided to test the asphalt properties of the top 3/8-inch of some of the Bryan and Control pizzas and compare the measured viscosity with that of the asphalt from the entire 2-inch thick specimen. Data are presented in Table 13. In this way the volume of asphalt actually affected by solar radiation was isolated so that lower layers of asphalt could not confound the results. Also, a gradient or hardening versus depth analysis was made on some of the pizzas from the El Paso exposure. Data from these tests are presented in Table 14. A gradient was not available from these test data.

TABLE 13. Solar Analysis of Bryan And Control Specimens

(Top 3/8" Vs Full 2" Core)

Pizza No.	Pen. @ 77°		Visc. @ 77°, (x10 <sup>6</sup> )	
	Top	Full Core	Top	Full Core
29	30	37	18	7.6
74		24		16
84	28	56	9.6	12.4
160	20	37	32	13.6
228	128	56	0.2	5.0
Lab No.				
73	21	19	16.4	17
92	33	16	12	34
103	65	46	4.0	10.8
106	36	52	7.6	5.4
113	28	98	13.2	2.6

CAMPUS (CONTROL)

BRYAN, TEXAS  
TAMU RESEARCH ANNEX

TABLE 14. Solar Gradient Analysis

Pizza	Layer	Top Bottom		R & B, °F
		Pen. @ 77°	Visc. @ 77° (x10 <sup>6</sup> )	
A-4	A	35	7.8	132
	B	40	7.6	128
	C	43	15.2	126
	D	34	20.0	134
E-7	A	34	36.0	139
	B	23	14.4	138
	C	18	30.0	142
	D	33	26.0	134
A-5	A	28	22.0	137
	B	88	16.0	116
	C	113	19.2	112
	D	53	4.0	120
C-7	A	19	34.0	143
	B	55	4.7	123
	C	16	37.0	145
	D	11	14.0	148

## HARDENING MODEL

In order to specify asphalts that will best resist thermal cracking a method for predicting asphalt hardness from original properties is needed. Ideally, this predictive model should yield the in situ hardness at any time from shortly after lay-down to at least ten years later. It should employ basic, easily obtained parameters and be reasonably repeatable.

The first step in devising such a model involved study of asphalt hardening rates for the various field test sections. The objective was to find a generalized model to describe the basic relationship between asphalt hardening and time. Data for penetration and viscosity at 77°F were available in some combination of the following time intervals:

<u>1 1/2-Year Pavements</u>	<u>6-Year Pavements</u>	<u>9-Year Pavements</u>
1 Day	1 Day	1 Day
5 to 8 Months	1 Week	2 Weeks
15 to 20 Months	1 Month	4 Months
	4 Months	1 Year
	1 Year	2 Years
	2 Years	3 Years
	6 to 7 Years	9 to 10 Years

Two generalized aging models evolved from this study: a viscosity model and a penetration model. They took the following form:

$$V = at^b$$

$$P = a + b \ln(t)$$

where,

V = Viscosity at 77°F.

(megapoises, shear rate =  $0.05 \text{ sec}^{-1}$  sliding plate viscometer)

P = Penetration at 77°F.

(M.M., ASTM D-5)

t = Time from lay-down

(Months)

a & b = Non-dimensional coefficients derived from least squares regression analysis.

These general models consistently described the behavior of the age hardening curves obtained from the field test sites. Of the fourteen nine-year old pavements studied, all of the fitted penetration curves had a correlation coefficient greater than 0.95. Figures 2 thru 15 show both viscosity and penetration hardening curves for these nine-year old pavements. Several outliers are noted for the viscosity curves, and are probably due to the highly complex and sensitive nature of the sliding plate viscosity test, especially for hardened materials. In general though, both types of curves follow the experimental results reasonably well.

The key to predicting the shape of either of these two generalized curves for a given pavement is the ability to predict the parameters "a" and "b". To be of practical benefit these values must be predictable before or, at least, during construction. They should also have physical interpretation and lack redundancy. In other words, "a" and "b" should be two independent and meaningful indices of age hardening. For this reason the penetration model was chosen over the viscosity model. In the

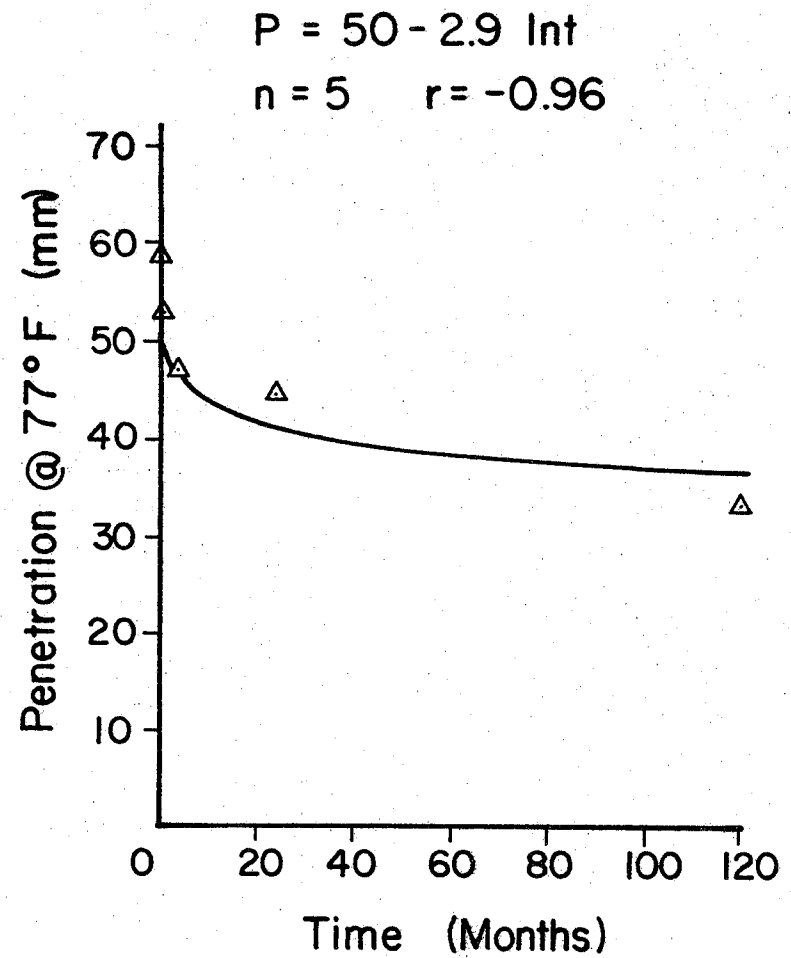
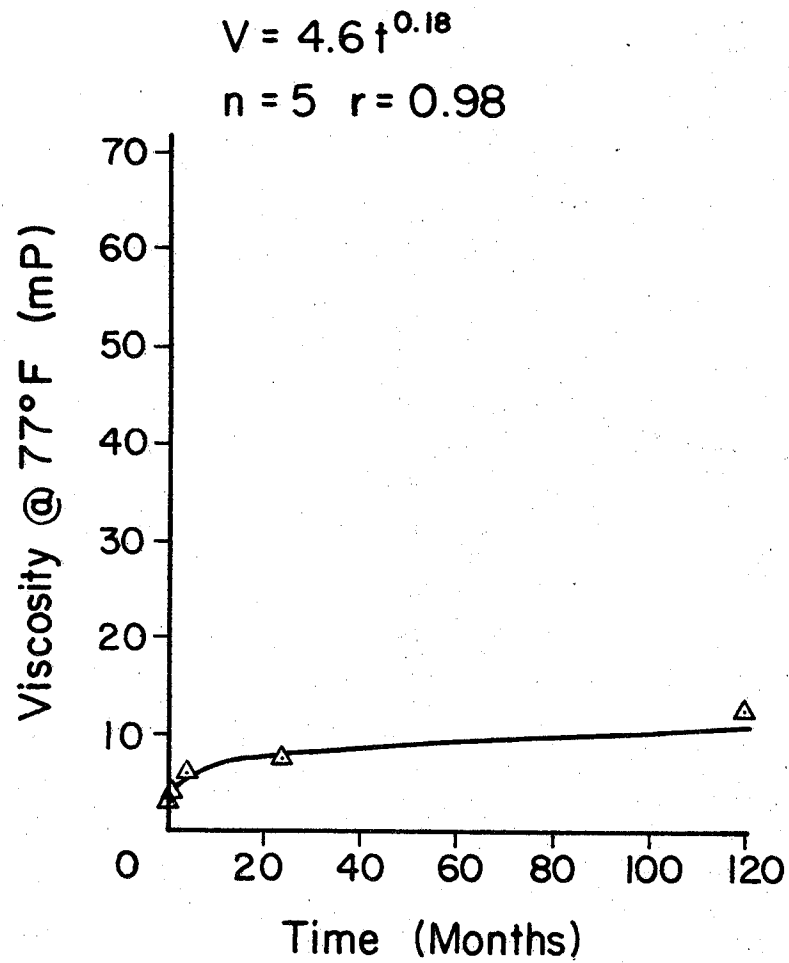


