Sulfur Extended Asphalt Pavement Evaluation

SR 270 Highway Pavement Performance
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16. Abstract

This report summarizes the placement and performance of sulfur extended asphalt (SEA) paving mixtures at a highway test site (SR 270) near Pullman, Washington. The report includes a summary of the mixture and structural designs and construction details. This is followed by a discussion of the data collection and analysis accomplished over a three year evaluation period (1979 - 1982). A major experimental feature of the study was the use of 0/100 (conventional asphalt concrete), 30/70 and 40/60 SEA binder ratios (sulfur/asphalt ratios are expressed as weight percents) in the experimental paving mixtures.

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SULFUR EXTENDED ASPHALT PAVEMENT EVALUATION IN THE STATE OF WASHINGTON: SR 270 HIGHWAY PAVEMENT PERFORMANCE REPORT

by

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Acknowledgement is made to the Washington State Department of Transportation, Federal Highway Administration, the Sulphur Development Institute of Canada and the Asphalt Paving Association of Washington for their interest and support of the study.

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CHAPTER I

INTRODUCTION

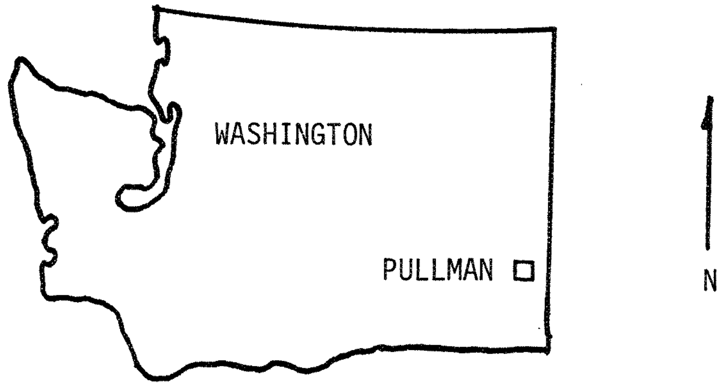
A number of laboratory-analytical studies have been conducted by various research organizations to investigate the effect of combining sulfur, asphalt and various aggregates in paving mixtures. Additionally, a number of full-scale experimental highway projects have been built in the United States, Canada and elsewhere using various combinations of these materials.

Much of this work has been reported and some research efforts, particularly the field trials, are still in progress. It has been observed that, in general, the laboratory-analytical studies have shown the use of sulfur extended asphalt (SEA) binders to be promising, and possibly superior to conventional asphalt concrete paving materials. The results of the full-scale experimental highway projects are, at this time, somewhat inconclusive. This is not to say that compared to conventional paving mixtures sulfur-asphalt bound pavement materials are performing poorly under normal highway traffic and realistic environmental conditions.

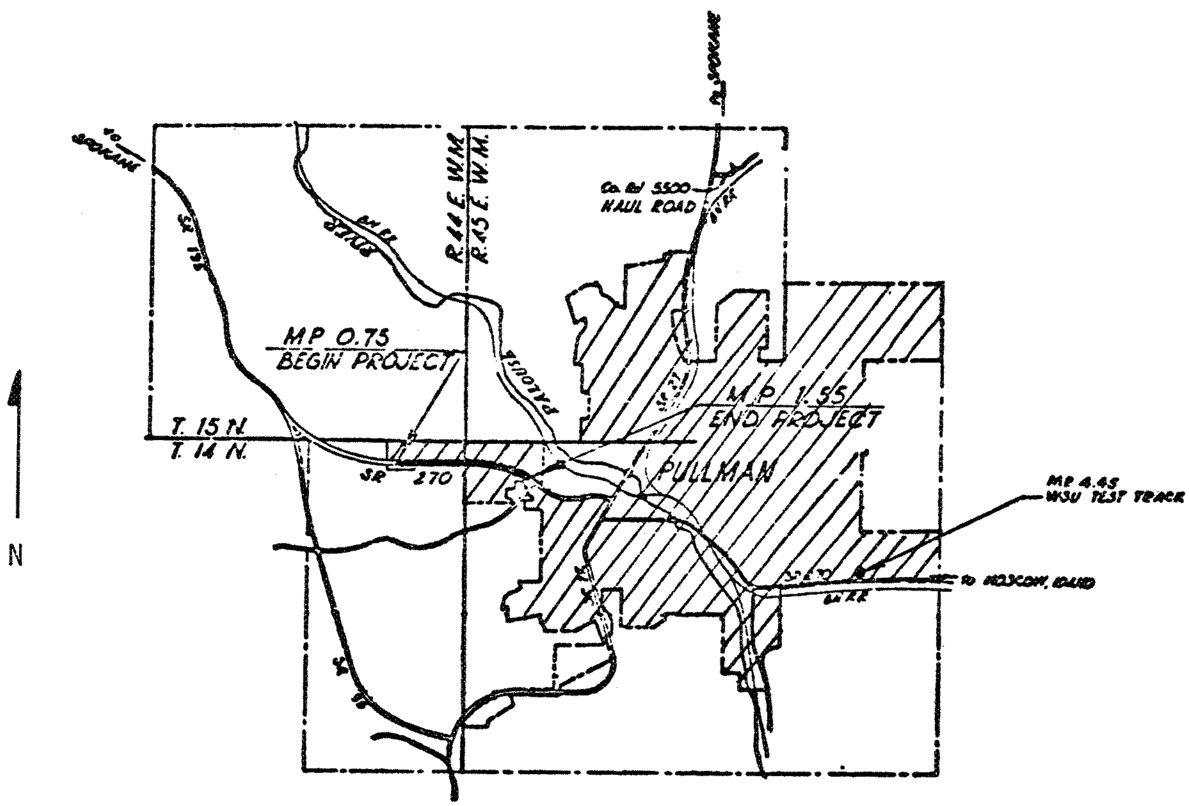
The study being reported is intended to help bridge the gap between the laboratory-analytical studies and the full-scale experimental highway projects. This work comprised building full-depth pavement structures for repetitive wheel load testing at the Washington State University G.A. Riedesel Pavement Test Facility (hereafter identified as the "Test Track") as well as construction and evaluation of a companion highway project. Both types of test pavements were located in the immediate vicinity of Pullman, Washington (Figure 1), and were constructed in August, 1979. This experimental configuration allowed for the concurrent construction of both the test track and highway project. Thus, the same materials and central batch plant were used for both jobs.

The sponsors for the study included the Washington State Department of Transportation (WSDOT), Federal Highway Administration (FHWA), Sulphur Development Institute of Canada (SUDIC), and the Asphalt Paving Association of Washington. The prime contractor for the conduct of the study was the University of Washington (UW) with Washington State University (WSU) as subcontractor. The Washington State Department of Transportation provided substantial funding for the study as well as participation in the construction and evaluation of the test pavements.

This is the third and final report for the study. The report will be used to present a summary of the performance of the highway test sections on SR 270. Further, in the sections which follow in this chapter, a brief overview of the mixture and structural designs will



(a) Location of Pullman, Washington



(b) Vicinity Map of Test Sites

Figure 1. Location of Experimental Pavements

be presented along with a discussion of the construction phase. Previously issued study reports are:

1. Design and Construction Report, September, 1981.

2. Test Track Pavement Performance Report, January, 1982.

MIXTURE AND STRUCTURAL DESIGNS

The pre-construction laboratory mix designs are summarized in the initial study report [1]. However, additional material and mixture data have been developed since that time.

INDIVIDUAL MATERIAL CHARACTERISTICS

Asphalt Cement

Characteriszation tests for the AR-4000W (Husky Oil Co.) asphalt cement used in the actual paving mixtures were reported previously [1]. Subsequently, the asphalt cement was tested for total sulfur content (naturally occurring) by Matrecon, Inc., Oakland, California. The sulfur content of this asphalt cement and two others from Chevron (for comparison) are as follows (percent by weight):

1. AR-4000W, Husky: 4.86%

2. AR-4000W, Chevron: 2.34%

3. AR-2000, Chevron: 4.34%

The analysis was performed for total elemental sulfur content only, thus the form in which the sulfur exists in the asphalt cement is not known.

SEA Binders

The SEA binder (30/70) produced on August 27 and 28, 1979 during test section construction at Pullman were sampled both by WSDOT and UW (all SEA ratios are expressed as weight percents of sulfur and asphalt cement). The UW binder samples were stored in one gallon cans. In January 1982, these binder blends were sampled from the cans for each of those two production days. To achieve this, the 30/70 SEA binder in each can was reheated and thoroughly mixed. This was followed by pouring the heated binder into small tins which were delivered to Matrecon for further analysis. Matrecon performed analyses on samples from both the top and bottom of each tin. As might be expected, some settelment of sulfur occurred.

A summary of the Matrecon results is shown in Table 1 (testing accomplished by gel permeation chromotography and differential scanning calorimetry). The total sulfur content of the 30/70 SEA binders produced on August 27 and 28, 1979, should have averaged about 35 percent

Table 1. Compositional Analysis of "30/70" SEA Binder

	"30/70" SEA Binder (% by wt)			
	Produced: 8/27/79		Produced: 8/28/79	
	Top	Bottom	Top	Bottom
1. Elemental analysis				
(a) Carbon	50.44	53.27	63.64	60.00
(b) Hydrogen	7.17	6.12	7.33	7.07
(c) Sulfur (total)	29.72	40.08	28.61	31.96
2. Free sulfur	25.4	37.4	25.0	28.9
3. Combined sulfur	4.3	2.7	3.6	3.1
4. Differential scanning calorimetry				
(a) Othorhombic sulfur (α)	9.2	24.5	8.1	9.6
(b) Monoclinic sulfur (β)	2.6	2.8	3.2	7.0
(c) Total crystalline sulfur	11.8	27.3	11.3	16.6
(d) Amorphous and dissolved sulfur	13.6	10.1	13.7	12.3
(e) temperature of transitions (°C)				
(i) $\alpha \rightarrow \beta$	106	107	106	106
(ii) $\alpha \rightarrow$ liquid	112	112	112	112
(iii) $\beta \rightarrow$ liquid	117	117	117	116

(30 percent added sulfur plus 5 percent natural sulfur in the Husky AR-4000). The production for August 27 came close to the target value (34.9 percent). The August 28 total sulfur content was lower (30.3 percent). However, it is quite possible that the observed differences are primarily due to sampling variations (sampling at plant, transfer between containers, etc.). At any rate, the expected amount of added sulfur is in the correct range.

The dissolved or amorphous sulfur content (Table 1 - Item 4d) ranges from about 10 to 14 percent which is similar to values reported by others; however, it is important to note that the SEA binder as originally sampled at the hot-mix plant was reheated for sampling a few weeks ahead of the testing reported in Table 1. Thus, the "ultimate" dissolved sulfur content range will probably be somewhat less. The amount of dissolved sulfur is, in part, significant due to its ability to truly extend the asphalt cement (as opposed to crystalline sulfur which may or may not extend the asphalt cement).

