HIGHWAY RESEARCH BOARD

Bulletin 270

Asphalt Characteristics and Asphaltic Concrete Construction

National Academy of Sciences—
National Research Council

publication 808

Department of Materials and Construction

R. R. Litehiser, Chairman
Engineer of Tests, State Highway Testing Laboratory
Ohio State University Campus
Columbus

BITUMINOUS DIVISION

William H. Goetz, Chairman
Joint Highway Research Project
Purdue University
Lafayette, Indiana

A. B. Cornthwaite, Vice Chairman
Testing Engineer
Virginia Department of Highways
Richmond

COMMITTEE ON THE EFFECTS OF PHYSICAL AND CHEMICAL CHANGES ON BITUMINOUS MATERIALS

A. D. Morgan, Chairman
Resident Engineer
North Carolina State Highway Commission
Raleigh

Edwin J. Barth, New York
Carl A. Carpenter, Bureau of Public Roads, Washington, D.C.
James V. Evans, Director, Asphalt Research and Development, American Oil Company, Baltimore, Maryland
Harry K. Fisher, Consulting Engineer, Washington, D.C.
J.H. Goshorn, Managing Engineer, The Asphalt Institute, Columbus, Ohio
John M. Griffith, Engineer of Research, The Asphalt Institute, University of Maryland, College Park
F.C. Gzemski, The Atlantic Refining Company, Research and Development Department, Philadelphia, Pennsylvania
W.J. Halstead, Bureau of Public Roads, Washington, D.C.
J.H. Havens, Research Chemist, Kentucky Department of Highways, Lexington
Arnold J. Hoiberg, Assistant Director, The Flintkote Company, Whippany, New Jersey
J.O. Izatt, Senior Engineer, Products Application, Shell Oil Company, New York
W.G. O'Harra, Engineer of Materials, Arizona State Highway Department, Phoenix
Vaughn Smith, Vice President (Technical), American Bitumuls Asphalt Company, San Francisco, California
D.E. Stevens, California Crude Oil Sales Company, Perth Amboy, New Jersey
E.G. Swanson, Staff Materials Engineer, Colorado Department of Highways, Denver
R.N. Traxler, Texas Transportation Institute, Texas A and M College, College Station, Texas
W.B. Warden, Miller-Warden Associates, Swarthmore, Pennsylvania
J. York Welborn, Bureau of Public Roads, Washington, D.C.
Frank M. Williams, First Assistant Engineer of Tests, State Highway Testing Laboratory, Ohio State University Campus, Columbus
L.E. Wood, Department of Civil Engineering, Purdue University, Lafayette, Indiana

COMMITTEE ON THE EFFECT OF WATER ON BITUMEN AGGREGATE COMBINATIONS

A.B. Cornthwaite, Chairman
Testing Engineer
Virginia Department of Highways
Richmond

Fred A. Brill, Jr., Atlantic Refining Company, Philadelphia, Pennsylvania
Paul F. Critz, Bureau of Public Roads, Washington, D.C.
D.D. Dagler, District Engineer, The Asphalt Institute, Harrisburg, Pennsylvania
Jack N. Dybalski, Armour Industrial Chemical Company, Chicago, Illinois
A.W. Eatman, Materials and Tests Engineer, Materials and Tests Division, Texas Highway Department, Austin
James V. Evans, Director, Asphalt Research and Development, American Oil Company, Baltimore, Maryland
William H. Goetz, Joint, Highway Research Project, Purdue University, Lafayette, Indiana
F.C. Gzemski, The Atlantic Refining Company, Research and Development Department, Philadelphia, Pennsylvania
F.N. Hveem, Materials and Research Engineer, California Division of Highways, Sacramento, California
J.O. Izatt, Senior Engineer, Products Application, Shell Oil Company, New York
E.W. McGovern, Koppers Company, Inc., Tar Products Division, Technical Department, Verona, Pennsylvania
Hans F. Winterkorn, Head, Soils Physics Laboratory, Princeton University, New Jersey

Contents

Effect of Short Asbestos Fibers on Basic Physical Properties of Asphalt Pavement Mixes

J. H. KIETZMAN, Research Engineer, Johns-Manville Research Center, Manville, New Jersey

The results of comprehensive laboratory tests on asphalt paving mixtures with chrysotile asbestos are described. The main effect of the fiber appears to be on plastic strength as measured by static load tests. Static tensile strength of the control mix was increased as much as 20 times at standard asphalt contents by adding 3 percent 7M fiber. Similar increases in static compression strength were shown at asphalt contents higher than normal.

Static compression test data suggest that a wide range of flexibility characteristics might be attainable by adding asbestos, as a wide range of asphalt contents appears to be permitted.

Samples of asphaltic concrete removed from pavement mixes placed for service evaluation sustained a 510-psi concentrated compression load for 1 hr at 140 F without failure in laboratory tests.

Laboratory test results on control mixes suggest that use of a dynamic tension (cohesion) test at 140 F for measuring resistance to plastic deformation in mix design is justified. However, for pavement mixes with asbestos included, dynamic tests at 140 F do not appear to be adequate for measurement of resistance to plastic deformation.