Figure 2. Hardening Curves For Site 38

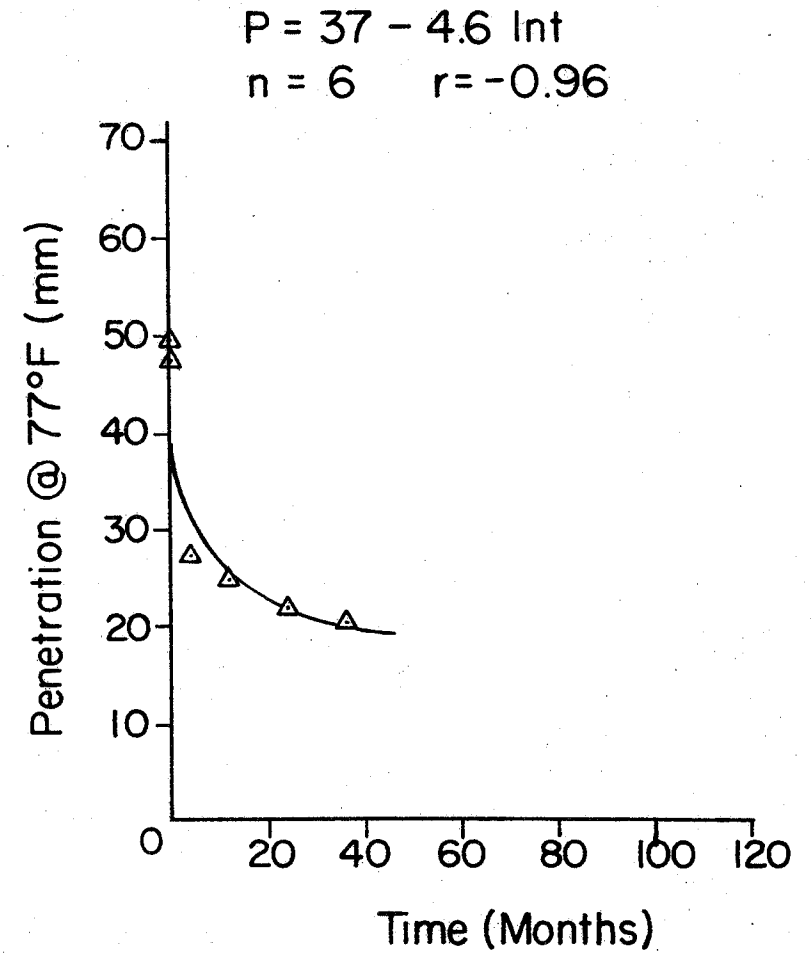
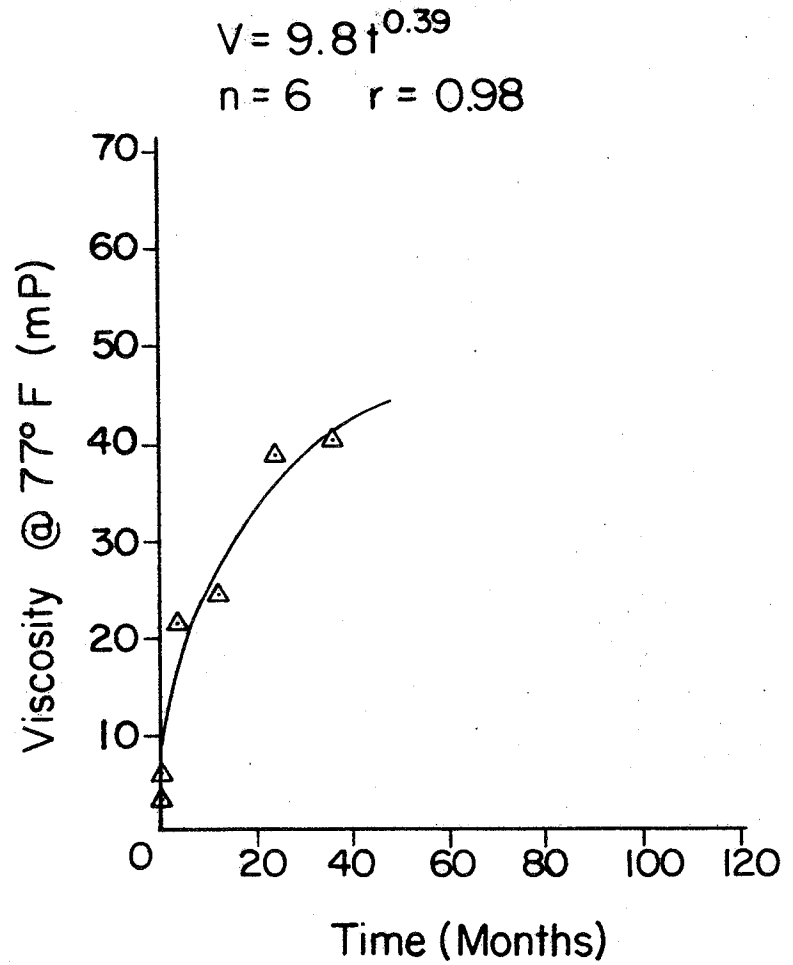


Figure 3. Hardening Curves For Site 39

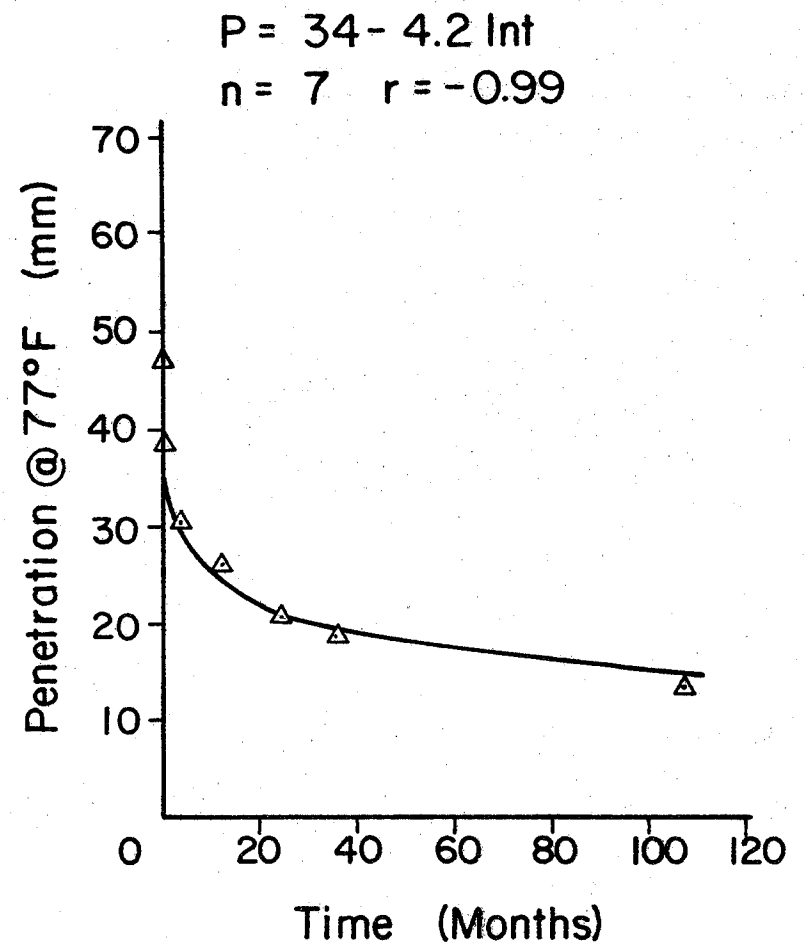
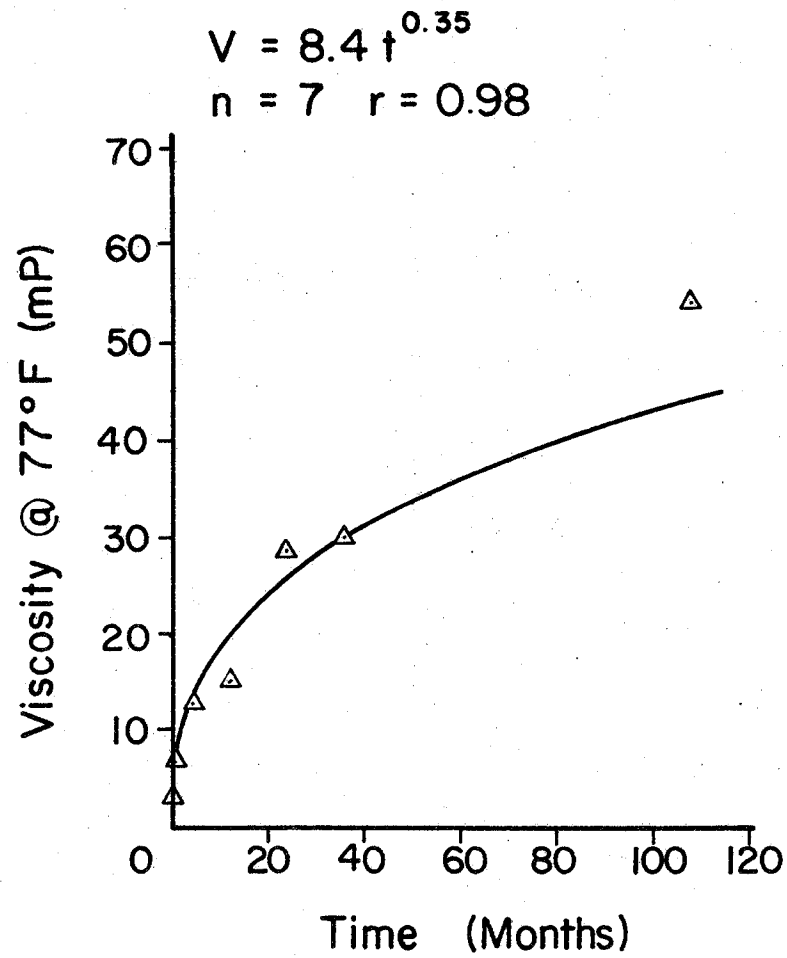


Figure 4. Hardening Curves For Site 40



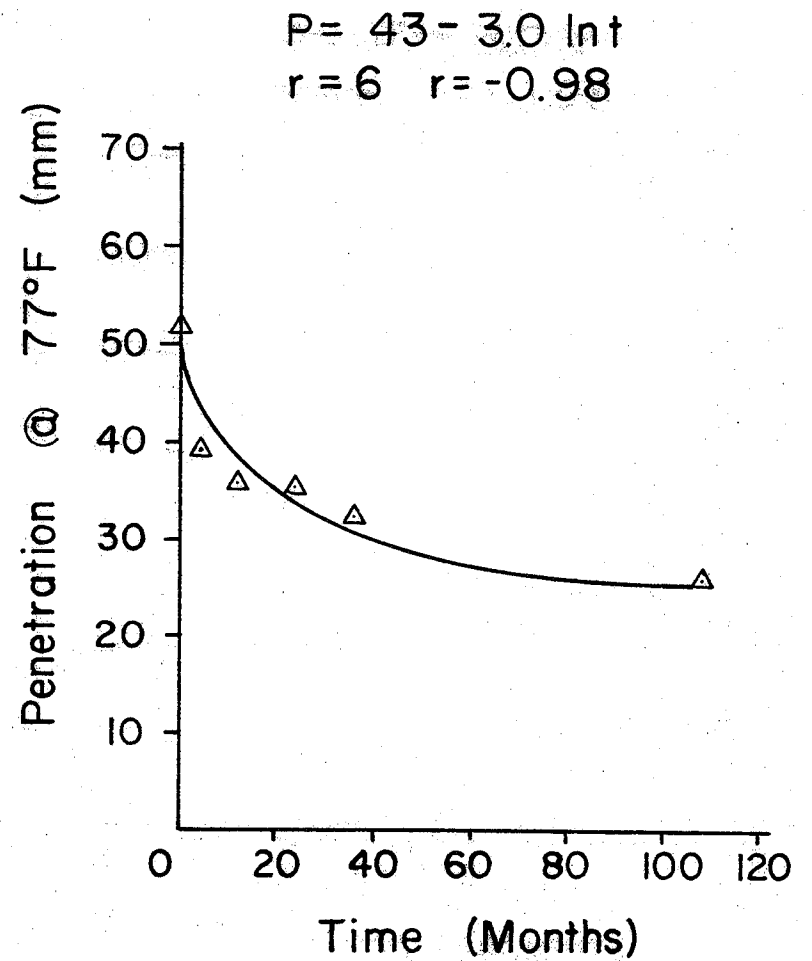
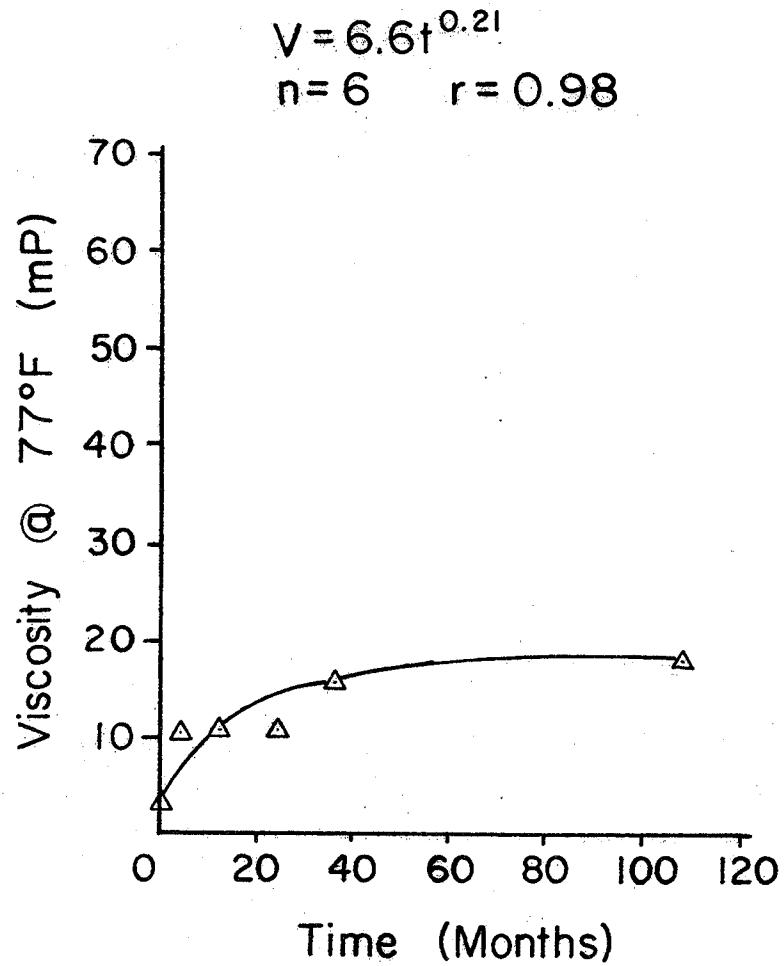