The phase temperature transitions (Table 1 - Item 4e) are essentially the same temperatures as those of pure crystalline sulfur. This probably indicates that the asphalt cement is not plasticizing the sulfur.

OPTIMUM MIXTURE DESIGNS

A complete summary of the mix design process was presented previously for both the Hveem and Marshall procedures [1]. The original project goal was to produce SEA mixtures with ratios of 0/100, 30/70 and 50/50 and optimum binder contents (by weight of total mix) of 5.5, 6.5 and 7.4 percent, respectively (based on laboratory mix designs). Due to plant start-up and initial calibration problems and the relatively small amount of hot-mix produced, the final paving mixes consisted of 0/100, 30/70 and 40/60 SEA mixtures. The actual binder contents used were 5.7 percent (by weight of total mix) for the 0/100 mixture, and 6.6 percent for the 30/70 and 40/60 mixtures. By equivalent volume of asphalt cement, the 40/60 SEA binder would have been placed at a content of 7.1 percent (by weight). Thus, the 40/60 SEA sections were placed at approximately 0.5 percent binder content below optimum. This was not planned in the initial experimental design but provided the opportunity to evaluate a SEA mixture below the volume equivalent of the 0/100 SEA mixture. Additionally, a short segment of the 30/70 SEA section on SR 270 was paved at 7.1 percent (by weight) instead of 6.6 percent. This is providing the opportunity to investigate the surface wear differences of the 30/70 SEA mixture at optimum binder content (6.6 percent) and at 0.5 percent higher (7.1 percent).

STRUCTURAL DESIGN

Unlike the detailed analysis performed in order to develop the WSU Test Track structural designs, the selected paving thickness for the

SR 270 test sections (all mixtures) was 1.8 in. (4.6 cm) as determined by WSDOT. Thus, all paving mixtures were placed as an overlay over an existing asphalt concrete surfaced pavement (pre-construction pavement surface shown in Figure 2). The original highway section was experiencing numerous transverse cracks and a few, small areas of alligator cracking prior to paving. None of the pre-construction pavement distress warranted special treatment such as patching, etc.

CONSTRUCTION

TEST PAVEMENT CONFIGURATIONS

The test pavements placed on SR 270 were constructed as an overlay 1.8 in. (4.6 cm) thick (Figure 3). There are a total of six sections so that each SEA mixture would experience traffic in either direction. The primary value of the SR 270 sections is to provide a long-term durability and surface wear evaluation of the three SEA mixtures. The total length of the SR 270 project is 0.8 mi (1.3 kilometre). Paving was accomplished between the outer edges of each shoulder (two main lanes and shoulders were paved). This provided for observing possible differences between traffic associated surface wear in the main lanes and environmental deterioration on the non-trafficked shoulders.

PAVING OF TEST SECTIONS

The paving sequence associated with the construction of the WSU Test Track and SR 270 sections was as shown in Table 2. All paving was accomplished during a total of six working days. The paving mixtures were essentially the same for both paving locations. The total size of the project was small, about 2,300 tons (2090 metric tons). The size of the project is important in that little time (or tonnage) was available to achieve an efficient, smooth running mix production - paving sequence.

The major equipment items used in constructing the test pavements include the following:

1. Mixing plant: Standard (3,000 lb (1360 kg) pugmill capacity) with 30 second wet mix time for all mixes.

2. Pavers: Blaw Knox 8 and 12 ft (2.4 and 3.7 m) widths.

3. Rollers: Gallion 8 and 10 ton steel wheel (static) supplemented with a pneumatic roller as required.

Thickness

Based on the initial pavement cores from each of the test sections, the average thickness for the SR 270 overlay was 1.9 in (4.8 cm) versus the planned 1.8 in (4.7 cm).

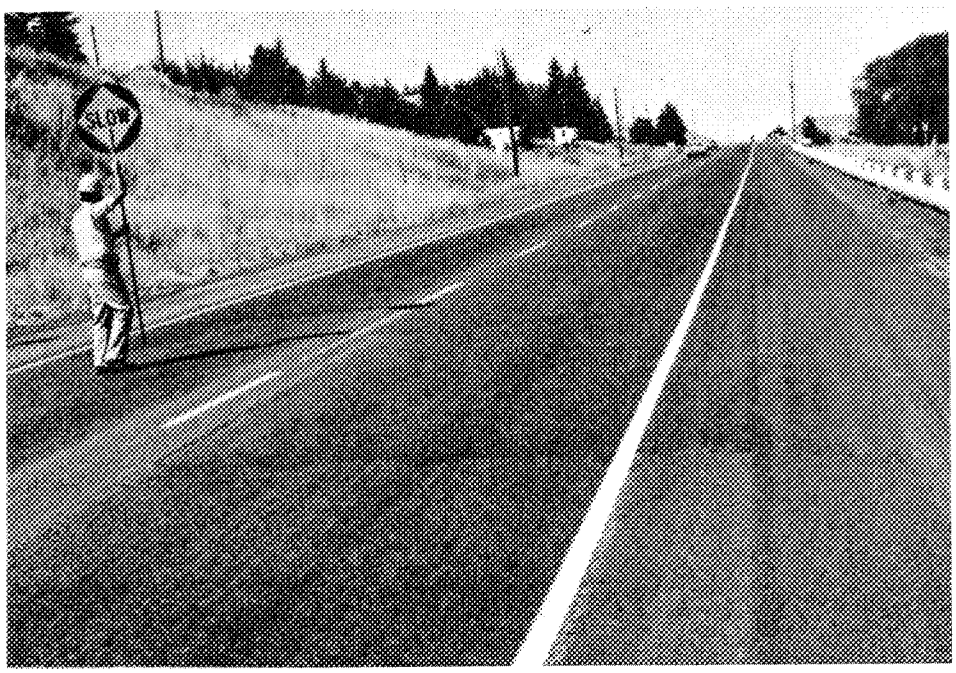


Figure 2. Pre-construction Pavement Surface - SR 270 (August 1979)

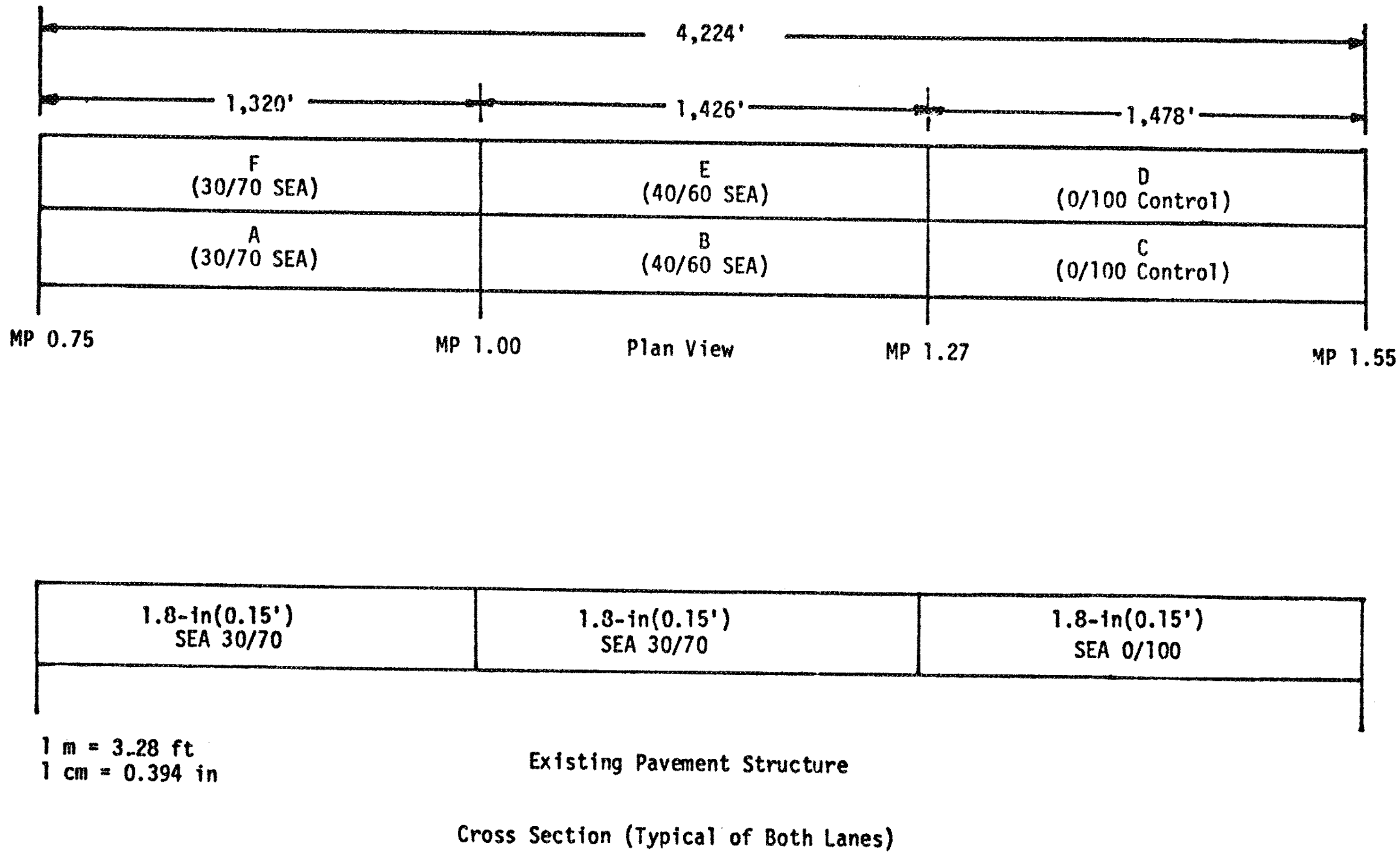


Figure 3. Layout of Experimental Highway Project (SR 270)

Table 2. Paving Sequence

Date	Type of Mix (SEA Ratio)	Mix Produced (tons)	Paving Location
8/20/79	0/100	772	SR 270
8/21/79	0/100	105	SR 270 WSU Test Track (Sections 1,2,8)
8/22/79	0/100	15	WSU Test Track (Sections 7,8)
8/24/79	40/60	596	SR 270 WSU Test Track (Sections 3,4,9,10)
8/27/79	30/70	424	SR 270 WSU Test Track (Sections 5,6,11,12)
8/28/79	30/70	435	SR 270 WSU Test Track (Wearing Course - all sections)
		Total 2,347	

1 ton (metric) = 1.102 ton (short, 2000 lbm)

Paving Temperatures

Numerous paving mixture temperature measurements were obtained throughout the construction sequence. In general, the mixing and compaction temperatures were within acceptable ranges.