Significant increases in dynamic tensile strength and repeated blow impact strength at 0 F were also evident when short asbestos fibers were included in pavement mixes.

The ability of short chrysotile fibers to prevent cracking in thin films of asphalt-filler mixes during accelerated weathering tests is described.

● CHRYSOTILE ASBESTOS is a hydrous magnesium silicate mineral ($Mg_6(OH)_8Si_4O_{10}$) with a fibrous structure. A few pertinent physical properties of chrysotile are given in Table 5, Appendix A, including tensile strength which is comparable per unit area to steel wire, The uniqueness of asbestos is in the fineness of its ultimate physical fiber size which accounts for its high flexibility and facilitates microscopic dispersion in a mixture.

A pertinent property is heat stability. Chrysotile may be heated at 350 F indefinitely with no change in tensile strength.

In addition, the surface of chrysotile fibers is electropositive. In asphaltic mixtures a chemisorption of the binder apparently occurs on the fiber surface which, together with its high surface area, makes for retention of excess binder chemically and mechanically.

The use of asbestos in connection with materials used in pavement construction is not new. For many years, asbestos has been a standard constituent of asphalt bridge planks, bituminous joint filling compounds, seal-coating compounds, and more recently in paint for asphalt curbing and pavement.

In each of these materials asbestos serves a distinct purpose, such as: producing toughness in bridge planks and joint filling compounds, weather resistance in seal-coating compounds, and resistance to cracking in paint for asphalt mixes.

Patents were obtained by the Warren Brothers Company of Boston in 1917 and 1918 for the use of asbestos in sheet asphalts. Warren Brothers' special uses for this mix included bridge pavements to prevent bleeding of asphalt during hot weather service.

Asbestos in small quantities is being used at present in some cold laid asphalt pavements to prevent segregation of aggregate during placement. (Information has been received that the Russians have been experimenting with short asbestos fibers in bituminous pavements.)

Because of the interest of the Corps of Engineers at the Waterways Experiment Station, Vicksburg, Miss. and the Asphalt Institute in College Park, My., the Johns-Manville Co. recently initiated an evaluation of asbestos in hot-mix asphalt pavements. The results of these tests are presented here with other data to demonstrate changes which take place in the basic physical properties of asphalt pavement mixes when short asbestos fibers are included.

Interpretations of these results are limited because the science of pavement design is at present a complex one and the criteria for pavement performance are not readily determined from laboratory data.

The tests in most cases were not designed to show maximum physical changes, inasmuch as the degree of change desired in most physical properties of a pavement will have to be determined by service performance of test pavements.

TEST METHODS

Materials

The tests described relate to three types of compacted mixes: fine aggregate asphalt mixes prepared in the laboratory, asphaltic concrete sampled from pugmill mixes during placement of experimental asphalt pavements, and asphalt-filler mixes from the same source of materials as the fine aggregate mixes.

For the fine aggregate mix the aggregate gradation of the Asphalt Institute Type IVa Dense Graded Surface Mix (Table 6, Appendix A) was chosen arbitrarily. The gradations were taken from the approximate center of the range allowed for each fraction in the specification, with the coarse fractions (retained on a #8 sieve) withheld and the other fractions, including asphalt, filler, and fine aggregate increased proportionately.

The fine aggregate mix used was a natural sand obtained from a local producer of asphalt pavement mixes. The sand was water-washed to remove clay and then dried at 220 F before being fractioned by sieving in a standard Rotex machine (model #12). Grade #300 limestone filler was dried at 220 F before sieving to remove +200 mesh grains. To obtain sufficient -100, +200 mesh material for the IVa aggregate gradation, it was necessary to use part of the +200 mesh fraction removed from the filler.

The bitumen used in the fine aggregate asphalt and filler-asphalt mixes was 60 penetration asphalt made from Venezuelan crude in the Curacao Refinery of the Shell Oil Company (identification #90-056).

Tensile Test Procedure

Tensile specimens of the IVa mix were prepared by the method outlined in Appendix C, using an ASTM briquet gang mold (1).

The static tension tests were performed by means of the simple apparatus shown in Figure 1. For tests at 140 F, the entire apparatus was placed in a glass-doored oven. The load was increased in increments every 30 min until failure occurred. For tests at 75 F the load was increased in increments of 5 lb; at 140 F, 1 lb increments.

For static tensile tests, the ultimate strength was calculated by summation of the load in psi multiplied by the time that the sample sustained the load in each step of the test.

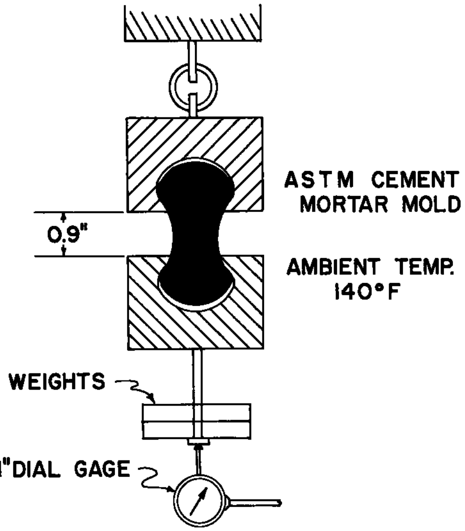


Figure 1. Static tension tests.