Figure 5. Hardening Curves For Site 41

07

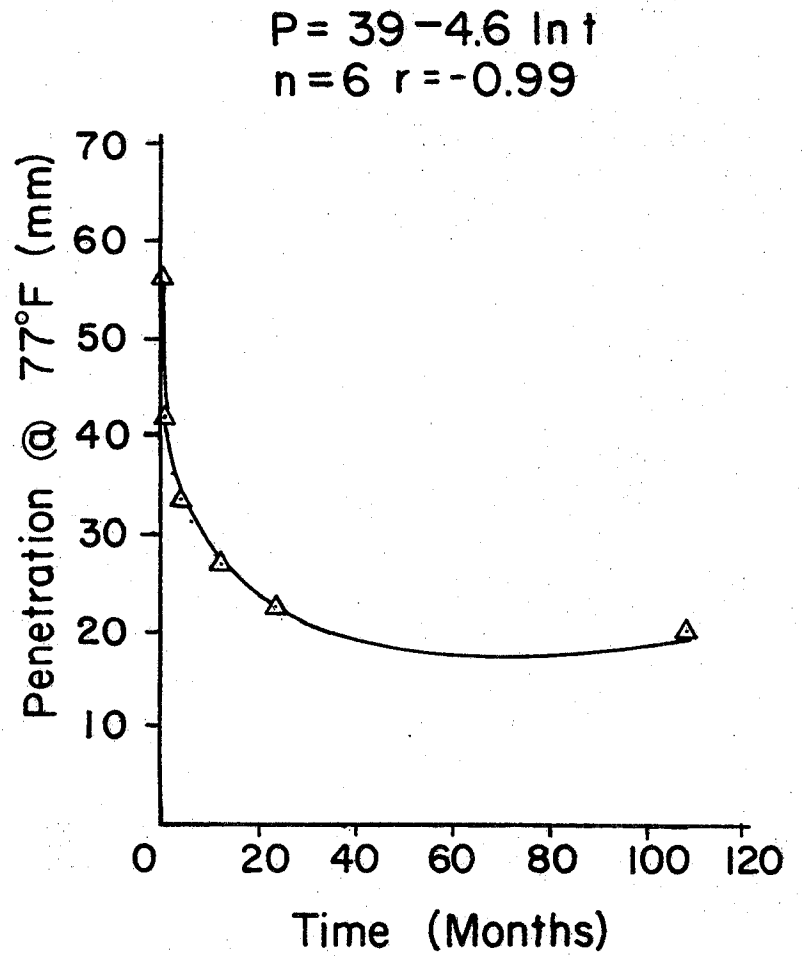
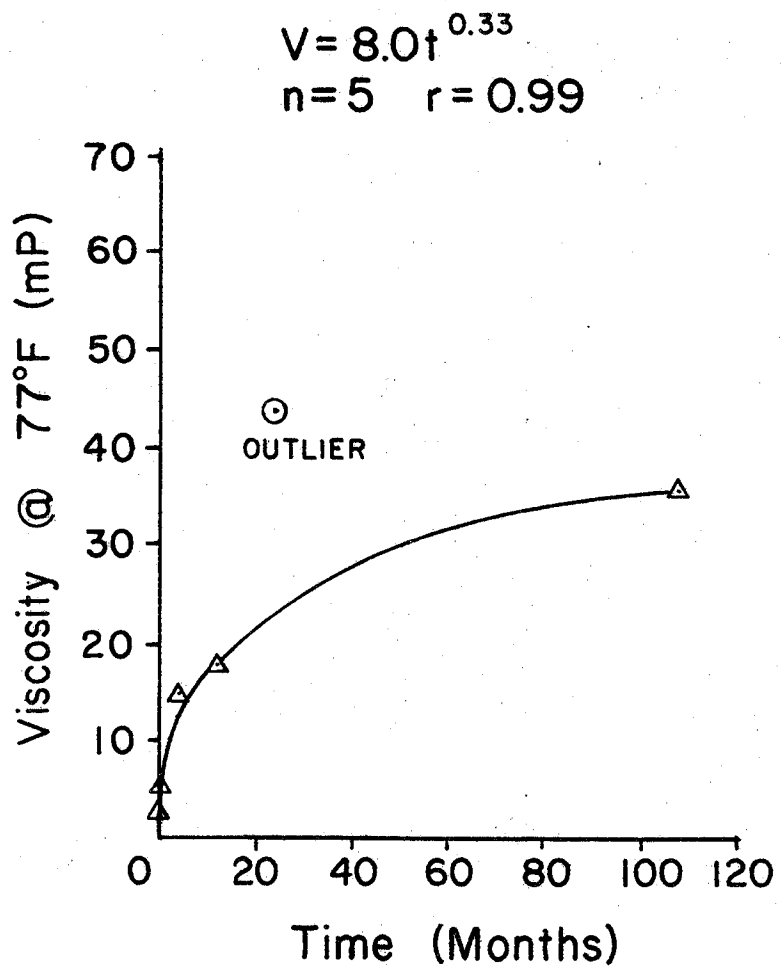


Figure 6. Hardening Curves For Site 42

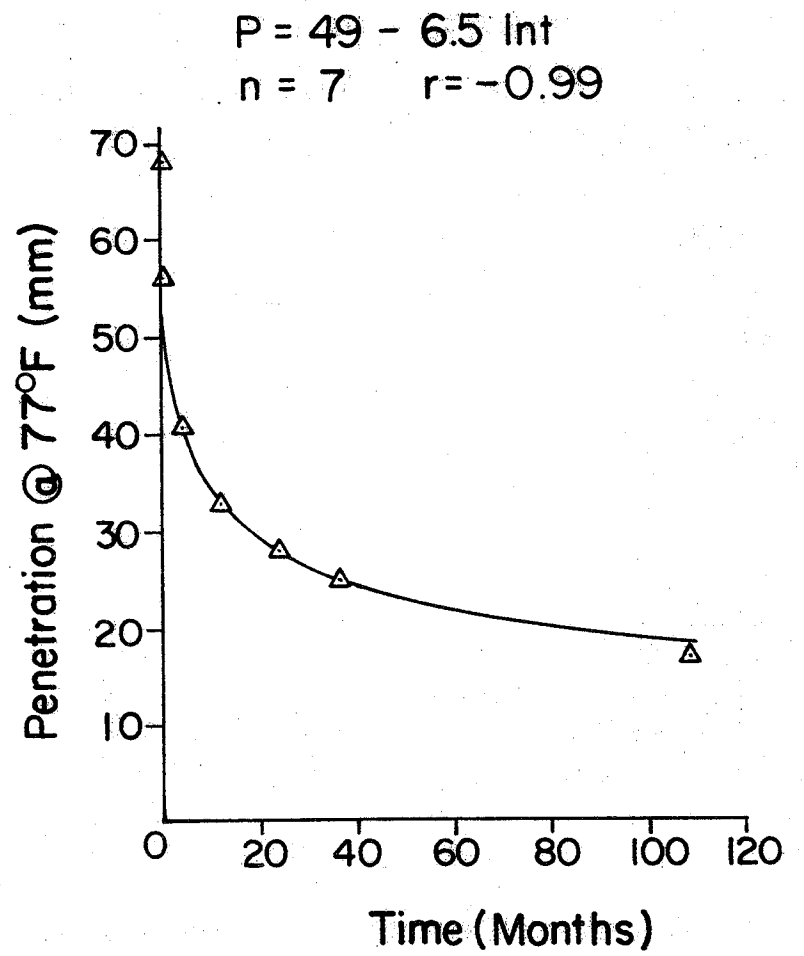
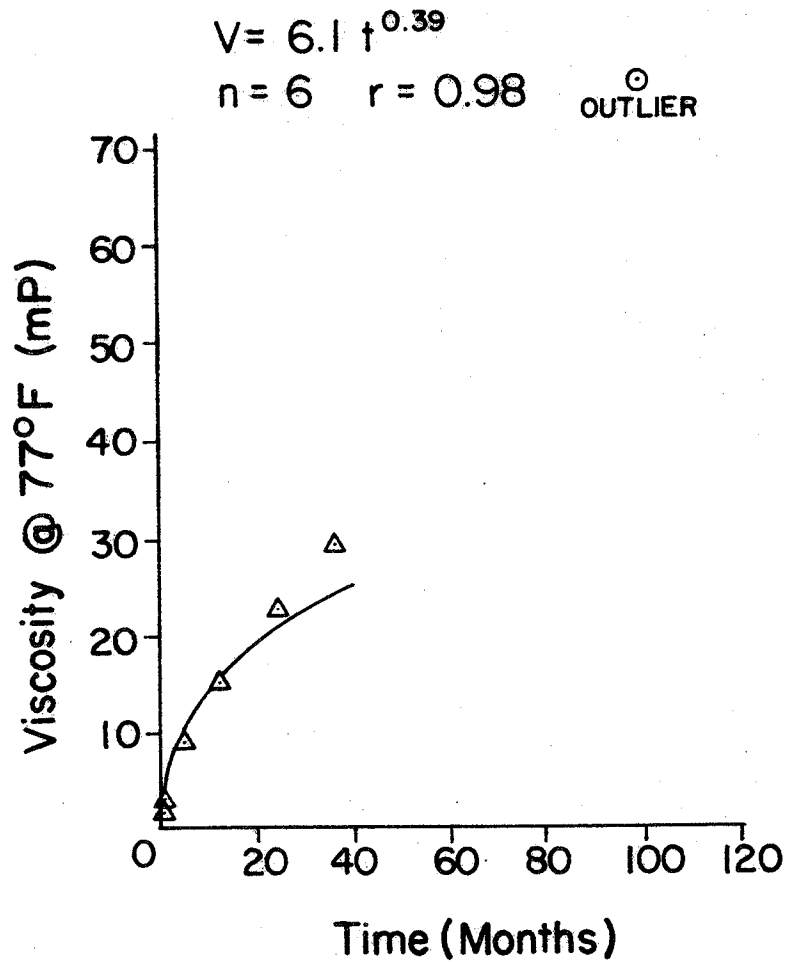