Paving Densities

The target density for all mixes was set at 92 percent of maximum theoretical (which was 160 pcf (2.564 Mg)). Thus, during rolling, a lower limit of 147.2 pcf (2.359 Mg) was targeted. All field densities were measured by WSDOT personnel using a Troxler Nuclear gage (No. 4661). Table 3 is used to present a summary of the field densities obtained during construction.

Overall, for all three paving mixes, the measured densities generally meet or exceed the target value. The mean values are generally slightly above the target and the associated standard deviations are relatively low. One comment made by the WSDOT Project Inspector was that the 30/70 and 40/60 SEA mixes compacted more readily than the 0/100. This is due to the lower viscosity of SEA binders because of added molten sulfur.

Mixture Gradations

Mixture gradation tests were performed by WSDOT District 6 personnel at the field laboratory located adjacent to the contractor's mixing plant. The tests were accomplished in accordance with the WSDOT Quick Wash test method. In general, the percent passing all sieves (both mean and range) were within Class B specification limits. The 30/70 and 40/60 SEA mixes as well as the initial production of the 0/100 mix were on the high side of the No. 200 sieve specification limit (seven percent). The allowable range is three to seven percent.

Table 3. Nuclear Gage Densities Obtained During Construction of SR 270 Sections

Date	Type of Mix (SEA Ratio)	Milepost	Travel Direction*	Densities (pcf)				No. of Roller Passes
				Mean	Standard Deviation	Range	No. of Tests	
8/20/79	0/100	1.30-1.53	WB	149.7	2.2	147.1-152.9	10	-
		1.30-1.40	EB	144.2	2.1	142.1-147.4	10	-
		1.45-1.54	EB	147.5	1.5	145.6-150.8	10	-
8/24/79	40/60	1.10-1.25	WB	150.8	1.9	148.3-153.3	10	-
		1.05-1.22	EB	144.8	3.0	138.9-147.1	10	Initial
		1.05-1.22	EB	147.1	2.6	143.5-150.4	10	Final
3/27/79	30/70	0.80-0.96	WB	147.9	1.1	146.5-149.5	10	8
		0.80-0.90	EB	148.9	2.4	144.3-151.7	10	-

1 Mg/m^3 = 62.4 lb/ft^3

*WB = Westbound
EB = Eastbound

CHAPTER II

DATA COLLECTION AND ANALYSIS

Immediately following construction of the SR 270 test pavements, the highway was reopened to traffic. Subsequently, several types of data were collected in order to properly evaluate the SEA mixtures studied.

The following post-construction data was collected and summarized:

1. Deflection measurements
2. Pavement cores
   (a) Density
   (b) Air voids
   (c) Resilient modulus
   (d) Stability and flow
3. Surface friction testing
4. Traffic counts and truck weight survey
5. Surface cracking and wear

Each of the above items will be summarized in the information which follows.

DEFLECTION MEASUREMENTS

Pavement surface deflection measurements were made periodically with a Benkelman Beam and a truck with a rear single axle loaded to 18,000 lbs (8165 kg). The resulting deflection measurements are summarized in Table 4.

In general, the adjusted deflections (adjusted to a standard temperature of 70°F (21°C)) are low to modest indicating a structural section of modest overall stiffness. Further, the deflections obtained during August 1979 (prior to overlayment - refer to Figure 4) are slightly higher than measurements obtained during August 1980 - one year after placement of the overlay (as one would expect). The highest deflections were recorded during May 1980 and May 1982. These higher values are probably due to seasonal factors such as increased moisture in the base course and subgrade following the spring thaw. Further, detailed analysis of the deflection data on a section-by-section basis did not reveal any significant differences between SEA mixtures (deflection measurements of August 1979 and May 1980).

Table 4. Summary of Benkelman Beam Deflections

Date	Travel Direction	Unadjusted Deflection (in)			Adjusted Deflection** (in)	Temperature (°F)	
		Mean	Standard Deviation	No. of Points		Pavement	Air
8/17/79	WB	0.025	0.009	31	0.019	102	85
	EB	0.024	0.008	22	0.015	117	86
5/27/80	WB	0.020*	0.007	18	0.022	61	54
	EB	0.017*	0.007	19	0.019	60	54
8/13/81	WB	0.020	0.009	3	0.013	109	93
	EB	0.020	0.007	3	0.013	109	93
5/6/82	WB	0.017	0.002	3	0.019	58	61
	EB	0.020	0.005	3	0.022	58	61

1 cm = 0.394 in
°C = (F-32)(5/9)

*Adjusted to standard of 18,000 lbs (8165 kg) on single axle-dual tires.
**Mean deflection adjusted to standard temperature of 70°F (21°C) by use of Canadian Good Roads Association Benkelman Beam Rebound Procedure.

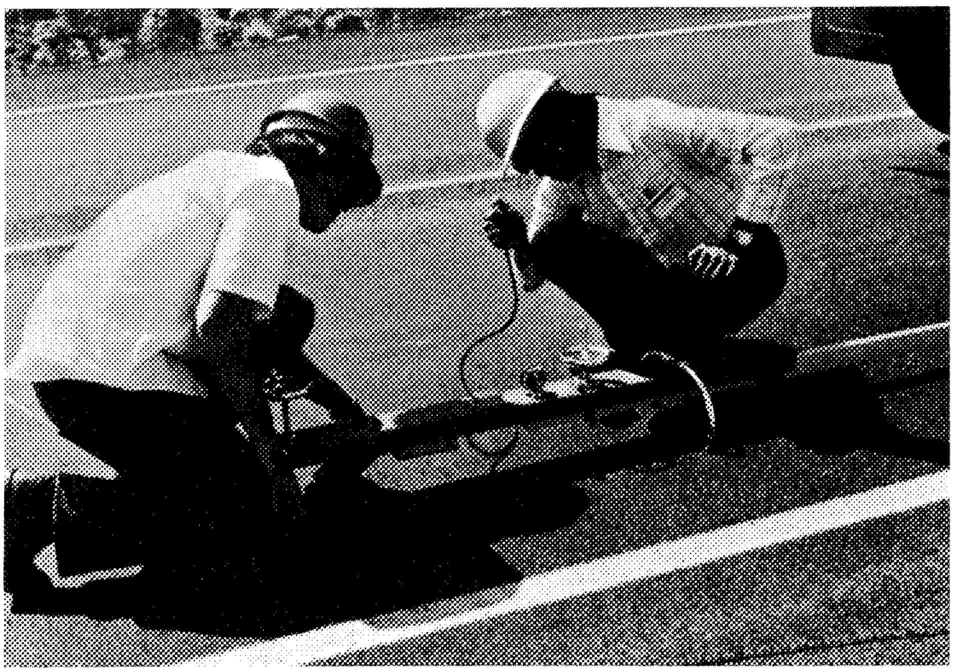


Figure 4. Benkelman Beam Survey at SR 270 Test Site (August 1979)

PAVEMENT CORES

Core samples were obtained by WSDOT personnel for use in determining the post-construction material properties. These properties include resilient modulus, Marshall stability and flow, density and air void contents. The initial set of cores (sampled December, 1979) were obtained by use of a core barrel measuring 3.75 in (9.5 cm) in diameter. Thereafter, all cores were 4.0 in (10.2 cm) in diameter. Three full-depth cores were obtained from each of the highway test sections. Of the total cores obtained approximately one-half were sent to the UW and the remainder retained by the WSDOT Materials Laboratory.

The UW laboratory investigation included a determination of resilient modulus (at 41°F (5°C), 77°F (25°C) and 104°F (40°C)), density and air void determinations. Marshall stability and flow were obtained on those cores which had suitable diameters and thicknesses.

Cores were sampled during December 1979, December 1980, June 1981, October 1981 and July 1982 or about 4, 16, 21, 25 and 34 months after construction, respectively. All cores were prepared for testing by saw cutting the overlay from the underlying asphalt concrete pavement.

TEST PROCEDURES

A pneumatic repeated loading device, two linear variable displacement transducers (LVDT) and strip recorder were used to determine the resilient modulus of each of the test samples, using a repeated load of about 100 lb (445 N). This was done first at 77°F (25°C). The samples were then placed in a refrigerated environment, cooled to 41°F (5°C) for a minimum of 24 hours, and then tested again. Following this series of tests, the samples were placed in a 104°F (40°C) enclosure for 24 hours and then once again tested. The resulting deformation values together with the measured thickness of each sample were used to determine the resilient modulus.

Following the determination of all resilient modulus values, the same samples were used in obtaining Marshall stability and flow and bulk and theoretical maximum specific gravities in accordance with ASTM standards.

RESILIENT MODULUS

Figures 5 through 7 and Table 5 provide summaries of resilient modulus data for all sampled cores. Figure 5 shows for 0/100 SEA mixtures how such data varied over approximately a three year period for three temperature levels. Similarily, Figures 6 and 7 provide summaries for the 30/70 and 40/60 SEA mixtures, respectively. In general, for all mixtures, the initial resilient moduli increased followed by a decreasing trend. This is further verified in Table 5.