The criterion for failure was first visible cracking.

Stress movements were prevented in the dynamic tests by using a turnbuckle connection above the upper clamp and in all tests by lubrication of the inside of the clamp with silicone grease.

Use of this type of test for bituminous paving mixtures is not new. It was reportedly used in research by the Texas State Highway Department many years ago.

Dynamic tension tests were performed on a Scott X-3 testing machine. Each specimen was removed from a 140 F oven and tested immediately at a constant deformation rate of 1 in. per min. Failure occurred within 10 to 20 sec.

Static Concentrated Compression Tests

For static compression tests, specimens were prepared by the gyratory compaction method of the Texas Highway Department (2). To provide a 4-in. diameter briquette 2-in. in height 2,000-gram samples were used.

The hand gyratory equipment (Fig. 2) was mechanized simply. The bottom platen was mounted on the top of the specified hydraulic jack and guided by the four vertical columns which supported the upper platen. Fixed to the upper surface of the bottom platen was an 8-in. diameter rotary table driven by a compressed air motor. The base plate was attached to a cross slide (with hand wheel adjustment) mounted on the rotary table which allowed the die to be positioned off-center with respect to the center of the table at any desired distance. A ball thrust bearing between the base plate and the cross slide prevented transmission of rotational power from the rotary table to the base plate (and mold).

After preparation of mixes as specified by the Texas State Highway Department method, the mold with the mix inside and the base plate were placed on the cross slide in the center of the table. The bottom platen was lifted until a load of 50-psi gage pressure was observed. The cross slide was then positioned off-center by 0.1 in. using the hand wheel. Proper compaction of the sample was then achieved by the same procedure, and the criterion was the same as specified in the standard hand compaction method, except that the compressed air motor provided the gyration power (the speed of gyration was approximately 30 rpm). The sample was removed from the mold by use of an Arbor press after being cooled

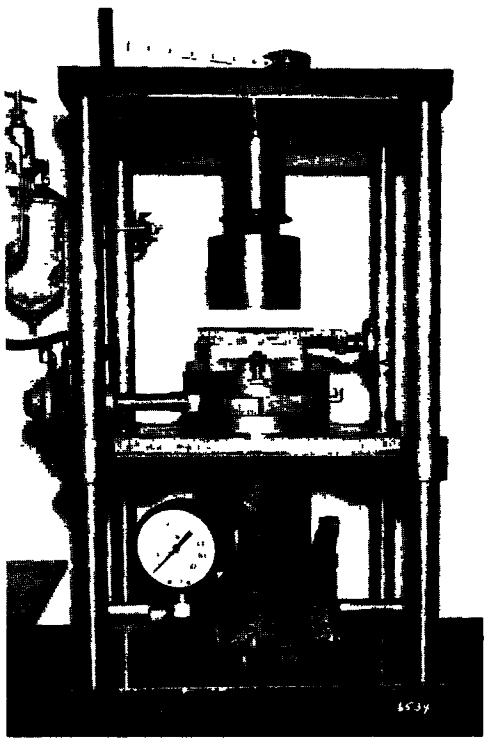


Figure 2.

in cold water for 2 min and was cured in air at room temperature for 48 hr before being tested.

The test was performed on the specimens immersed in a water bath at 140 F ± 1 deg. A cut-away schematic drawing of the apparatus is shown in Figure 3. During testing, the specimen rested in the bath on the base of a box frame which kept the loading rod positioned vertically but free to move up or down with a sliding fit inside a guide hole in the top of the frame. The base of the rod fitted into the top of the cylindrical indentor which transmitted the load to the sample.

Penetration of the indentor was measured with a 0.001-in. dial gage mounted with the foot resting on the top of the loading rod and decreasing the diameter of the indentor.

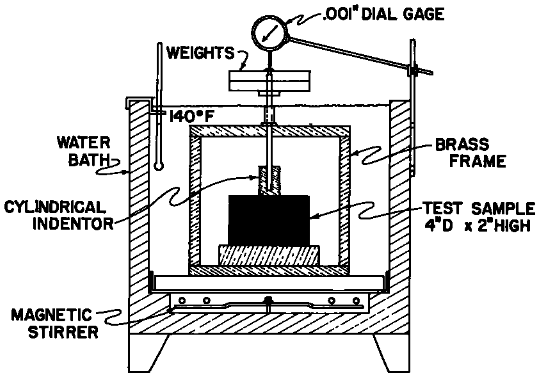


Figure 3. Static concentrated compression load test.

The load on the sample was increased incrementally (Table 1) by adding cyclindrical weights at the top of the loading rod and decreasing the diameter of the indentor. Each load was applied for 1 hr or until failure occurred.

ASTM Compression Tests

To compare the results of several different types of compression tests, the ASTM test D-1074-58-T (3) was performed on samples taken from experimental pavement mixes placed in Buffalo, N.Y. in August 1959. The standard procedure was followed except that the pugmill samples were reheated to 290 F before compaction. In addition, the test machine required controlling the rate of loading (10,000-lb per min) instead of rate of deformation.