Figure 7. Hardening Curves For Site 43

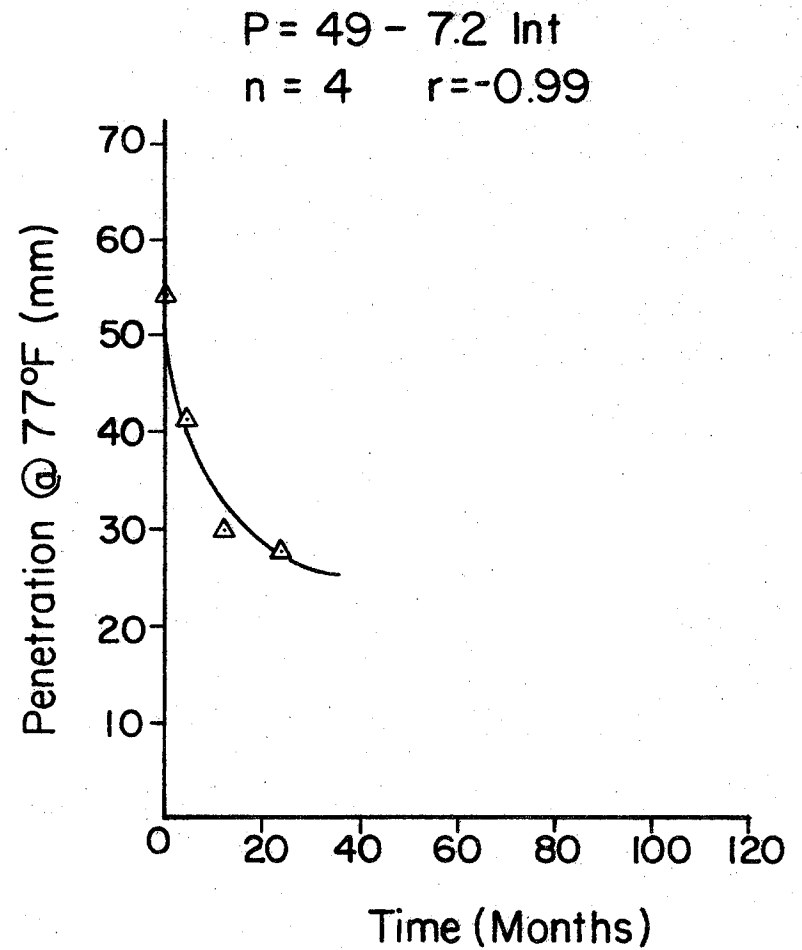
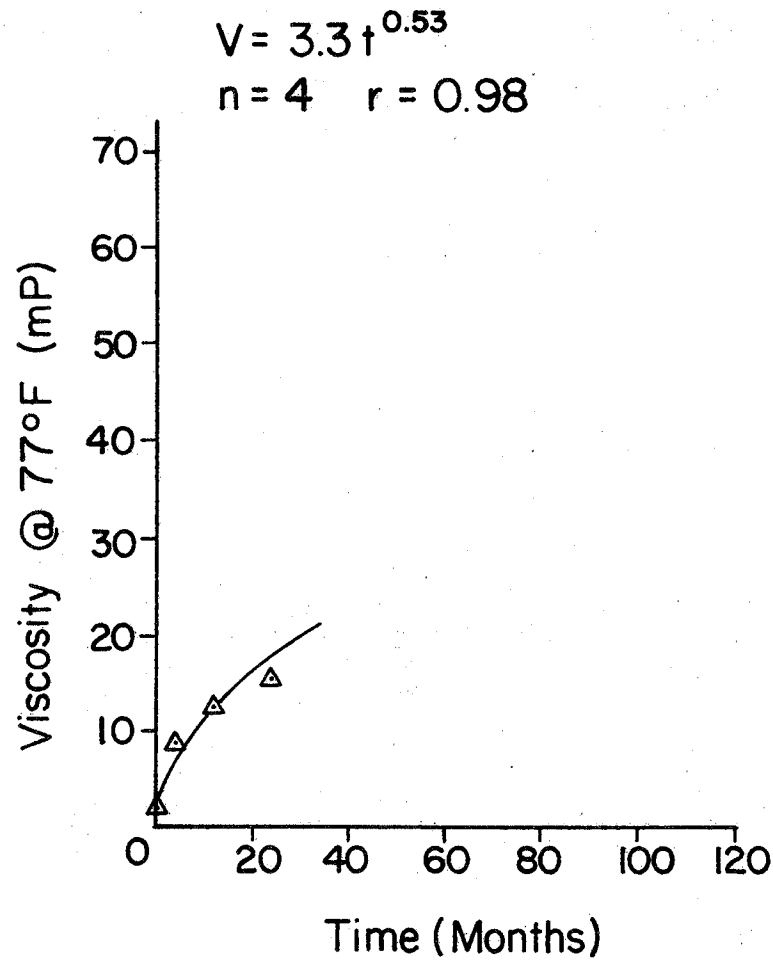


Figure 8. Hardening Curves For Site 44

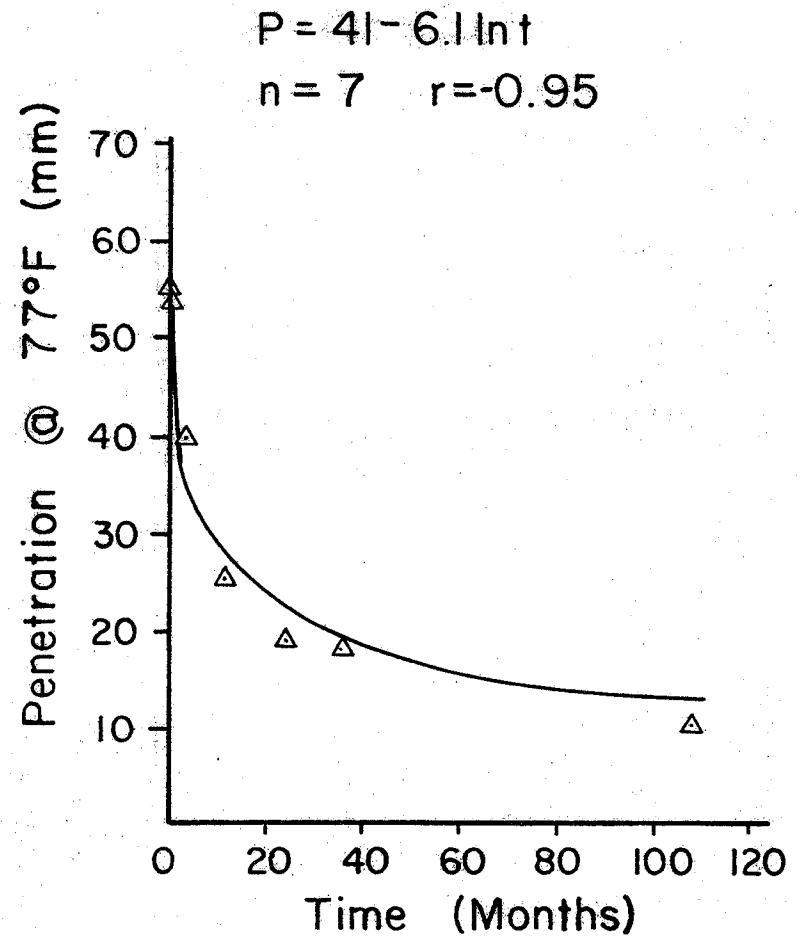
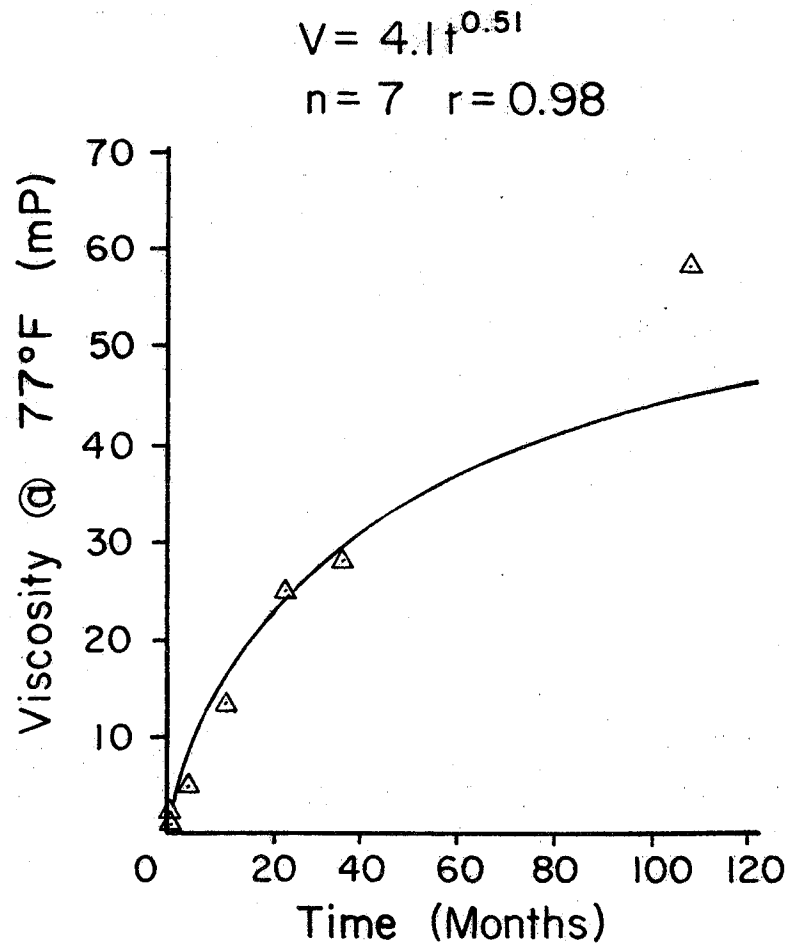