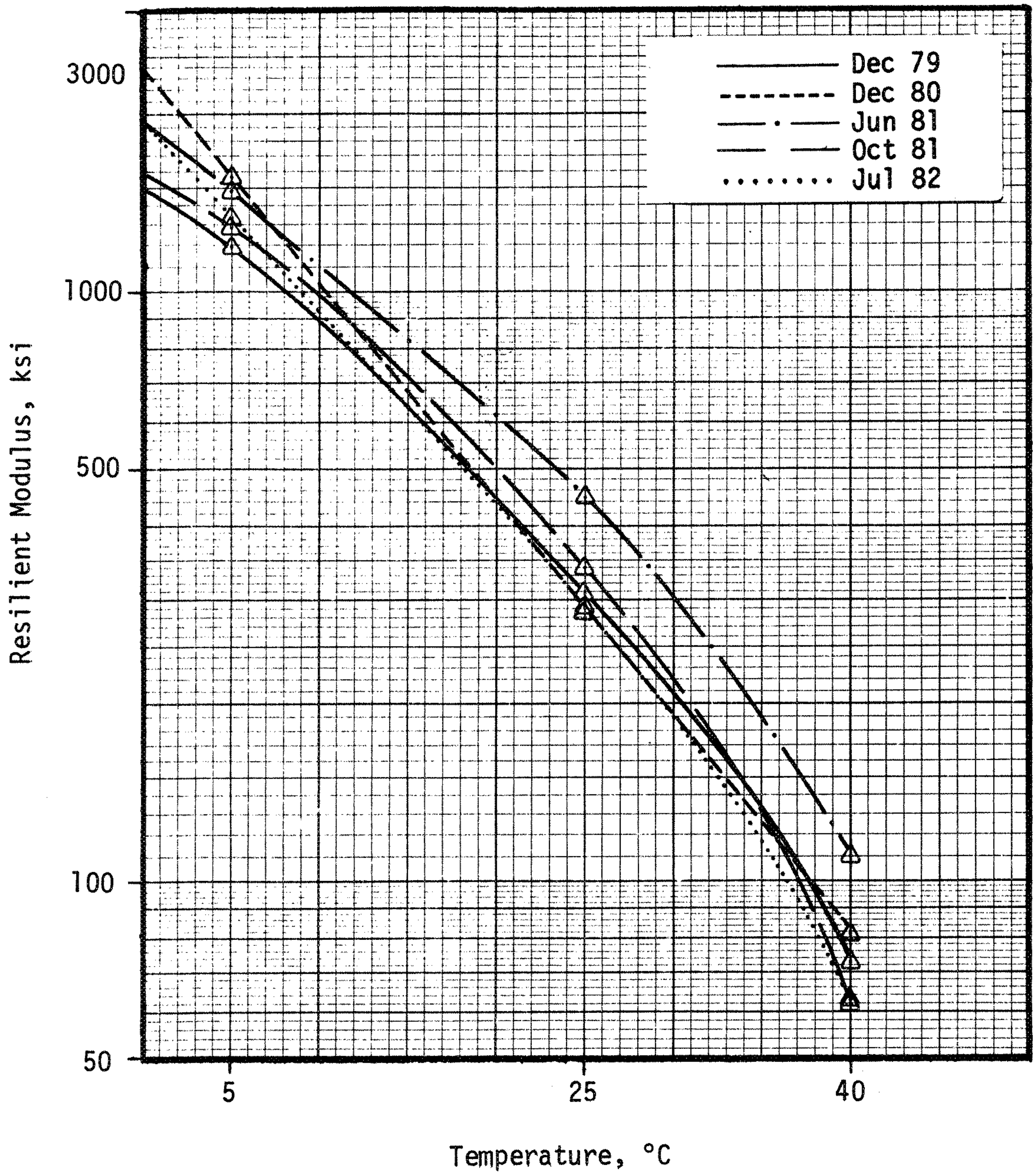


Figure 5. Resilient Modulus vs. Temperature, SR 270 Overlay, 0/100 Cores

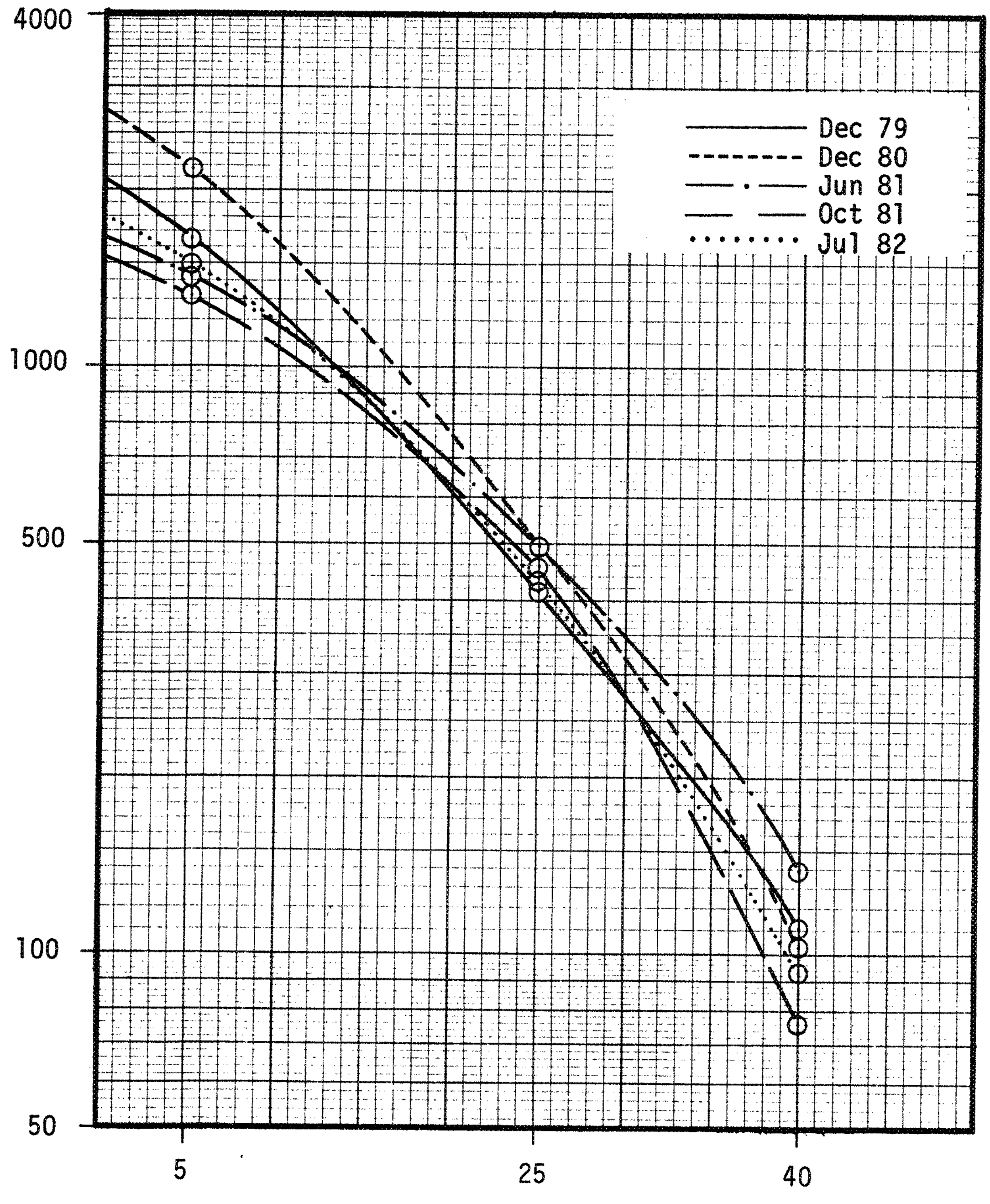


Figure 6. Resilient Modulus vs. Temperature, SR 270 Overlay, 30/70 Cores

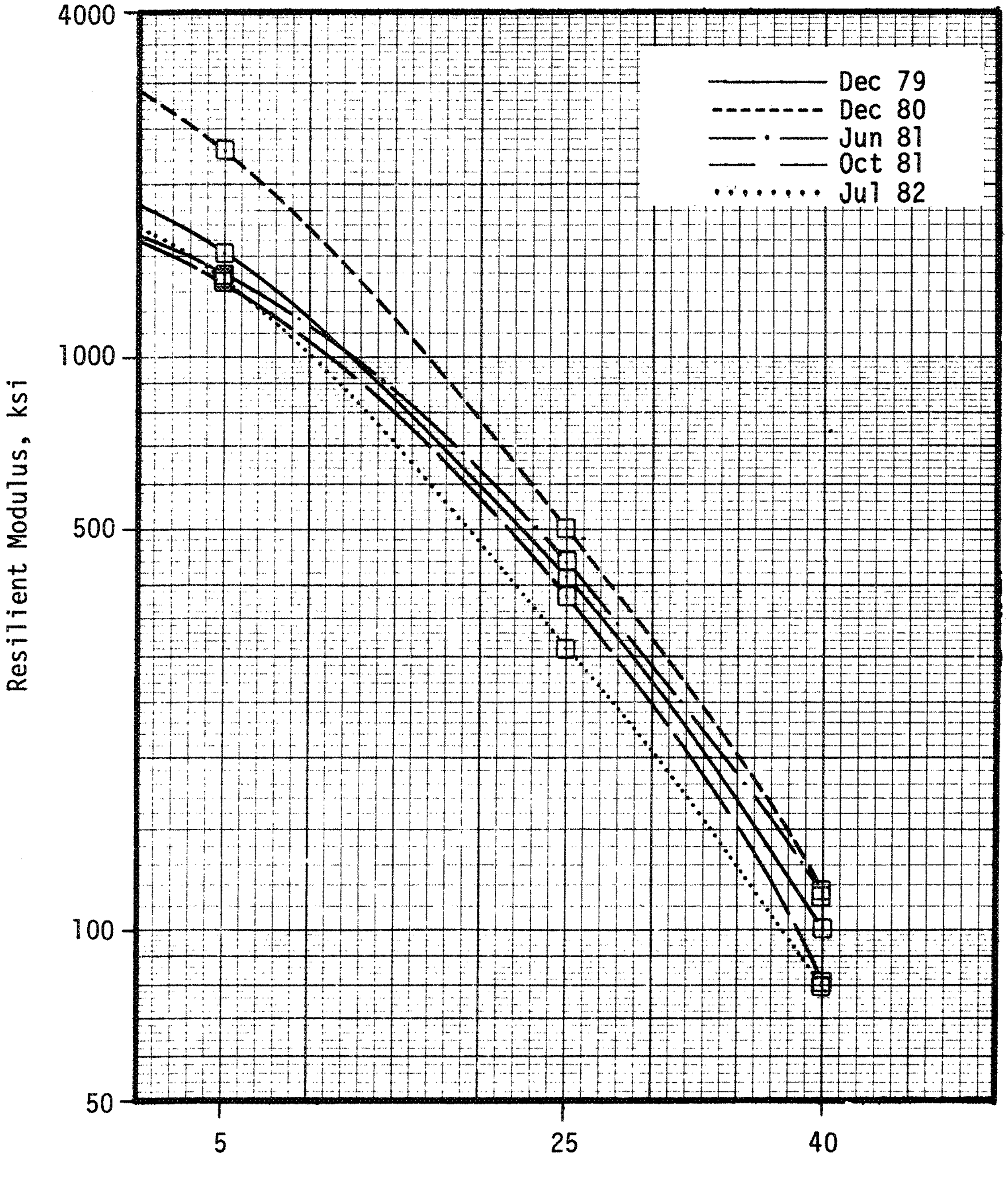


Figure 7. Resilient Modulus vs. Temperature, SR 270 Overlay, 40/60 Cores

Table 5. Summary of Resilient Moduli

Type of Mix (SEA Ratio)	Travel Direction	Resilient Modulus ($X10^3$ psi)																	
		41°F (5°C)						77°F (25°C)						104°F (40°C)					
		4mo*	16mo*	21mo*	25mo*	34mo*	%Δ**	4mo*	16mo*	21mo*	25mo*	34mo*	%Δ**	4mo*	16mo*	21mo*	25mo*	34mo*	%Δ**
0/100	WB	1170	1762	1456	1342	1303	+11	312	314	461	355	281	-10	82	77	122	68	59	-28
	EB	1214	1377	1507	1232	1343	+11	303	274	432	324	302	0	65	85	99	55	68	+5
30/70	WB	1872	2328	1160	1529	1477	-21	468	438	384	435	366	-22	110	97	107	62	63	-43
	EB	1440	2027	1693	1101	1521	+6	360	552	603	479	488	+36	112	109	171	89	123	+10
40/60	WB	1618	2547	1476	1317	1376	-15	405	444	486	387	283	-30	97	93	127	76	57	-41
	EB	1409	2042	1309	1379	1346	-4	423	559	392	377	337	-20	106	140	102	86	103	-3

1 kPa = 0.1451 psi
*Time between construction and sampling
**Percent change between 4 month cores and 34 month cores

Due to the small sample sizes, clear differences in resilient modulus losses for the three SEA ratios evaluated are difficult to determine. However, the differences in resilient moduli for the time period evaluated are clearly less for the eastbound lane (as opposed to the westbound lane). Further, the largest and most consistent resilient modulus loss trend occurs for the 40/60 SEA mixture. Figure 8 was plotted for one temperature level (77°F (25°C)) to illustrate this trend. Two possible causes may be:

1. The 40/60 SEA paving mixtures were placed with a binder content about 0.5 percent below the laboratory determined optimum.