Repeated Blow Impact Test

A ball drop test was used to measure impact strength of asphaltic concrete at 0 F. The method in general is a simplified version of the ASTM test D-3-18 (4).

The 500-gram samples of several mixes placed as test strips were reheated to 290 F, compacted according to the procedure specified in ASTM Method D-1074-58-T.

TABLE 1

Step	Diameter of Indentor, (in.)	Load lb	Load psi
1	1.00	10	13
2	1.00	20	25
3	0.50	25	127
4	0.35	25	260
5	0.25	25	510

The compacted specimens were cured at room temperature for 48 hr before density, and void measurement (ASTM). The nominal thickness of each 4-in. diameter specimen was 1 in.

Each specimen was conditioned at 0 F for 24 hr before testing. The test was performed in a cold room at 0 F with the specimen supported on a concrete base. A steel ball, approximately 0.9 lb in weight was dropped from a measured height and guided vertically by a steel tube of sufficient inside diameter to allow free fall. The tube was supported rigidly on a stand. A cord attached to the top of the ball permitted measurement of drop height from a scale marked on the outside of the tube. Drop heights were measured from the top surface of the sample. The ball was then positioned at the desired drop height inside the tube by suspension from the cord.

In each test the drop height was increased in increments of 3 in. starting with a 6-in. drop height. The drop height at which cracking was first visible was taken as maximum drop height. The summation of drop heights, including the maximum, was multiplied by the ball weight to determine total impact strength.

Asphalt Weatherability Tests

The thin-film weatherability test involves rather comprehensive methods which have been used for many years in research on industrial asphalts. The test includes film and panel preparation, exposure in standard weathering machines, and a method of measuring the degree of film failure, all of which are described in detail in ASTM Methods D-1669-59T, D-529-59T, and D-1670-59T, respectively.

Basically, film preparation consisted of pressing hot asphalt-filler mixes on a stainless steel plate between heated platens with metal spacer plates adjacent to the coated plate to insure the desired film thickness.

The films were exposed horizontally in Atlas Weatherometer machines in which the air temperature, cold water spray, and carbon arc radiation were alternated in prescribed cycles (Appendix B).

Film failure was measured by use of a spark tester to detect microscopic cracks. The method consists basically of passing a high voltage electrode (9,000 volts) across the face of the film and counting the spark discharges per unit area by placing a photographic paper between the film and the electrode.

TEST RESULTS

Tensile Tests

Figure 4 shows a comparison of tensile strengths of the IVa fine aggregate samples with 4 percent total mineral filler content (Fig. 5) under three different testing conditions: static loading at 140 F, dynamic loading at 140 F, and static loading at 75 F. It should be noted again that the weight percentages given are all based on the original proportions which included approximately 50 percent coarse aggregate.

Perhaps the most obvious conclusion from this comparison is that the relative difference in tensile strength between standard and asbestos mixes are much greater under static loading at 140 F than under dynamic loading at 140 F. Under static loading the asbestos samples were as much as 20 times stronger than the standard samples, considering the load multiplied by the time as static strength. Under dynamic loading, however, the asbestos mix showed a maximum about 85 percent greater strength than the control mix.

In addition, the extensibility at failure of the samples under static loading at 140 F appears to be considerably less than under dynamic loading at 140 F especially in the case of the asbestos mixes. Extensibility under static loading at 140 F also appears to be significantly less than at 75 F.

There appears to be much greater similarity in comparative values between the static tests at 75 F and the dynamic tests at 140 F than between the static tests at 140 F and the dynamic tests at 140 F or between the results of static test at 75 F and the static tests at 140 F.

The results of dynamic tensile tests at 140 F appear to be similar to typical Marshall Stability Data for asphaltic concrete (Appendix C). A correlation between cohesional (tensile) strength and Marshall has been demonstrated before (5).

Static Compression Tests

Static concentrated compression tests with incremental increase in load were per-