Figure 9 . Hardening Curves For Site 45

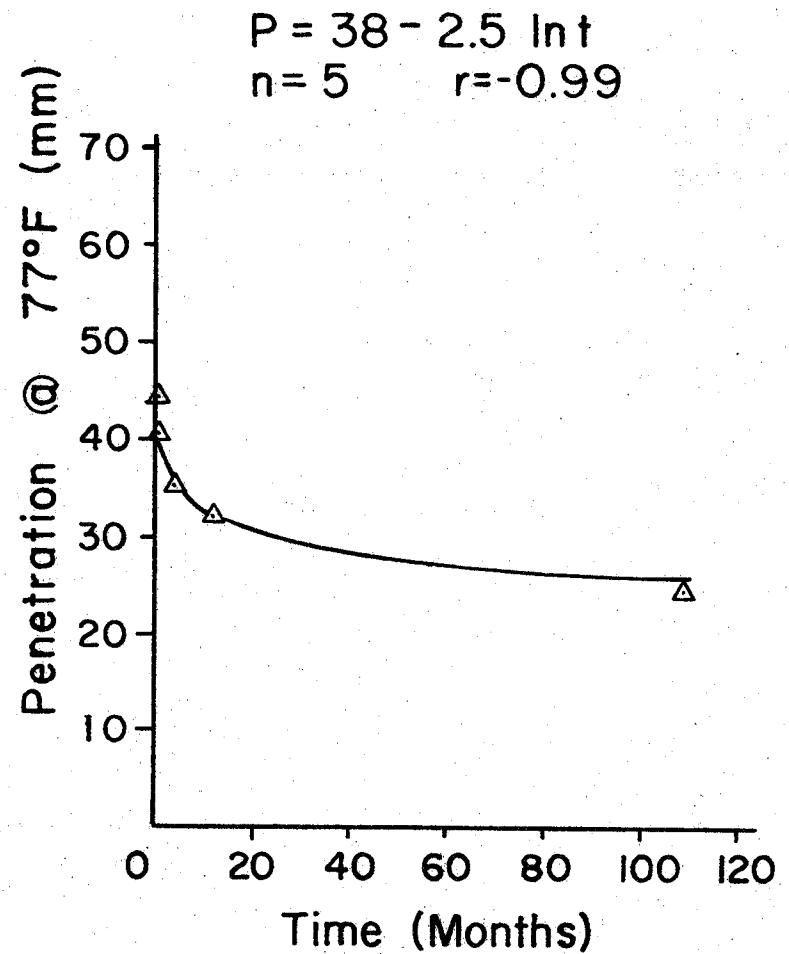
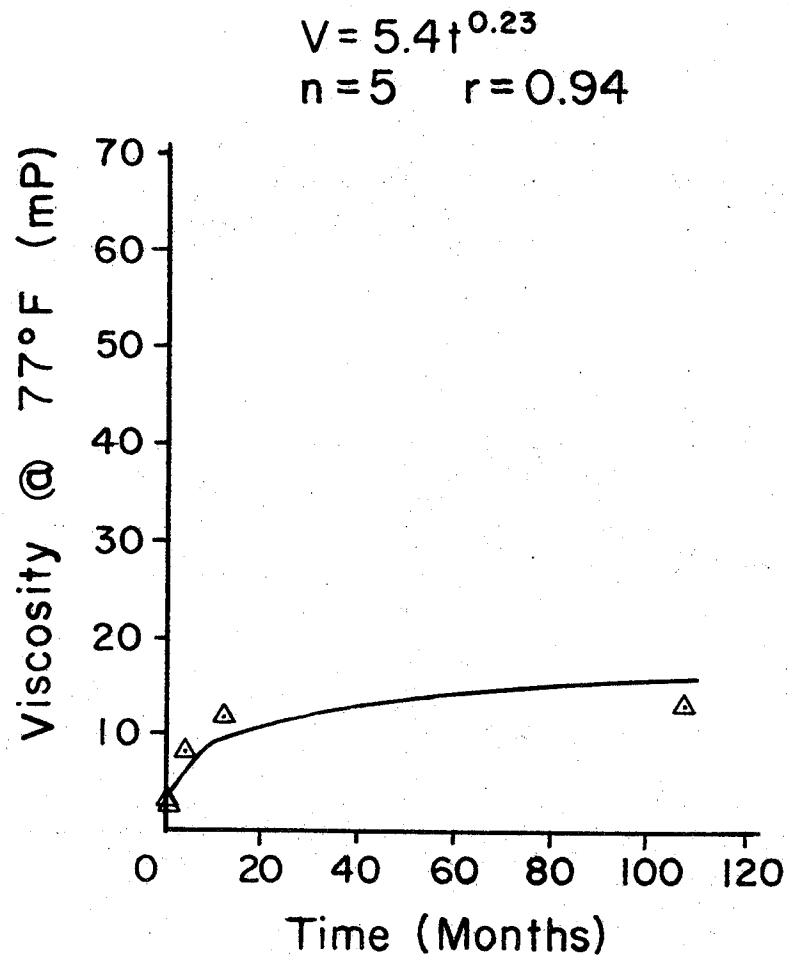


Figure 10. Hardening Curves For Site 46

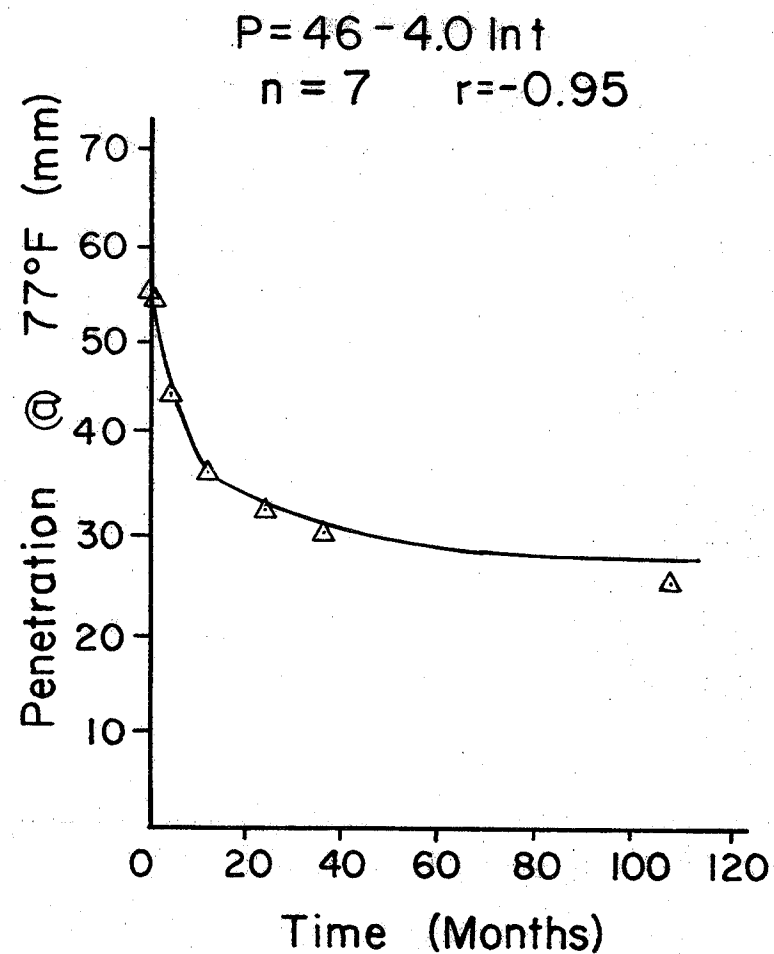
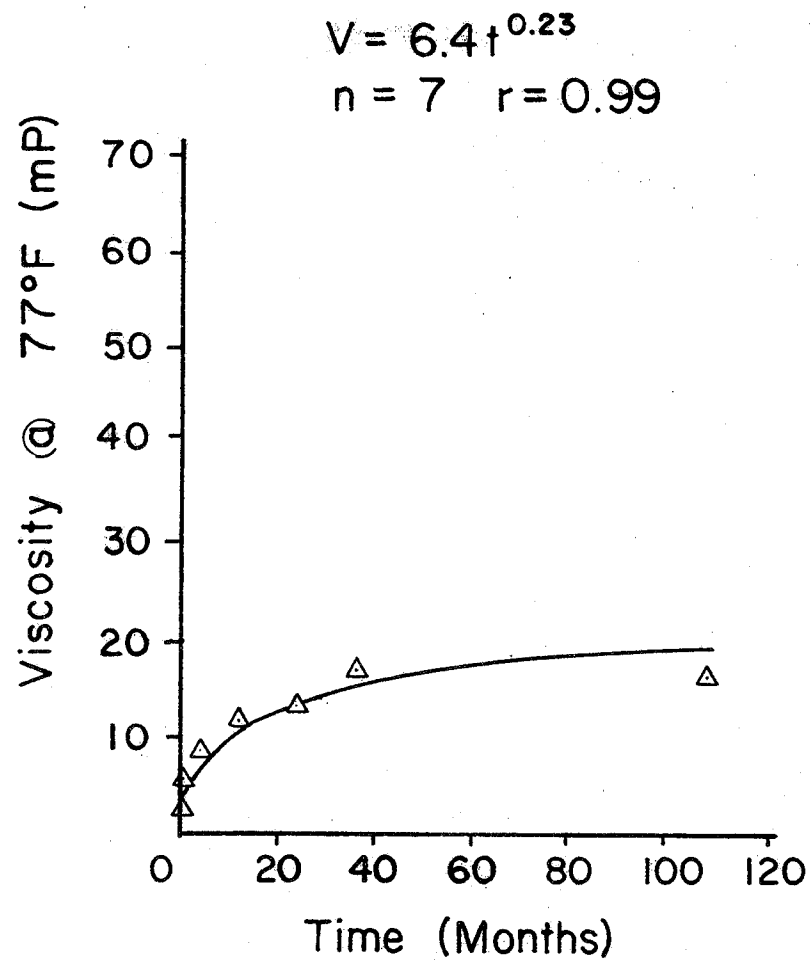