2. SEA mixtures generally exhibit an increased susceptibility to moisture damage (hence loss of stiffness).

MARSHALL STABILITY AND FLOW

For all pavement cores, unit weight and air voids were determined and on those samples which were of suitable diameter and thickness Marshall stability and flow were obtained. Tables 6 and 7 are summaries of this data.

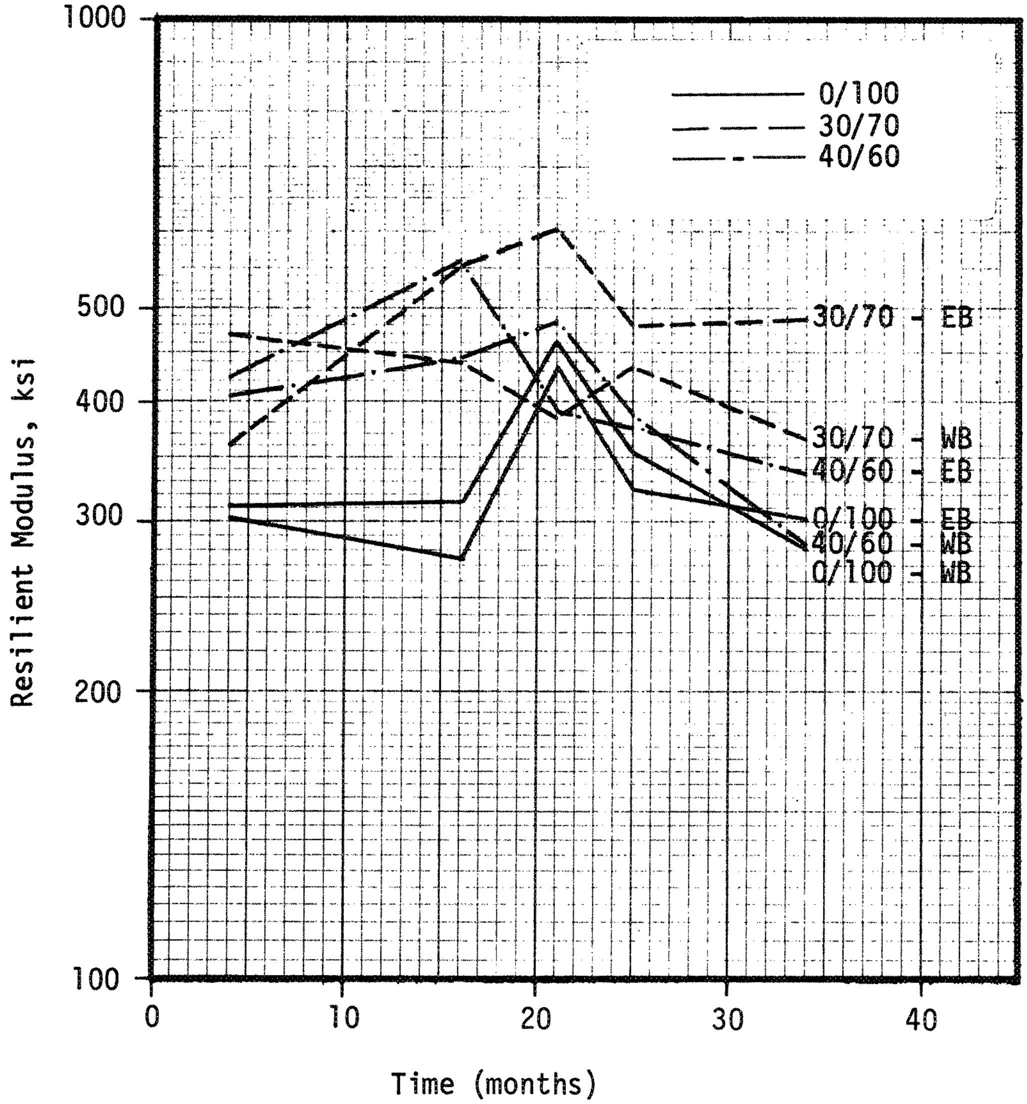
The Marshall stability and flow data shown in Table 6 is at least in part explained by the corresponding air voids, i.e., the lower the air voids the larger the stability. Overall, no significant difference is noted for the individual SEA mixtures over the time period evaluated. A modest increase in stability is noted for the mixtures containing added sulfur. The flow values for most of the tested cores are within the range normally recommended by the Asphalt Institute [2].

DENSITY AND AIR VOIDS

Table 7 is used to summarize the density and air void levels for the five sets of pavement cores obtained during the three year evaluation period. In general, no consistent trend exists which would indicate an increase or decrease in density with a corresponding change in air voids. The variation in air voids is assumed to be due to the normal mixture variations experienced in sampling paving mixtures. However, most of the reported densities meet or exceed the original construction specification requirement (92 percent of maximum theoretical density (0.92 x 160 pcf = 147.2 pcf)).

SURFACE FRICTION TESTING

The friction tests as obtained by WSDOT for the SR 270 sections are summarized in Table 8. The skid tester used to obtain this data is the locked-wheel type (manufactured by K.J. Law, Inc.). The initial tests (September 1979) were obtained shortly after construction completion. Approximately three years after construction, the Friction Numbers for all sections are adequate and have generally increased since the initial construction condition.



1 kPa = 0.1451 psi

Figure 8. Resilient Modulus Changes as a Function of Time (77°F (25°C))

Table 6. Marshall Stability and Flow for Pavement Cores

Type of Mix (SEA Ratio)	Travel Direction	Marshall Tests							
		12/80*		6/81*		10/81*		7/82*	
		Stability (lb)	Flow	Stability (lb)	Flow	Stability (lb)	Flow	Stability (lb)	Flow
0/100	WB	1670	13	-	-	-	-	1650	15
	EB	1700	12	2710	19	1650	15	1730	**
30/70	WB	2060	16	-	-	-	-	1890	9
	EB	1700	11	2500	12	-	-	3200	9
40/60	WB	2450	17	2880	18	2620	17	2630	14
	EB	1790	8	-	-	-	-	1800	12

1N = 0.2248 lbf

*Date cores sampled
**Thin core sample

Table 7. Unit Weight and Air Voids for Pavement Cores

Type of Mix (SEA Ratio)	Travel Direction	12/79*		12/80*		6/81*		10/81*		7/82*	
		Unit Weight (pcf)	Air Voids (%)	Unit Weight (pcf)	Air Voids (%)	Unit Weight (pcf)	Air Voids (%)	Unit Weight (pcf)	Air Voids (%)	Unit Weight (pcf)	Air Voids (%)
0/100	WB	145.9	10.2	149.1	8.1	156.2	4.0	151.4	7.1	147.0	10.1
	EB	149.6	6.4	147.3	9.2	157.2	4.3	152.8	6.4	151.7	7.5
30/70	WB	150.3	8.0	154.1	5.7	146.9	11.1	152.1	6.8	154.7	5.4
	EB	148.4	9.8	147.9	10.6	152.3	8.5	150.0	10.3	152.4	8.3
40/60	WB	153.4	7.5	157.2	4.9	152.0	5.9	156.6	4.6	158.2	3.3
	EB	148.4	12.5	147.9	10.6	151.8	7.2	146.9	10.9	149.3	9.6

1 $Mg/m^3 = 62.4\ lb/ft^3$

*base cores sampled

Table 8. Surface Friction Tests for SR 270 Sections

Type of Mix (SEA Ratio)	Travel Direction*	Mean Friction Numbers		
		Date: 9/79	Date: 4/80	Date: 7/82
0/100	WB	39	45	37
	EB	32	48	44
30/70	WB	41	49	47
	EB	42	48	49
40/60	WB	38	49	44
	EB	38	48	49

*WB Westbound
EB Eastbound

TRAFFIC SUMMARY

A permanent mechanical traffic count station was installed at milepost 1.32 on SR 270 following completion of the paving. A summary of some of the resulting data is shown in Table 9. Based on this information the average daily traffic (both directions) for this highway is about 3,400. Since construction of the test pavements, a total of approximately 3,400,000 vehicles have trafficked the test pavements (or 1,700,000 vehicles in each direction) as of July 1982.

A survey was conducted during October 1979 by WSDOT personnel to classify and weigh the trucks traveling through the test site. Truck weights were obtained by the use of portable scales as shown in Figure 9 (both sides of each axle were weighed). The sample size (trucks weighed) and origin and designations are shown in Tables 10 and 11.

The truck survey is further summarized in Tables 12 and 13. These tables show the truck types surveyed, the sample size for each type and associated average gross weight (Table 12) and average axle loads (Table 13). In general, the majority of the trucks were within legal load limits. The heaviest steering axle was on a Type 3-2 truck which weighed 15,000 lbs (6800 kg). The trucks which exceeded allowable axle load limits (allowable limits are 20,000 lbs (9070 kg) for single axles and 34,000 lbs (15,420 kg) for tandems) were principally Type 3S2 vehicles. The two sets of tandem axles on these overweight vehicles were both over allowable limits with the measured weights ranging from a low of 37,400 lbs (16,960 kg) to a high of 45,700 lbs (20,730 kg). The cargo hauled by these overweight vehicles were either petroleum or agricultural products.

SURFACE CRACKING AND WEAR

Two basic measures can be used to evaluate the in-situ durability of the paving materials used in the study - cracking and surface wear. To evaluate the cracking of the test sections, cracking patterns were determined prior to overlayment and three years later. Surface wear was visually estimated.