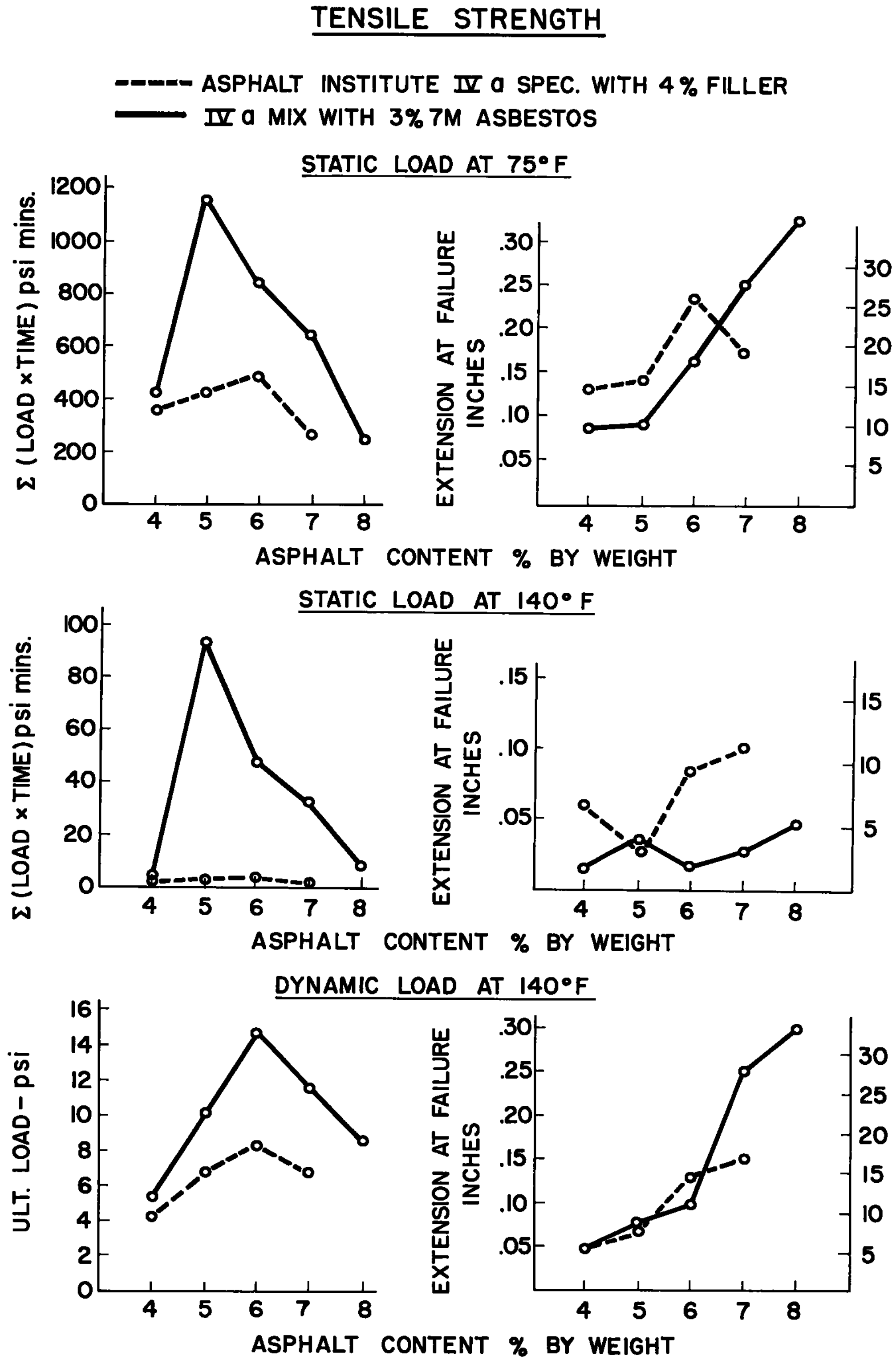

TENSILE STRENGTH
ASPHALT INSTITUTE IV a SPEC. WITH 4% FILLER
IV a MIX WITH 3% 7M ASBESTOS
STATIC LOAD AT 75° F
Σ (LOAD × TIME) psi mins.
EXTENSION AT FAILURE INCHES
%
ASPHALT CONTENT % BY WEIGHT
STATIC LOAD AT 140° F
Σ (LOAD × TIME) psi mins.
EXTENSION AT FAILURE INCHES
%
ASPHALT CONTENT % BY WEIGHT
DYNAMIC LOAD AT 140° F
ULT. LOAD - psi
EXTENSION AT FAILURE INCHES
%
ASPHALT CONTENT % BY WEIGHT

Figure 4.

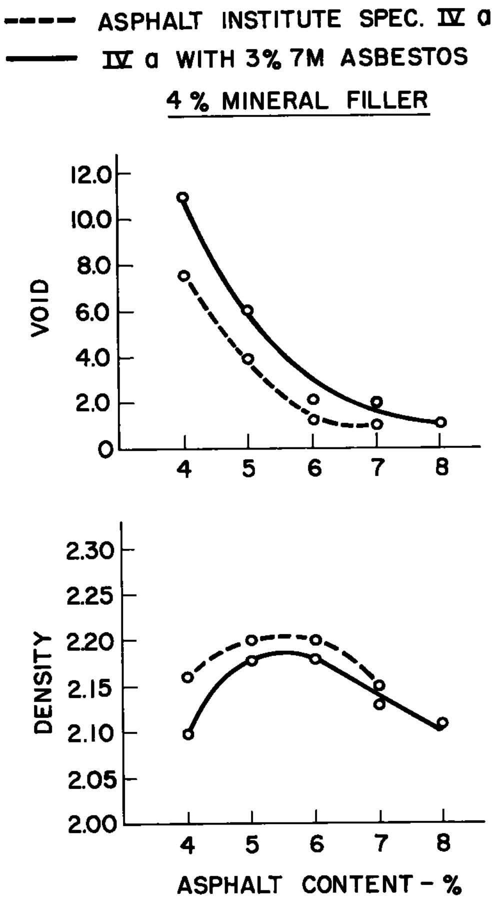


Figure 5. Average density and void content for tension test samples.

formed on mixes based on the Asphalt Institute IVa aggregate gradation in which the coarse aggregate was withheld, keeping all other constituents in the same relative proportions. The asphalt used was 60 penetration.