Figure II. Hardening Curves For Site 47

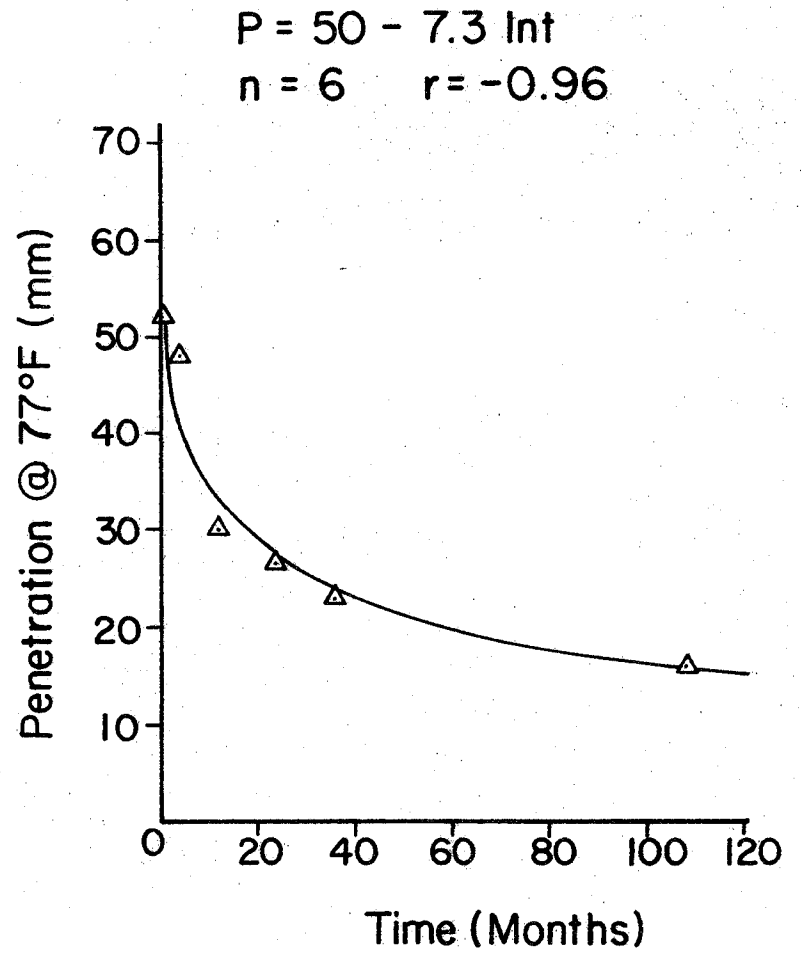
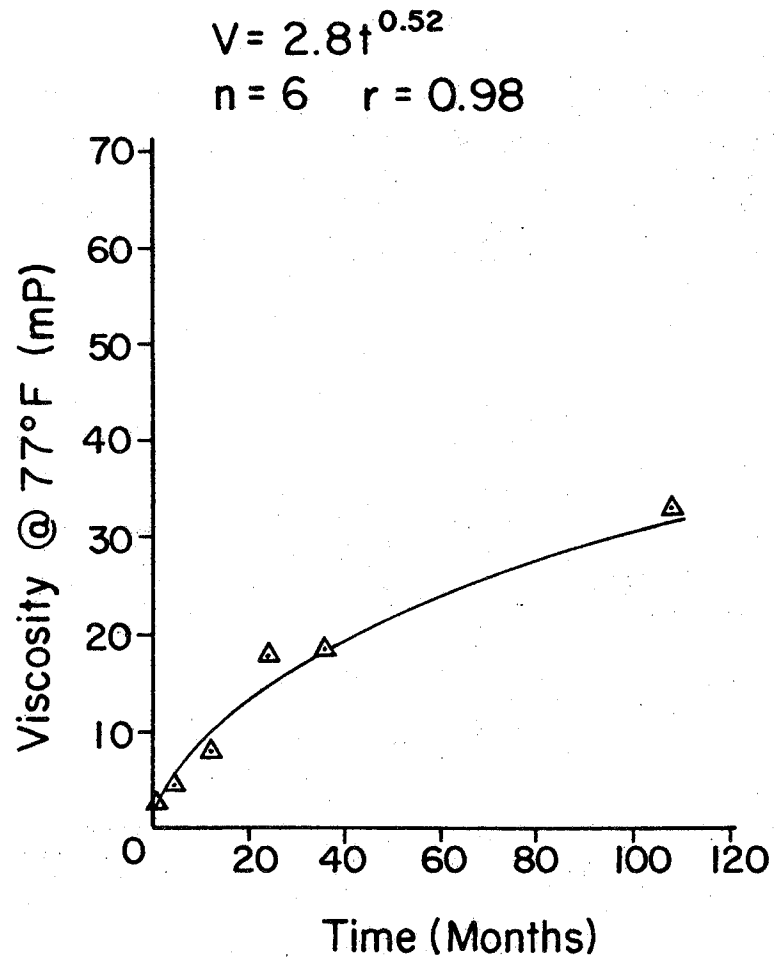


Figure 12. Hardening Curves For Site 48



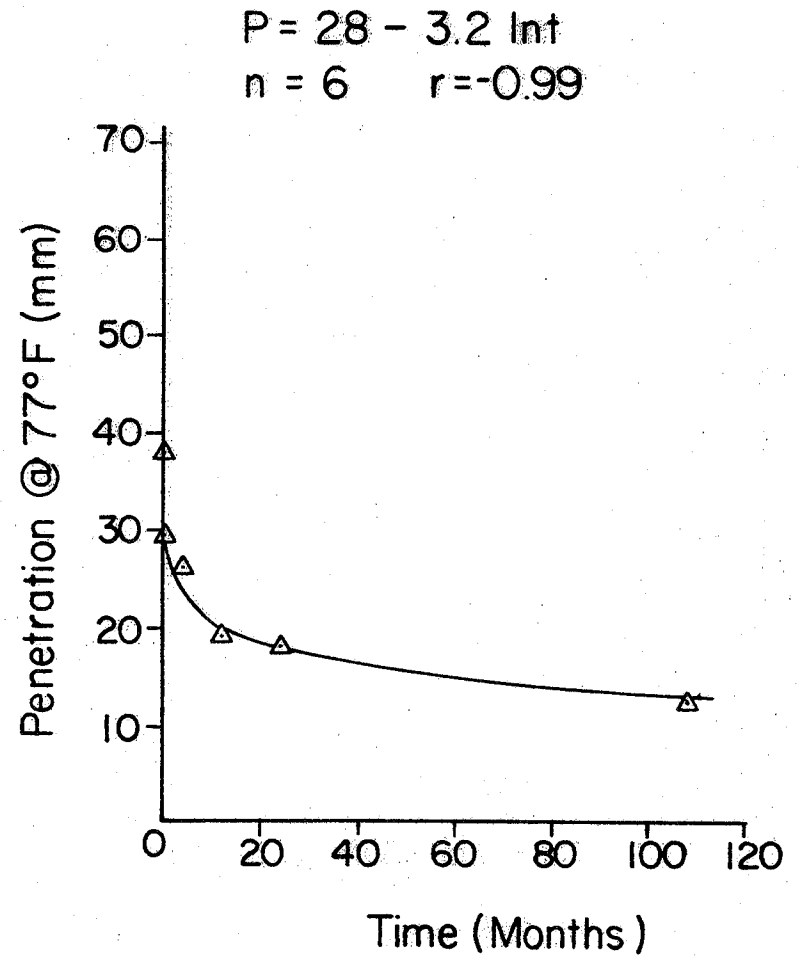
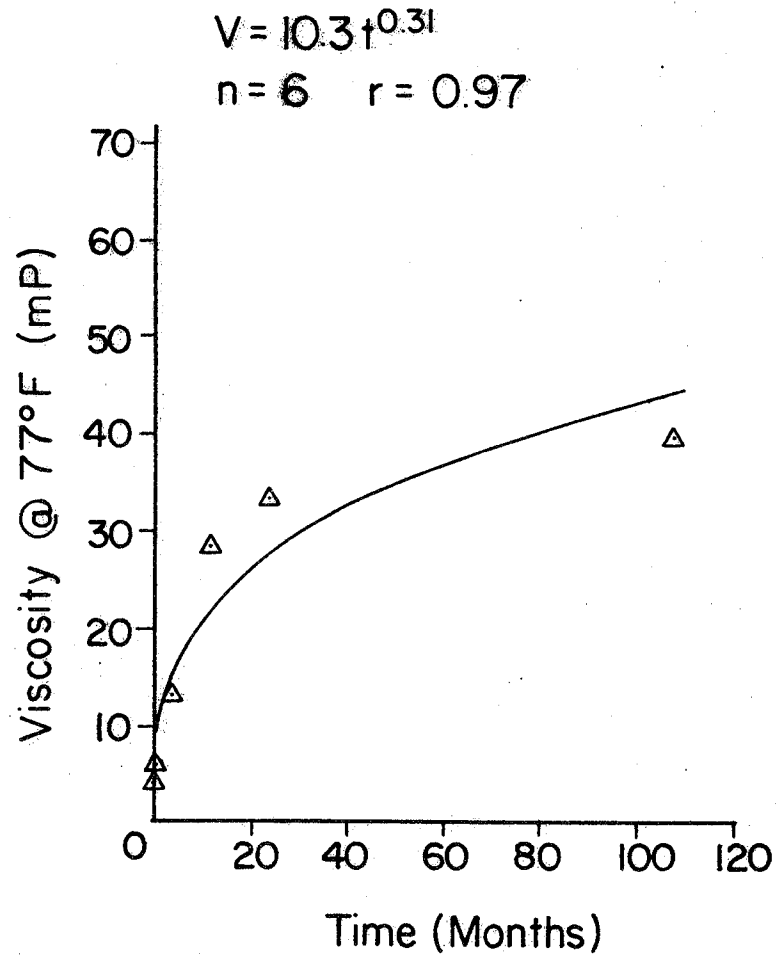