CRACKING

Figures 10 and 11 are sketches of the visual surface cracking in the six test sections. Figure 10 shows the cracking patterns prior to overlayment. The eastbound (downhill) lane had experienced substantially more transverse cracking than the westbound (uphill) lane. The cracking patterns show that transverse cracking (presumably thermal cracks) is the dominant type of cracking followed by substantially lesser amounts of longitudinal cracking and minor amounts of alligator cracking. The pre-construction transverse cracks were generally less than 1/8 in. (0.3 cm) in width. However, a few cracks in Section B and several in Section C were larger than 1/4 in. (0.6 cm) in width. Figure 11 shows the location and number of transverse cracks in each of the test sections as of July 1982 (three years after placement of the 1.8 in. overlay). The sections with the 40/60 and 0/100 SEA paving mixtures (Sections B and C, specifically) show the largest

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Table 9. Summary of SR 270 Traffic Count Data

Month	ADT			Accumulated Traffic	
	Both Dir.	EB	WB	EB	WB
1979 Oct	3640	1810	1830	56,100	56,700
Nov	3700	1830	1870	111,000	112,800
Dec	2880	1360	1520	153,200	160,000
1980 Jan	2540	1330	1210	194,400	197,500
Feb	3260	1670	1590	241,200	242,000
Mar	3280	1630	1650	291,700	293,100
Apr	3440	1730	1710	343,600	344,400
May	3030	1500	1530	390,100	391,900
Jun	3350	1630	1720	439,000	443,500
Jul	3080	1540	-	486,700	491,200
Aug	3130	1560	1570	535,100	539,900
Sep	4370	2310	2060	604,400	601,700
Oct	-	-	-	660,500*	658,400*
Nov	3100	1550	-	707,000	704,900
Dec	2810	1320	1490	747,900	751,100
1981 Jan	3140	-	1570	796,600	799,800
Feb	3100	-	1550	840,000	843,200
Mar	3360	-	1680	892,100	895,200
Apr	3550	1780	1770	945,500	948,300
May	3880	1920	1960	1,005,000	1,009,100
Jun	3920	1890	2030	1,061,700	1,070,000
Jul	3420	1710	1710	1,114,700	1,123,000
Aug	3160	1580	1580	1,163,700	1,172,000
Sep	4110	2160	1950	1,228,500	1,230,500
Oct	-	-	-	1,284,600*	1,287,200*
Nov	3770	1890	1880	1,341,300	1,343,600
Dec	2740	1290	1450	1,381,300	1,388,600
1982 Jan	2400	1220	1180	1,419,100	1,425,200
Feb	3260	1690	1570	1,466,400	1,469,100
Mar	3390	1700	1690	1,519,100	1,521,500
Apr	3650	1860	1790	1,574,900	1,575,200
May	3710	1830	1880	1,631,600	1,633,500
Jun	3860	1860	2000	1,687,400	1,693,500
Average	3360				

*Used ADT for Oct 1979

(a) Overview of Truck Weighing

(b) Closeup of Portable Scales

Figure 9. Weighing of Trucks with Portable Scales

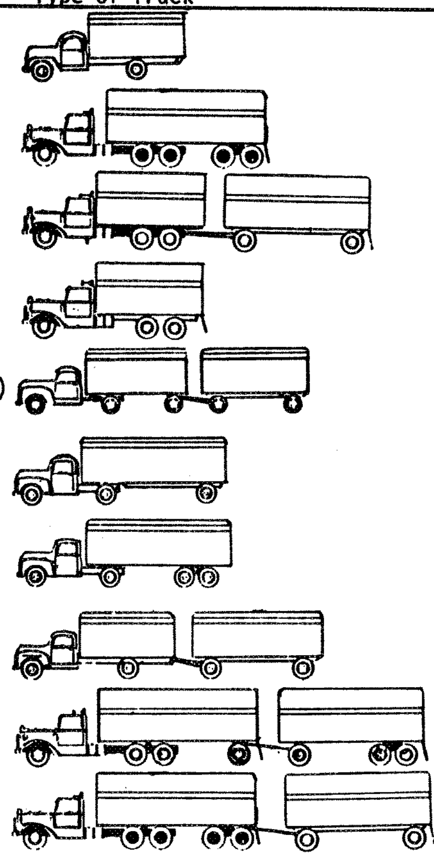
Table 10. Truck Survey Sample Size

Travel Direction	No. of Trucks	Percentage
WB	59	48
EB	65	52
Total	124	100%

Table 11. Origin and Destination of Truck Trips

Origin	No. of Trucks	%	Destination	No. of Trucks	%
Pullman	38	31	Pullman	31	25
Spokane	35	28	Spokane	21	17
Moscow	14	11	Moscow	22	18
Colfax	9	7	Colfax	10	8
Others	28	23	Others	40	32
Total	124	100	Total	124	100

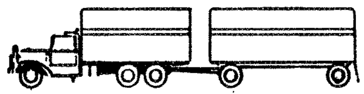
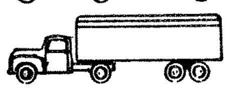
Truck 12. Average Gross Weight by Type of Truck



Type of Truck	Number of Trucks	Percentage	Average Gross Weight (1000 lbs)
2D (Class 1)	52	42	14.4
3S2 (Class 8)	31	25	57.0
3-2 (Class 9)	16	13	56.6
3A (Class 6)	9	7	33.8
2S1-2 (Class II)	6	5	49.0
2S1 (Class 2)	3	2	33.3
2S2 (Class 3)	2	2	33.1
2-2 (Class 4)	2	2	29.4
3S1-3	1	1	117.6
3S2-2	1	1	97.4
Totals	123	100	Weighted Average = 36.2

1 kg = 2.205 lb m

Table 13. Average Axle Load by Type of Truck

Type of Truck		Number of Trucks	Percentage	Average Axle Load (1000 lbs)	
				Single	Tandem
2D (Class 1)		52	42	7.2	-
3S2 (Class 8)		31	25	10.1	11.9
3-2 (Class 9)		16	13	10.8	24.3
3A (Class 6)		9	7	9.1	24.9
2S1-2 (Class II)		6	5	9.8	-
2S1 (Class 2)		3	2	11.1	-
2S2 (Class 3)		2	2	9.0	15.1
2-2 (Class 4)		2	2	7.4	-
3S1-3		1	1	16.8	33.6
3S2-2		1	1	12.3	30.2
	Totals	123	100	Weighted Average = 8.9	Weighted Average = 17.9

1 kg = 2.205 lb m

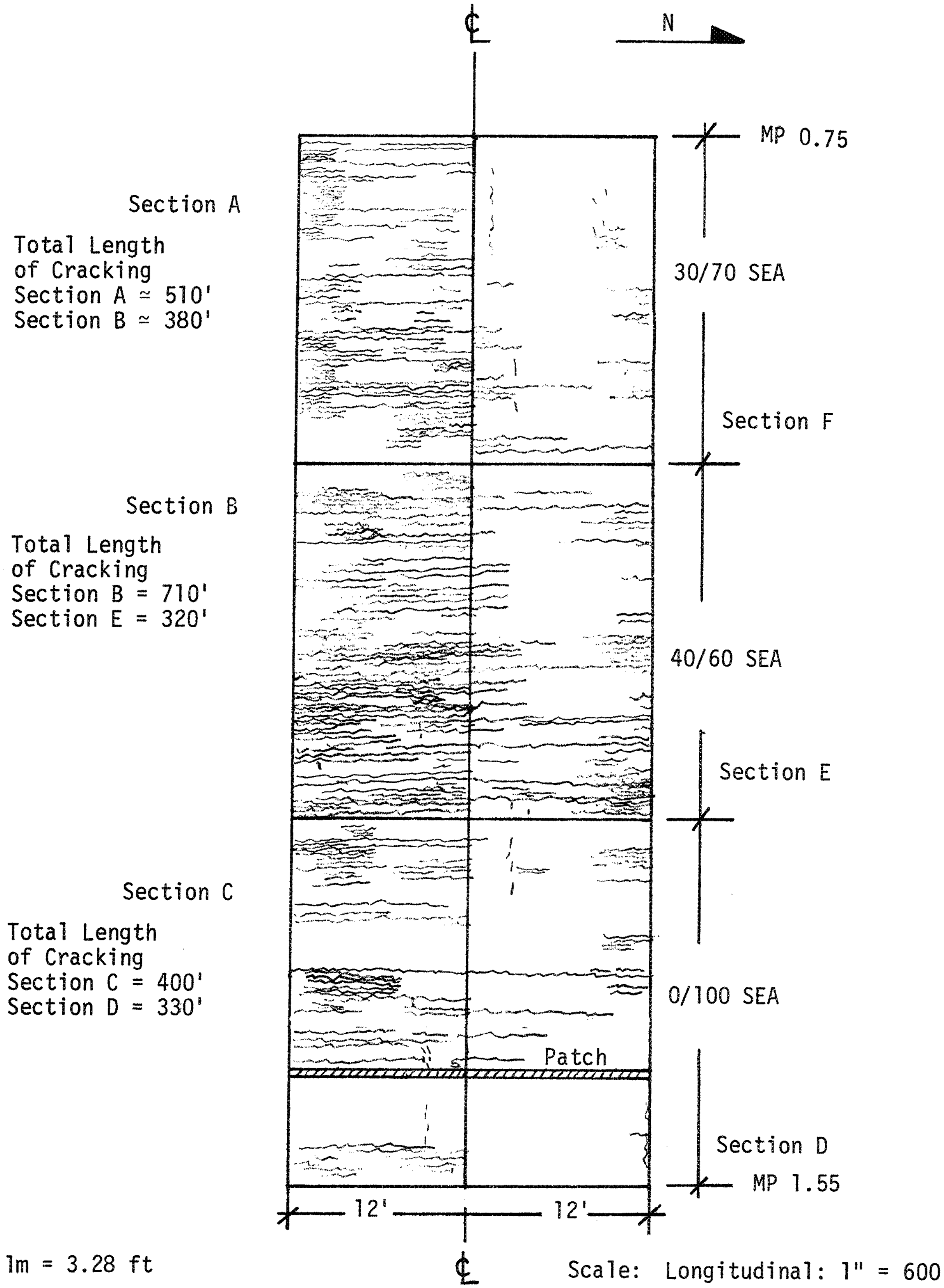


Figure 10. Sketch of Cracking Patterns on SR 270 Test Sections Prior to Overlay (Surveyed August 1979).