In Figure 6, the static compression test results are shown for samples with 4 percent mineral filler compacted by the gyratory method (a mechanized model of the Texas State Highway Department hand gyratory compaction method) which simulates actual pavement compaction procedure.

Maximum static compression strength was approximately 30 psi-hr for the control mix and 230 psi-hr for the asbestos mix. The maximum occurred at 5 percent asphalt content in the control mix but at 7 percent asphalt content in the asbestos mix. More directly, the maximum compression load sustained for 1 hr by the strongest control sample at 140 F was 25 psi. The maximum load sustained by the asbestos samples was 127 psi.

While penetration at failure in the standard samples increased continuously as asphalt content was increased, the asbestos mixes showed a peak in the curve at 5 percent asphalt content (the asphalt content at which maximum static tensile strength occurred in the same mix). The decrease in penetration at 6 percent asphalt content was followed by a second higher peak at 7 percent asphalt content which paralleled the peak in the strength curve. One of the asbestos samples with 8 percent asphalt content evidenced complete penetration of the indentor without cracking.

The void contents of these samples with 4 percent mineral filler (Fig. 5) indicate that compaction at 5 percent asphalt content in the standard mix was duplicated at approximately 6½ percent asphalt content in the asbestos mix. Compaction in the standard mix at 6 percent asphalt was duplicated at 8 percent asphalt content in the asbestos mix.

The "drop-off" in static compressive strength for the control mix as asphalt content was increased paralleled the drop-off in tensile strength (dynamic and static) and the typical rapid decrease in stability shown in standard mix design tests, which involve dynamic loading. Therefore, any of the dynamic tests could be used to determine the optimum asphalt content for static compressive strength.

However, the static compressive strength of mixes with asbestos did not show this progressive decrease as asphalt content was increased (as occurred in the tensile tests). This implies that the asbestos produced an effect apart from cohesional strength and suggests that "dynamic" mix design tests at 140 F could not be used to determine the optimum asphalt content for static compressive strength of paving mixes which contain asbestos fiber.

Asphaltic Concrete Mixes.—The static concentrated compression test was performed on samples removed from pavement mixes (N.Y. State DPW Physical Research Project No. 11) placed on a Thruway in Buffalo, N.Y. In Figure 7, typical load vs time curves are shown for these mixes after compaction at 260 F by the ASTM compression test method. The standard 2A mix sustained the 127-psi load at 140 F but failed almost immediately under the 260-psi load. The experimental mix with 1.3 percent 7M asbestos and approximately the same asphalt content showed approximately the same static strength as the standard 2A mix but evidenced almost twice the penetration at failure as was shown by the standard mix. Mix C also containing 1.3 percent asbestos, but with asphalt content increased 1 percent to 8.5 percent, evidenced a marked increase in strength and sustained both the 260-psi load for 1 hr and in one case the 510-psi load without plastic failure and without cracking. A third experimental mix (B) with 2.5 percent asbestos and 8.5 percent asphalt content also sustained the maximum load of 510 psi without failing. In mix B and mix C, and initial yield under 510-psi load was followed by a unique strength recovery which was not observed in the static compression tests on the IVa mixes with the coarse aggregate withheld. (However, similar strength recoveries were noted in the static tensile test with asbestos mixes at 7 percent and 10 percent total mineral filler content, but at a much lower strength.)

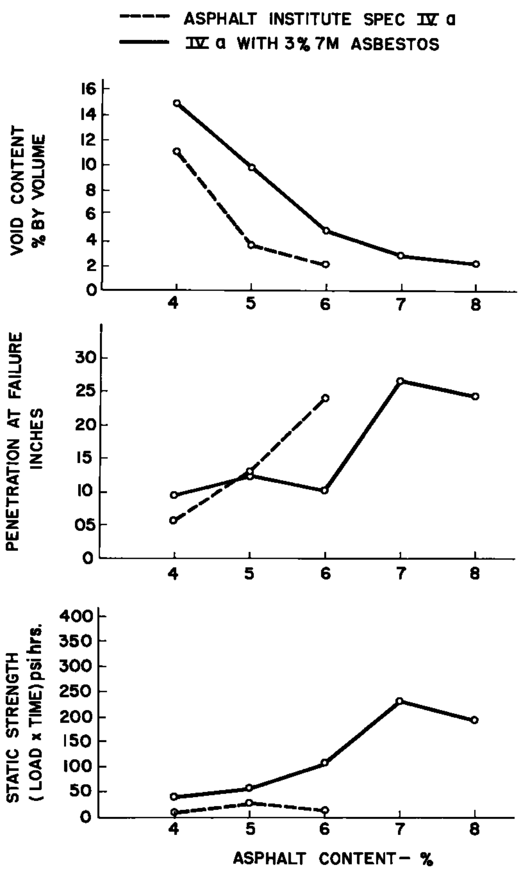


Figure 6. Static concentrated compression test at 140 F, 60 penetration asphalt, 4 percent mineral filler.