Figure 13. Hardening Curves For Site 49

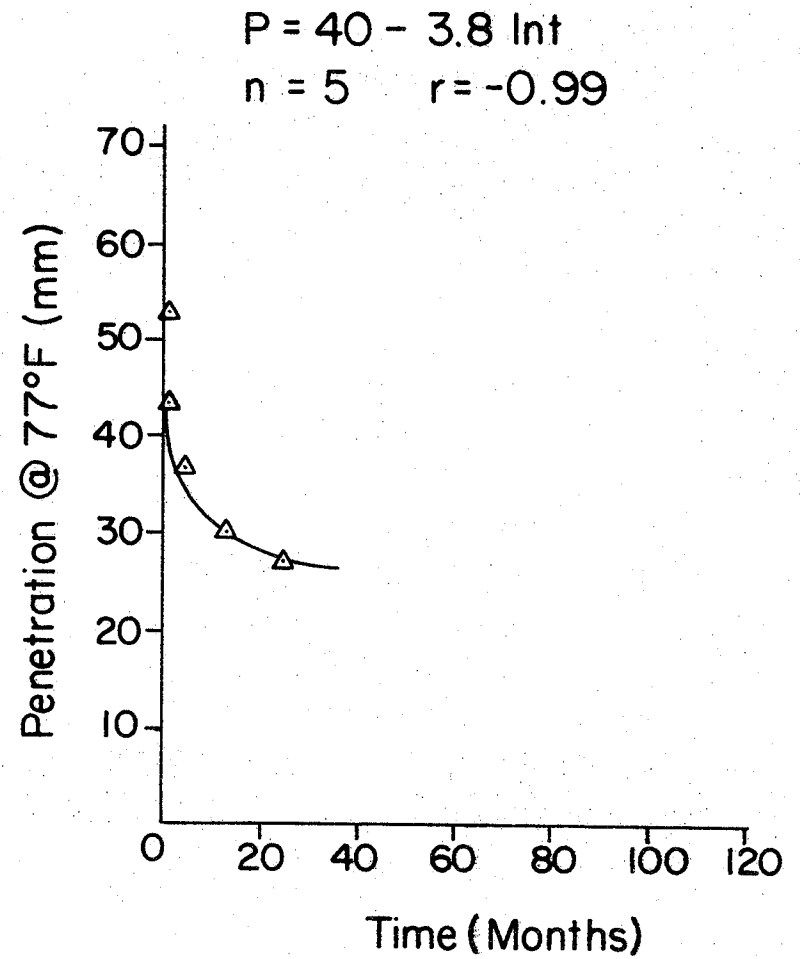
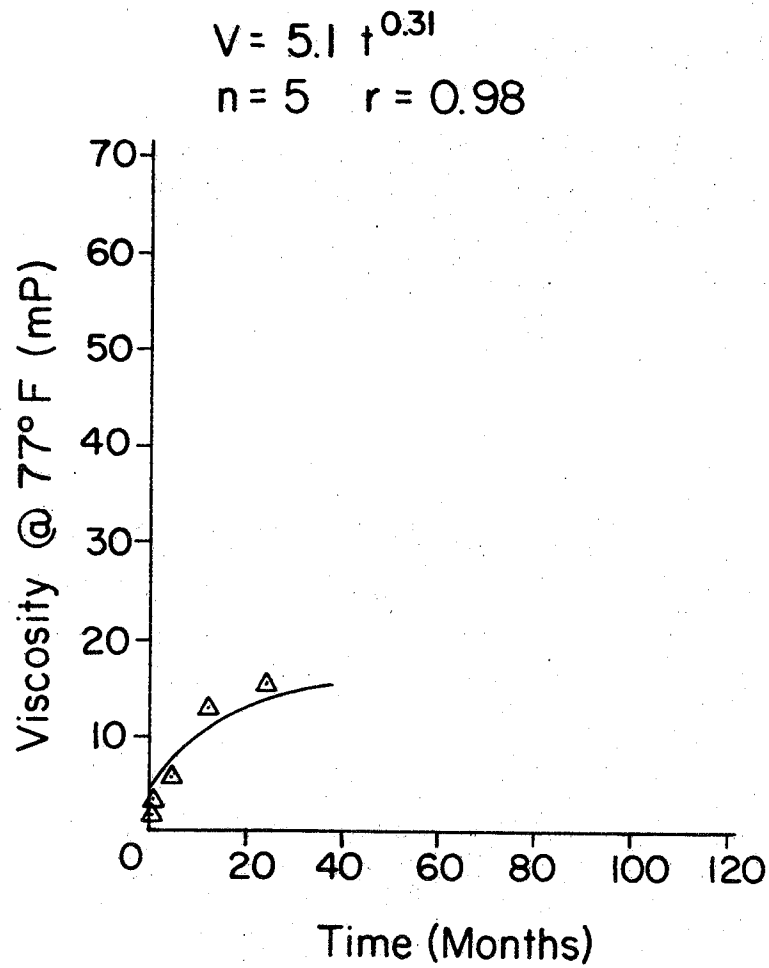
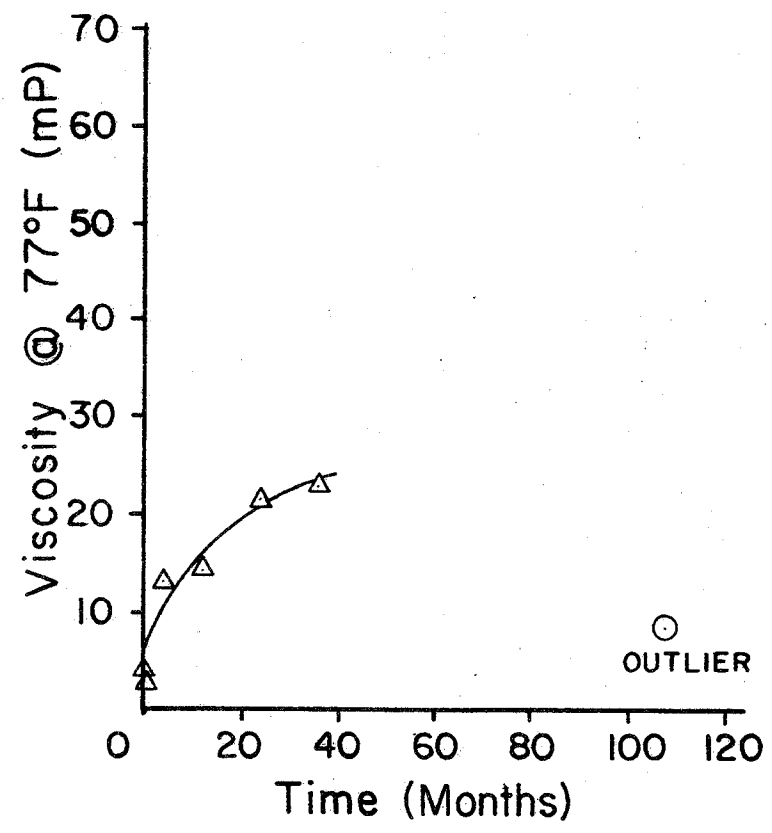


Figure 14. Hardening Curves For Site 50

$$V = 7.3t^{0.32}$$

n = 6    r = 0.98



$$P = 45 - 5.0 \ln t$$

n = 7    r = -0.97

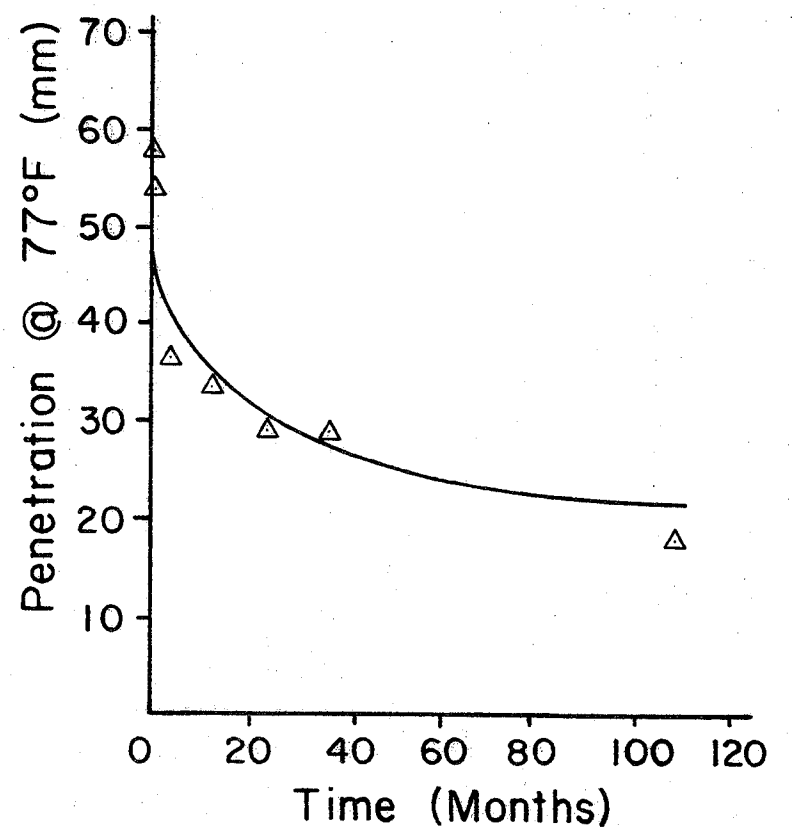


Figure 15. Hardening Curves For Site 51

penetration model "a" is a measure of short term or one month hardening (i.e.  $P = a + b \ln(t) = a$ ), while "b" influences only the model's curvature. The degree of curvature represents long-term susceptibility to hardening. In the viscosity model "a" and "b" reflect both short-term and long term characteristics of the hardening curve. Therefore, they are impossible to predict in any meaningful way. Other advantages to choosing the penetration model are its simplicity and universality.

The interpretation of "a" and "b" for the penetration model provides a starting point from which to build the full predictive model. It is expected that "a" will be a function of original asphalt properties, mixing conditions and asphalt susceptibility to mix hardening, original air voids and aggregate absorptivity. Whereas, "b" should be related to environmental factors and long term chemical reactions to which the asphalt might be susceptible.

A look at some of the hardening curves already presented shows the usefulness of such an interpretive model. Figure 2 (Site 38) represents an extremely durable asphalt which served well under one of the most harsh climates in the state (see Table 5). Nominal short term hardening occurred, followed by almost imperceptible long term hardening. The flat shape of the penetration hardening curve corresponds to a very desirable "b" of -2.9. Figure 4 (Site 40) represents a pavement in which severe short term hardening took place followed by moderate long term hardening. The original penetration of the asphalt used in this pavement was 90. After 24 hours it had dropped to 47, and by the end of one month was approximately 34. So in one month this asphalt underwent more hardening than the asphalt in the previous example had in ten years. Regardless of whatever resistance to long term hardening the pavement had, its

susceptibility to thermal cracking was already high. There are cases, however, where minimal short term hardening is followed by severe long term hardening. The effect is the same: a brittle pavement with high susceptibility to thermal cracking. Figure 12 (Site 48) provides a sample of this type of age hardening.

These examples clearly indicate that age hardening must be controlled on two fronts. The short term behavior is already being controlled to some extent through mixing temperature limitations and asphalt durability tests. The long term behavior is not so easily controlled. Weather patterns can be predicted and designed for to some extent, but complex, long term chemical reactions are not so easy to predict. Of the two schools of thought, accelerated simulations and constituent analysis, the latter seems to hold the best hope for the future. Unfortunately, results from such techniques as inverse gas-liquid chromatography were not obtained for this project. As has been seen, the actinic light test failed to prove a reliable indicator of long term hardening susceptibility.