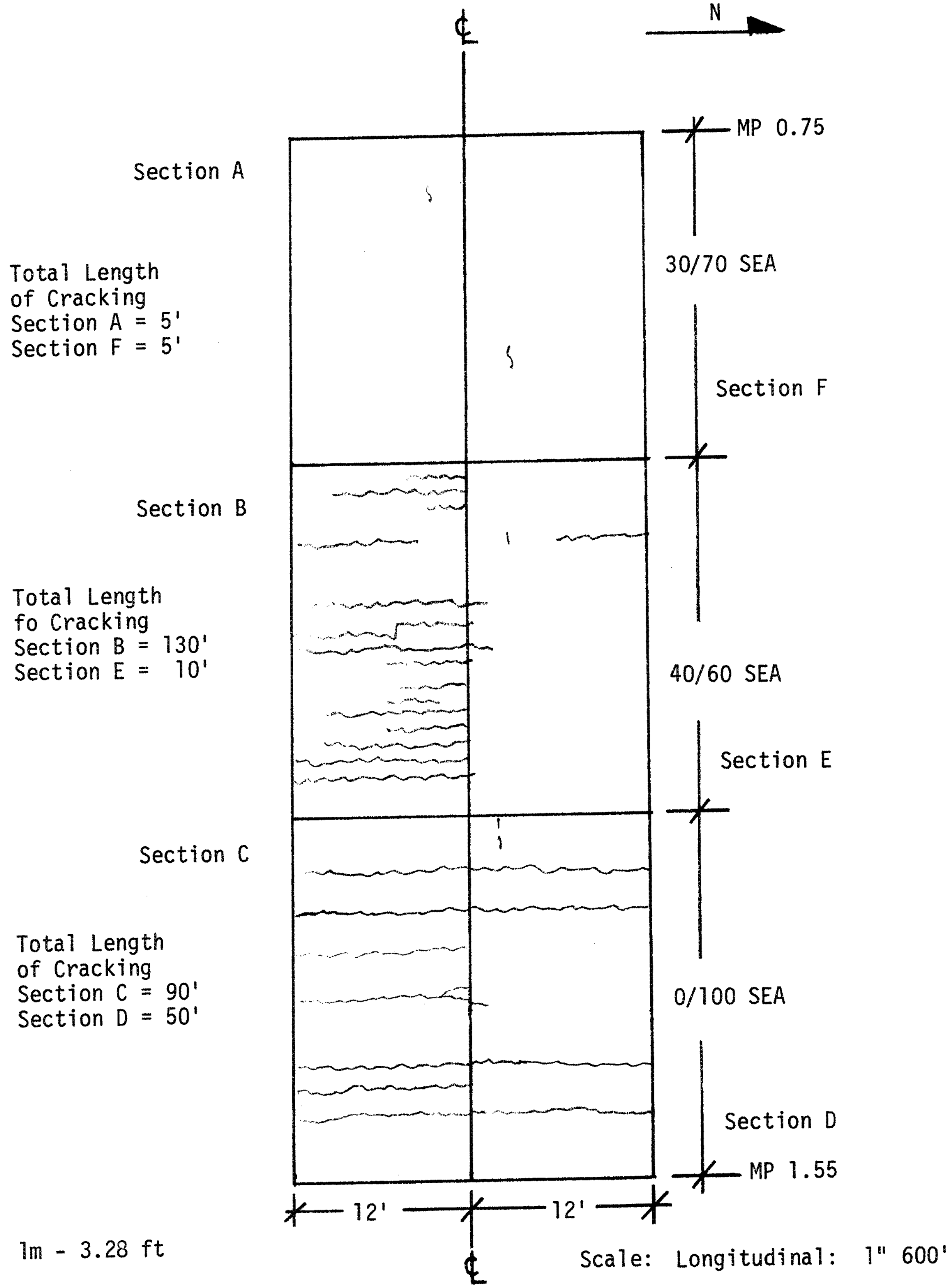


Figure 11. Sketch of Cracking Patterns on SR 270 Test Sections as of July 1982

amount of cracking - most of which is apparently reflective. However, Sections B and C had some of the widest pre-construction cracks (see Figure 12); hence, it is expected that reflection cracking would occur more quickly in these sections. Further, Section C (0/100) also contained narrow patching (for utilities) prior to placement of the overlay. This probably contributed to accelerated reflective cracking as well.

WEAR

Following the winter 1980 (about 6 months after construction), it was apparent that the 30/70 and 40/60 SEA sections were experiencing some surface wear in the wheel paths whereas the 0/100 conventional mix did not. However, the differences in wear were quite small. Surface wear is defined in this case as a loss of fine aggregate from the surface aggregate matrix. Periodic visual condition surveys over a three year span reveal that this trend is still valid; however, the surface wear differences between all six test sections is small. The sections with the most to least surface wear (as of July 1982) can be ranked as follows: 40/60, 30/70, and 0/100. Photographs of the surface conditions for the three mixtures are shown in Figures 13 through 15.

The level of traffic on SR 270 is relatively low - about 3400 vehicles per day (both directions). However, during the winter months a large number of vehicles use studded or otherwise abrasive traction tires. Such tires contribute to the pavement surface wear.

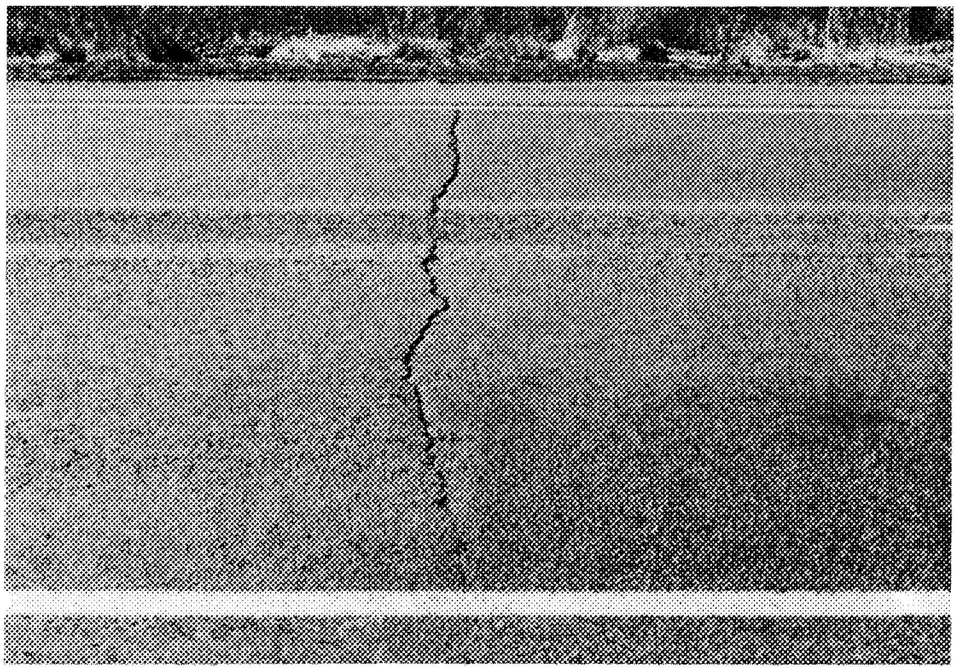
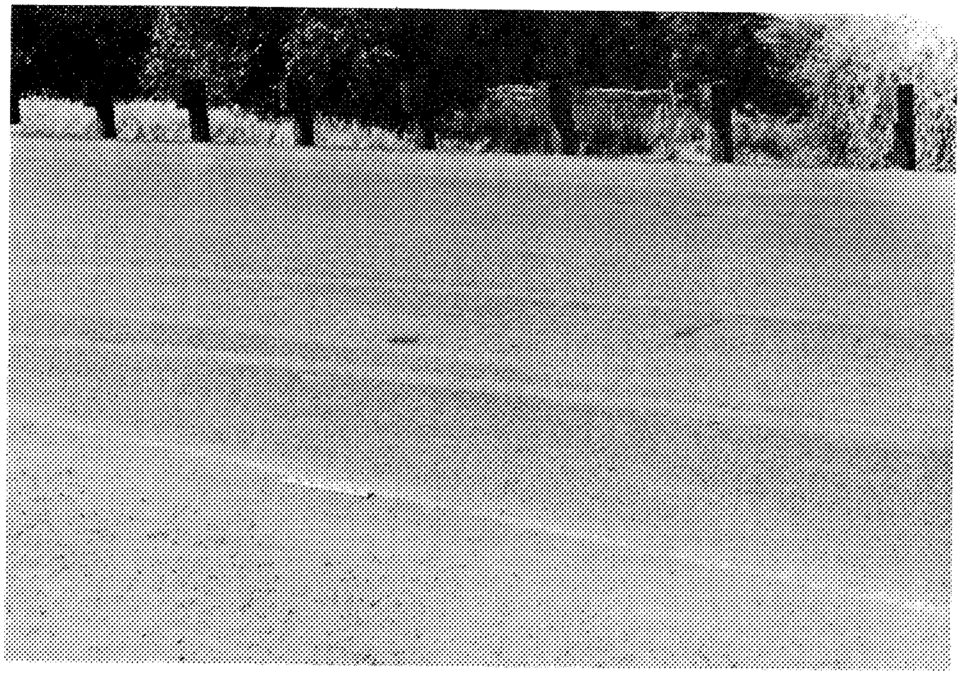
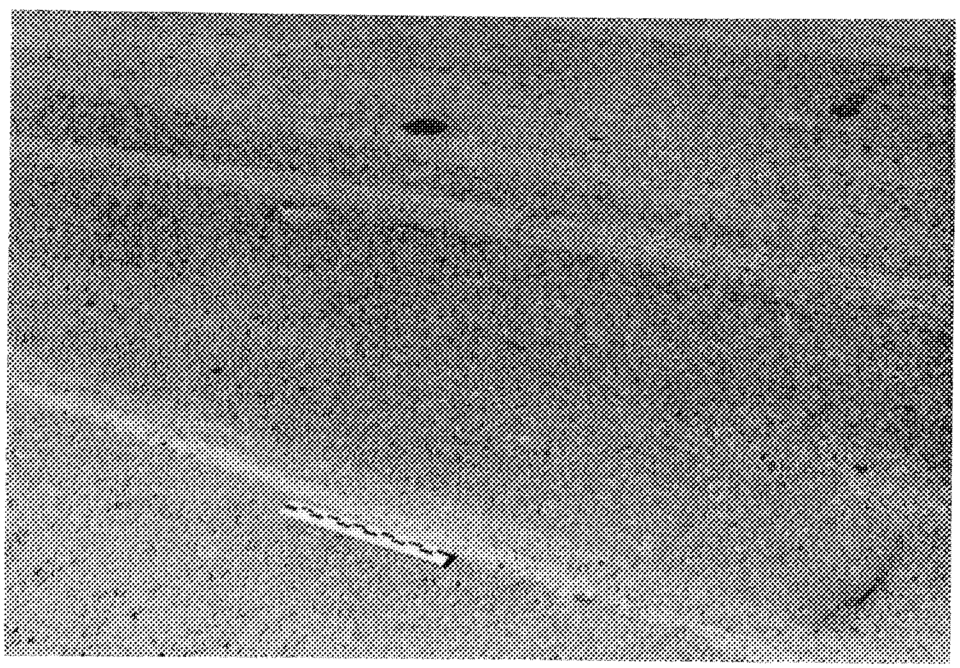