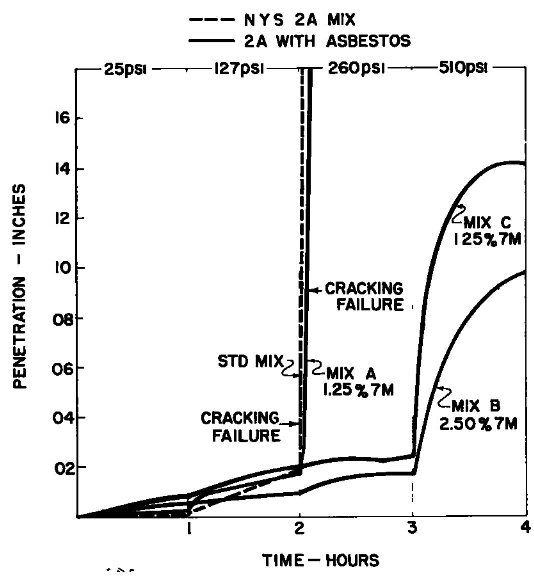


Figure 7. Static concentrated compression tests at 140 F on pavement mixes placed in Buffalo, N.Y.

The difference in static compression strength between mixes A and C, both with 1.3 percent asbestos, is at least partly due to the greater compaction allowed by the higher asphalt content.

Marshall stability and flow properties (measured by the Buffalo Crushed Stone Company on the samples removed from mixes and compacted while still hot) for these

TABLE 2

COMPRESSION TEST RESULTS ON SAMPLES OF MIXES PLACED ON THE SCAJACADA CREEK EXPRESSWAY IN BUFFALO, N.Y.

Mix	Asb.[1] Cont. (%)	Asph.[2] Cont. (%)	Density[3] (gm/cc)	Void Cont.[3] (% vol.)	Static Conc. Comp. Test[3] Max. Load[4] (psi)	Total Strength (psi-hr)	Pen. at Fail. (in.)	Comp. Strength[5] (psi)	Marshall Values[6] Stab. (lb)	Flow (0.01 in.)
2A[7]	0	7.6	2.28	3.9	127	172	0.05	615	886	22
Mod. 2A:										
A	1.3	7.8	2.29	3.0	127	141	0.09	775	1245	16
B	2.5	8.4	2.28	2.2	510	922	-[8]	872	1098	34
C	1.3	8.6	2.33	0	510	861	0.47	664	673	53

[1]Type 7M06. [2]60 penetration. [3]Samples compacted by method described in ASTM D-1074-58T. [4]Sustained, 1 hr. [5]ASTM method at 75 F. [6]Tests performed at Buffalo Crushed Stone Corp. under direction of J. F. Morgan, Jr. [7]Control mix; N.Y. State Dept. Public Works specification for bituminous top course. [8]No failure.

mixes are given in Table 2, which shows that the maximum increase in Marshall stability is roughly comparable to that shown by the Marshall test data previously described.

One pertinent question seems to beg an answer at this point. In light of the rather marked increase in static tensile and compression strength of asbestos mixes, is the moderate increase in maximum load the only evidence in the Marshall test of the effect of adding asbestos? According to the report of the producers of the experimental asphalt mixes (report of Experimental Bituminous Mixes produced in Buffalo Crushed Stone Corporation plant #1 on August 11, 1959—Cooperative program with New York State Department of Public Works, Physical Research Project No. 11), "All specimens