With no reliable indicator available in the data, development of a predictive equation for the long term hardening index "b" was impractical. However, sufficient data were available to predict the short term hardening index "a". Control of this value can often make the difference between a pavement that will crack, and one that will not. This is easily seen by studying the penetration hardening curves for the pavements in the field performance analysis. Figure 16 shows that these curves can be roughly catalogued into two groups. In the upper group of curves virtually no transverse cracking occurred, while in the lower group transverse cracking was the rule, not the exception. The most noteworthy observation about these curves is that their long term hardening rates were very

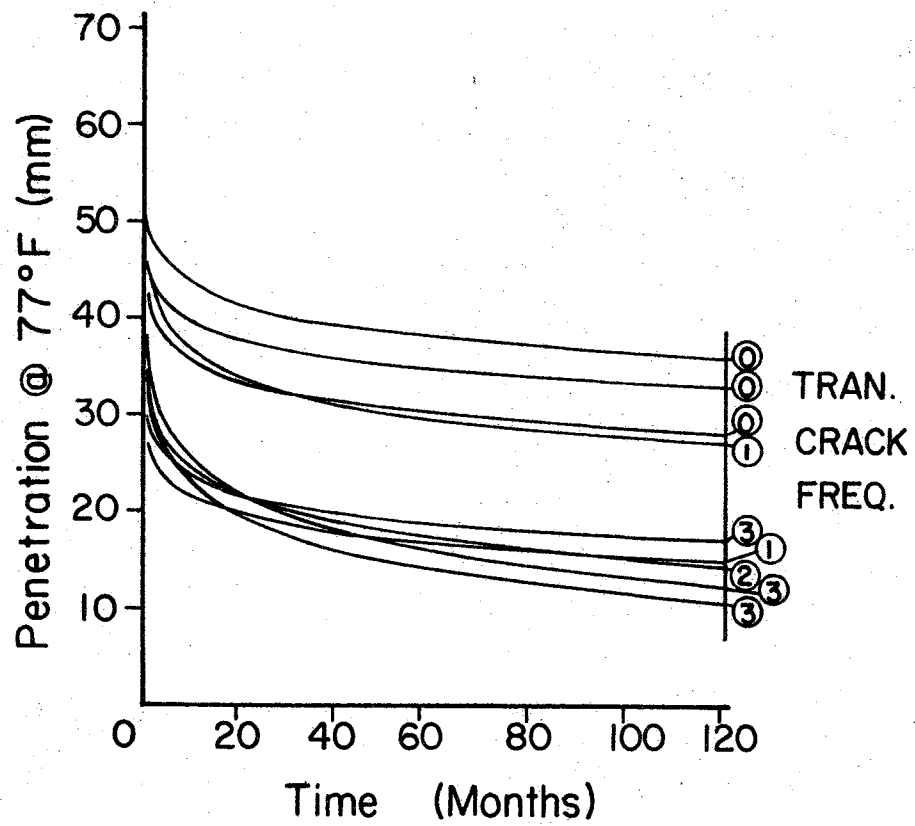


Figure 16. Penetration Hardening Curves For Test Sites In Table 4

similar. Division into the two groups correlated perfectly with their short term or one month penetration, not their long term hardening rates. Thus, a significant improvement in performance could have been expected if the short term hardening of the asphalts had been better controlled.

Several factors were looked at as potential predictors of short term hardening. These were:

- 1) Original penetration (prior to mixing)
- 2) Relative viscosity (or aging index test)
- 3) Void content (immediately after compaction)
- 4) Mixing temperature
- 5) Aggregate absorptivity (CKE)

Four and five were eliminated before multiple regression procedures commenced. All mixing temperatures were under the maximum 350°F allowed by Texas specifications and showed no correlation with "a". Only limited CKE data were available, but these did not indicate any significant correlation between aggregate absorptivity and short term hardening. The effect of both these factors, if any, was immeasurable.

Multiple regression analysis indicated that original void content and relative viscosity also had insignificant effects on the one month penetration value "a". Unfortunately, the field results were just not sensitive enough to assess the importance of these factors. This left original penetration as the overall indicator of one month penetration.

The equation yielded by the analysis is as follows:

$$a = 0.52 (\text{ORP}) - 2.0$$

where

ORP = Original penetration (MM)

It has a multiple correlation coefficient of 0.88, and a standard error

of the estimate of 5.9. This rather disappointing last statistic means one can only expect to predict one month penetration within  $\pm 12$  units. However, the equation can be used in conjunction with earlier field data as a sort of rough estimate of a maximum lower limit for original penetration of asphalts. Table 15 gives "a" and "b" values for all sites along with number of data points used and squared correlation coefficients for the penetration hardening model. Also, predicted values of "a" are shown.

Figure 16 shows that good resistance to thermal cracking can be expected if, after ten years, penetration of the extracted asphalt is no less than 25. From six- and nine-year old pavements in Table 15 can be seen that the worst long term hardening behavior that can be expected corresponds to a "b" of -7.3. These boundary conditions mean that "a" or one month penetration should be no less than 60 ( $25 = a^{-7.3} (\ln 120)$ ). From the prediction equation for "a", this would require an initial penetration of 120. Obviously, a method for controlling the "b" part of this equation is needed so that lower initial penetrations can be tolerated. The average "b" value for six- and nine-year old pavements in Table 15 is 4.0. Given this value, one month penetrations can run as low as 44 so that original penetrations should equal or exceed 88.

About 60% of the 1½-year old pavements studied in this project were built with asphalts having an original penetration less than 80. On the average, these pavements can be expected to develop transverse cracking within ten years, especially, if located in Central or West Texas. To combat thermally induced transverse cracking softer asphalts must be used. Naturally, accommodation for mixture stability at high summer temperatures must be made also. A better understanding of long term hardening mechanisms would certainly enhance the prospect of choosing asphalts that fulfill both these requirements.



TABLE 15. Summary of Penetration Model Data

$$P = a + b \ln t$$

Site No.	"a"	"b"	Number Points	r <sup>2</sup>	Predicted "a"
1	41	-5.8	3	0.97	43
2		Insufficient Data			
3		"	"		
4	69	-8.9	3	0.90	67
5	32	-2.4	3	0.98	48
6	37	-4.5	3	0.99	40
7	23	-2.4	3	0.98	29
8	17	-0.8	3	0.99	27
9	30	-3.4	3	0.85	33
10	14	-0.5	3	0.66	20
11		Insufficient Data			
12	33	-2.2	3	0.98	30
13	40	-3.5	3	0.92	39
14	31	-4.5	3	0.93	27
15	20	-1.3	3	0.80	25
16	45	-6.7	3	0.99	36
17	27	-1.6	3	0.86	33
18	53	-2.8	3	0.60	57
19		Insufficient Data			
20	42	-0.7	3	0.52	42
21*	28	-2.5	3	0.52	--
22*	30	-2.7	3	0.59	--
23		Insufficient Data			
24		"	"		
25	42	-5.7	5	0.91	37
26	30	-2.7	5	0.94	27
27	37	-4.7	4	0.97	36
28**	45	-2.5	6	0.25	--

TABLE 15 - (continued)

Site No.	"a"	"b"	Number Points	r <sup>2</sup>	Predicted "a"
29	39	-3.6	5	0.76	35
30	50	-3.0	4	0.65	42
31	34	-2.3	5	0.59	26
32	16	-1.1	5	0.79	21
33	22	-2.6	4	0.83	19
34	13	-0.7	4	0.92	17
35		Insufficient Data			
36	27	-2.6	4	0.73	28
37	38	-5.4	4	0.98	30
38	50	-2.9	5	0.93	38
39	37	-4.6	6	0.91	39
40	35	-4.2	7	0.98	40
41	43	-3.0	6	0.96	42
42*	39	-4.6	6	0.99	--
43	49	-6.5	7	0.99	44
44	49	-7.2	4	0.98	49
45	41	-6.1	7	0.91	38
46	37	-2.5	5	0.97	43
47	46	-4.0	7	0.91	41
48*	50	-7.3	6	0.92	--
49*	28	-3.2	6	0.98	--
50*	40	-3.8	5	0.98	--
51	45	-5.0	7	0.95	46

\*Omitted from multiple regression analysis because of missing data.

\*\*Omitted because of low correlation.

## FINDINGS AND CONCLUSIONS

1. It has been determined that thermally induced stresses are responsible for transverse cracking failures of some pavements in West and Central Texas.
2. Longitudinal cracks often accompany transverse cracking, and there is some evidence (see Figure 1) that transverse cracks may cause premature longitudinal cracks.
3. The amount of thermally induced transverse cracking is directly related to the hardness of the pavement binder. Therefore, it is desirable to control asphalt hardening susceptibility.
4. Control of asphalt hardening should be focused on the asphalt itself. Environmental differences within the West and Central Texas areas are not nearly as important to evaluating age hardening potential as are the pertinent characteristics of the asphalt. Therefore, a uniform specification can be applied to the entire area.
5. Asphalt hardening rates follow a generalized pattern. For penetration results this takes the form  $P = a + b \ln(t)$ . Here, the dimensionless factors "a" and "b" provide convenient indices of short and long term hardening, respectively. These correspond to the rational concepts of short term plant and placement hardening and long term environmental hardening. The value "a" equals approximately one half the original penetration. Undoubtedly, a more refined estimate of "a" could be developed. Further research may also reveal a reliable technique to predict "b", thus enabling the engineer to estimate the amount of age hardening that will take place over a given time period.

6. As a general rule of thumb, pavements located in West and Central Texas whose asphalt penetrations remain above 25 should not develop significant thermal cracking problems.

There is some doubt as to the sensitivity of the findings derived from analysis of the laboratory prepared pizzas. Ironically, experimental errors were much higher for the laboratory results than the field results. There were two probable causes for this. First, hand fabrication of the pizzas apparently yielded less uniform results than normal plant mixing and field placement of a 200-foot highway section. Second, recovery and testing of the laboratory specimens were handled by less experienced personnel than the earlier field tests. Thus, the large number of negative findings yielded by the laboratory analyses may have been due to a combination of large experimental errors and relatively small treatment effects. In looking at the remaining findings it should be remembered that asphalt type proved significant because it was a strong factor and was detectable despite the large experimental errors. The effects of other factors were either non-existent, or not strong enough to be detected.

7. Laboratory efforts and the hardening model analysis did not indicate a significant relation between mix density and asphalt hardening susceptibility. However, the field performance analysis showed a significant correlation between cracking, hardening and void content.
8. The additives investigated did not prove effective as asphalt hardening inhibitors after three years of field exposure.

9. Varying asphalt content  $\pm 1/2\%$  from design optimum did not prove to be critical to hardening susceptibility.
10. The Actinic Light Test, Thin Film Oven Test, Relative Viscosity Test and Vanadium Content did not prove to be reliable indicators of hardening susceptibility.

## IMPLEMENTATION

1. Until a reliable indicator of long term hardening susceptibility is perfected and integrated into a total asphalt hardening model, only asphalts with an initial penetration of 80 or more should be used for highway construction in West or Central Texas. If mitigating circumstances dictate use of a harder asphalt, the relative viscosity should be low (less than 3) and the mixture density high. This may provide some protection against thermal cracking.
2. Predictive models for asphalt hardening should treat short term plant hardening and long term environmental hardening as separate phenomena. It is suggested that the penetration model discussed in this report could form the basis for such a model. The penetration model has been proven to be generally applicable for pavements throughout Texas, and its form allows prediction to be separated into its natural division of short and long term components.
3. More research relating asphalt constituent analysis and aggregate interactions to long term asphalt hardening susceptibility is needed. Constituent analysis is more promising than accelerated testing because it can be correlated to asphalt hardening behavior with or without knowledge of long term asphalt hardening reactions. On the other hand, reliably simulating these complex reactions in the space of one or two days seems an unlikely prospect.

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