Figure 12. Pre-construction Transverse Crack in Sections B and E

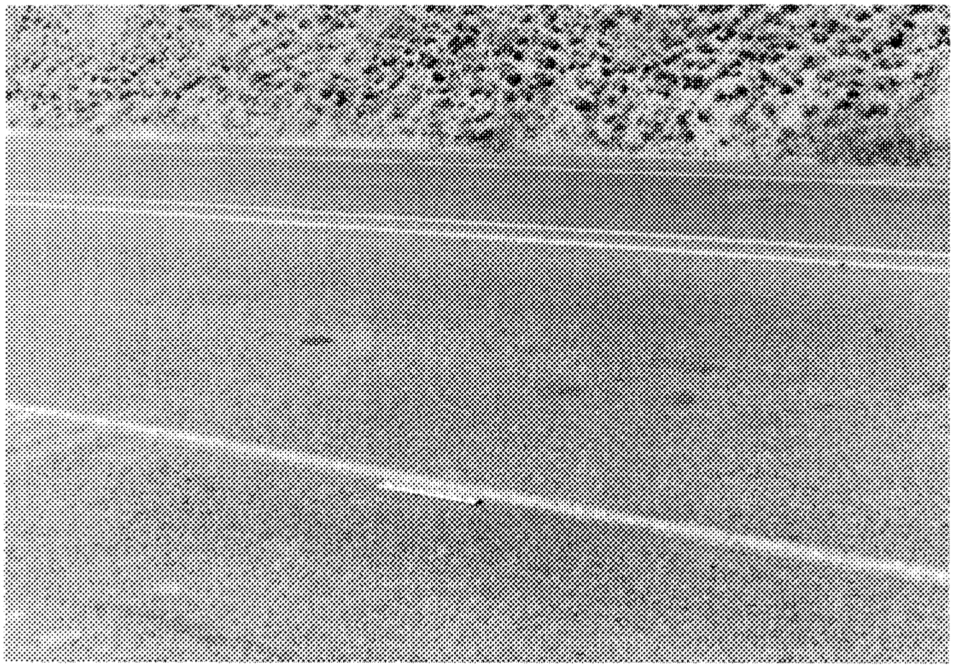


(a) Overview



(b) Close-up

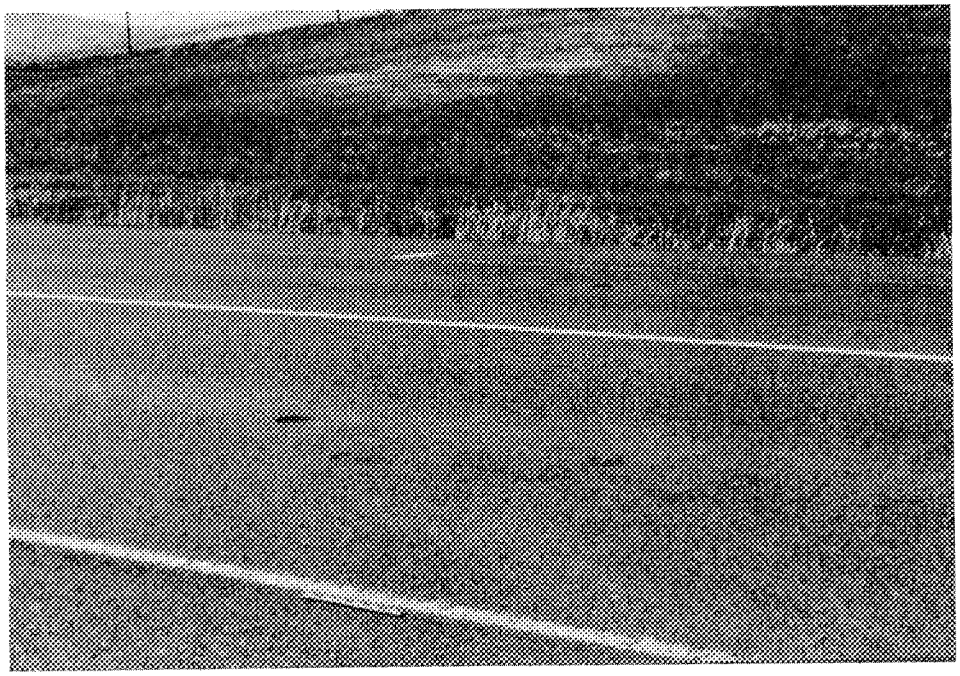
Figure 13. Surface Condition of 0/100 SEA Mixture (July 1982)



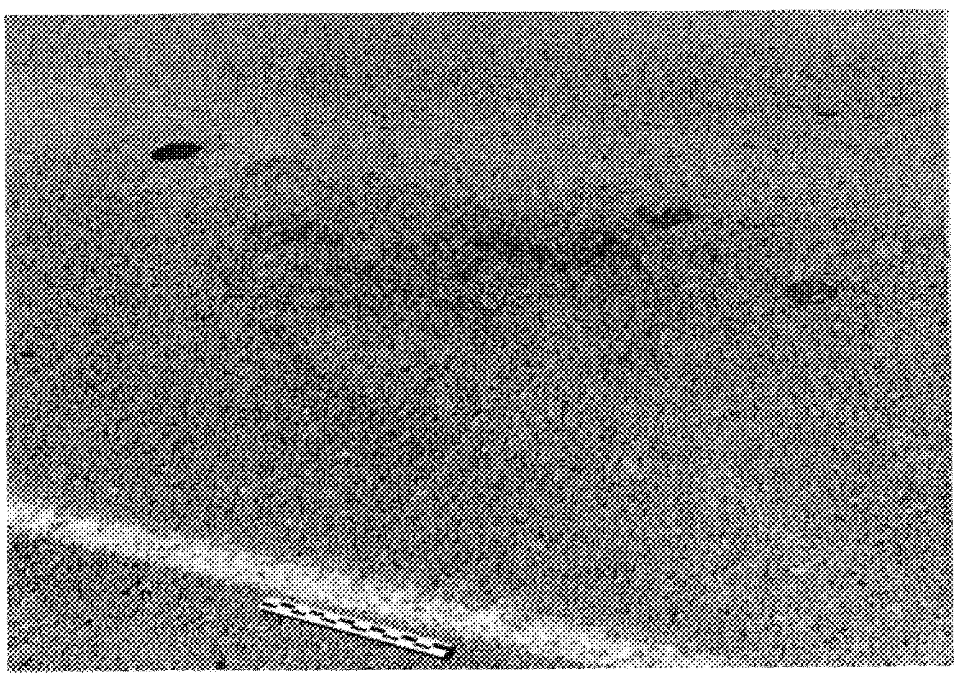
(a) Overview

(b) Close-up

Figure 14. Surface Condition of 30/70 SEA Mixture (July 1982)



(a) Overview



(b) Close-up

Figure 15. Surface Condition of 40/60 SEA Mixture (July 1982)

CHAPTER III

SUMMARY AND CONCLUSIONS

The following summary and conclusions are warranted:

1. SEA paving mixtures were prepared and placed at two test sites near Pullman, Washington during August 1979. One test site (the performance of which is described in this report) is located on an existing highway (SR 270). At this site, the paving mixes were placed as a 1.8 in (4.6 cm) thick overlay. The second test site was the WSU Test Track (the results of which were contained in a previous study report - Reference 3) sections of varying thickness were placed for subsequent controlled wheel tracking. The test mixtures used at both sites were 0/100 (conventional WSDOT Class B asphalt concrete), 30/70 and 40/60 SEA mixtures.

2. The construction of the six test sections on SR 270 (two sections for each SEA ratio) was accomplished with conventional paving equipment. The total tonnage of hot-mix placed was small (total of about 2,300 tons for both test sites).

3. An analysis of post-construction data revealed the following:

    (a) Benkelman Beam deflections. No significant differences exist between test sections. The measured deflections decreased slightly after placement of the overlay (as one would expect). The primary deflection differences appear to be due to seasonal moisture changes in the pavement structure and subgrade.

    (b) Pavement cores

        (i) Resilient Moduli. The resilient moduli measured for the 0/100 mixtures (no added sulfur) were generally lower than the 30/70 and 40/60 SEA mixtures over the three year evaluation period. All three mixtures generally exhibited an increasing resilient modulus trend followed by a decreasing trend. However, the timing of the trends did not necessarily occur simultaneously. The largest and most consistent resilient modulus loss trend occurred for the 40/60 SEA mixture. Possible causes for this may be the less than optimum binder content in the 40/60 SEA mixture and that SEA mixtures can have an increased susceptibility to moisture damage. Further, there is evidence that stiffness differences exist between the eastbound and westbound lanes of SR 270.

(ii) Density and air voids. No consistent trend exists which would indicate an increase or decrease in density over the three year evaluation period. Most of the measured densities met or exceeded the original construction specification requirement.

(c) Surface friction testing. Three years after construction, the Friction Numbers for all sections are adequate and have generally increased since the initial measurements were made following construction. The Friction Number increases are probably due to decreased binder coating on the surface aggregate (following construction) and increased surface texture with time.

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(d) Traffic. Since the construction of the test pavements, a total of about 3,400,000 vehicles have trafficked the test site (as of July 1982). Further a survey conducted during October 1979 characterized the type and weights of trucks traveling through the test sections.

(e) Surface cracking and wear. These two measures were used to evaluate the in-situ durability of the paving materials used in the study.

(i) Cracking. A pre-paving surface evaluation was conducted during August 1979. One of the principal features of the survey was a determination of all surface cracking. Further, a final crack survey was accomplished during July 1982. A comparison of these two surveys reveals that several reflection cracks have occurred in the 0/100 and 40/60 SEA sections and essentially none in the 30/70 SEA section. However, the pre-construction cracks in the 0/100 and 40/60 SEA sections were larger in width and hence would tend to reflect through the overlay sooner.

(ii) Wear. The 30/70 and 40/60 SEA mixtures have experienced a greater amount of surface wear in the wheel paths than the 0/100 (conventional hot-mix). However, the differences in wear are small (surface wear is defined as a loss of fine aggregate from the surface matrix). These differences are most probably due to the following reasons: vehicles which use studded or abrasive traction tires, the wet freeze-thaw cycles which occur in the Pullman area and the fact that SEA mixtures may exhibit slightly greater moisture susceptibility damage.

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