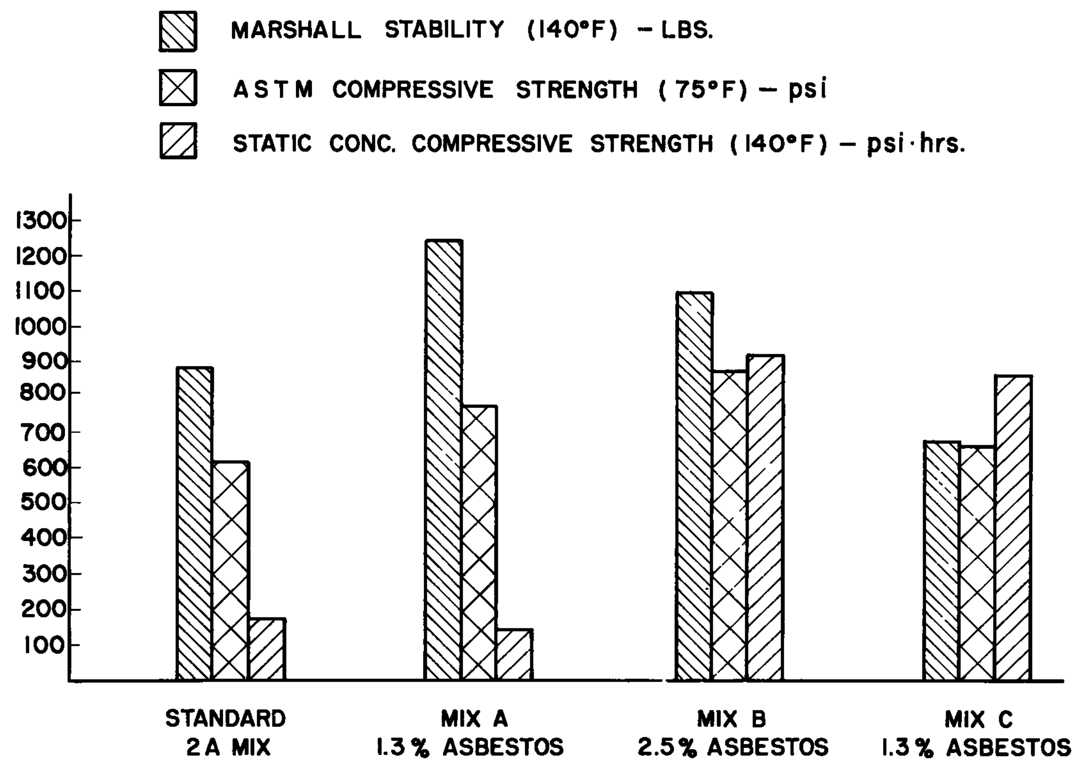


Figure 8. Compressive strength of pavement mixes placed in Buffalo, N.Y.

of mixes A, B, and C when reaching maximum stability, or load, remained at that value throughout the limit of distortion permitted by the equipment. The control mix after reaching maximum stability or load then fell off in value in the normal or expected manner as distortion continued." Similar Marshall test results have been reported from other sections of the country.

Comparison of Three Results on Test Pavement Mixes in Buffalo, N.Y.—Table 2 also gives the relative compressive strengths of the experimental mixes tested by the Marshall method, the ASTM compression test, and a static concentrated compression test (penetration test).

Void contents are believed to be of relative value only, because of errors inherent in the method of measurement. Because the densities and void contents of the Marshall samples are not known, comparing Marshall stability with the results of the other two tests may not be completely justified. However, Figure 8 shows results of these three compression tests.

These comparisons bear out the previous contention that use of the standard Marshall test criteria by itself, or any dynamic test, to evaluate asbestos-asphalt mixes might not be necessarily adequate.

TABLE 3

SUMMARY OF REPEATED BLOW IMPACT TESTS AT 0 F (SAMPLES TAKEN FROM PAVEMENT MIXES PLACED AS TEST STRIPS IN NEW JERSEY AND NEW YORK STATE)

Specification	Mix	Asbestos Content[1] (% by wt)	Asphalt (% by wt)	Asphalt Pen.	Density (g/cc)	Void (% by vol)	Impact Strength Max. Drop (in.)	Impact Strength Total[2] (ft/lb)
N.J. FA-BC	Std.	0	6	82	2.33	7.9	21	5.9
	Mod.	2	6		2.26	9.9	33	14.3
N.J. MA-BC	Std.	0	6	60	2.36	8.6	26	9.0
	Mod.	3	7		2.38	4.7	37.5	18.4
N.Y. State 2A	Std.	0	7.6	60	2.26	3.9	25.5	8.7
	Mod. (A)	1.3	7.8	60	2.29	3.0	30	12.0
	Mod. (C)	1.3	8.6	60	2.33	0	33	14.3
	Mod. (B)	2.5	8.4	60	2.27	2.2	37.5	18.4

[1]7M fiber.
[2]Summation of drop heights to failure multiplied by weight of ball.

It can be seen that the asbestos mix which showed the highest Marshall stability displayed the lowest static compressive strength.

Mix C, although lowest in Marshall stability, showed a static (concentrated) compressive strength more than twice that of the standard, even though the void content was effectively reduced to zero. In addition, mix C showed the greatest penetration under concentrated loading without cracking.

In general, the results of the asbestos mixes showed that as asphalt content was increased, void content decreased, Marshall stability decreased and static compressive strength increased.

Repeated Blow Impact Test

The results of impact tests at 0 F on specimens of asphaltic concrete made in the laboratory from samples taken from pugmill mixes are given in Table 3.

Failure in all cases was by transverse cracking of the 4-in. diameter briquettes. Fracture of the coarse aggregate particles at impact failure suggests that the impact strength of the matrix (as measured by this test) was roughly equivalent to that of the coarse aggregate.

Repeated blow impact strength at 0 F was apparently proportional to asbestos content and asphalt content. The maximum increase in impact strength was 47 percent measured by maximum drop height and 110 percent measured by total impact load in the New York State 2A mix with 2.5 percent total weight 7M asbestos.

Thin-Film Weatherability Tests

A series of accelerated weathering tests on thin films of tar emulsion containing short asbestos fiber were performed by the Corps of Engineers at Vicksburg in 1957. The favorable results of their tests prompted similar tests at the Johns-Manville Research laboratory on several grades of paving asphalt with mineral filler added in the proportions similar to the filler-asphalt ratio used in asphaltic concrete.

The purpose was to measure the comparative ability of a standard filler and short asbestos fibers to maintain the structural integrity of the thin films of asphalt between fine aggregate and/or mineral filler grains.

In Figures 9 through 12, typical visual failure is shown for thin films of 82 penetration asphalt after 100 hr of accelerated weathering (cycle A). Figure 9 shows a film of

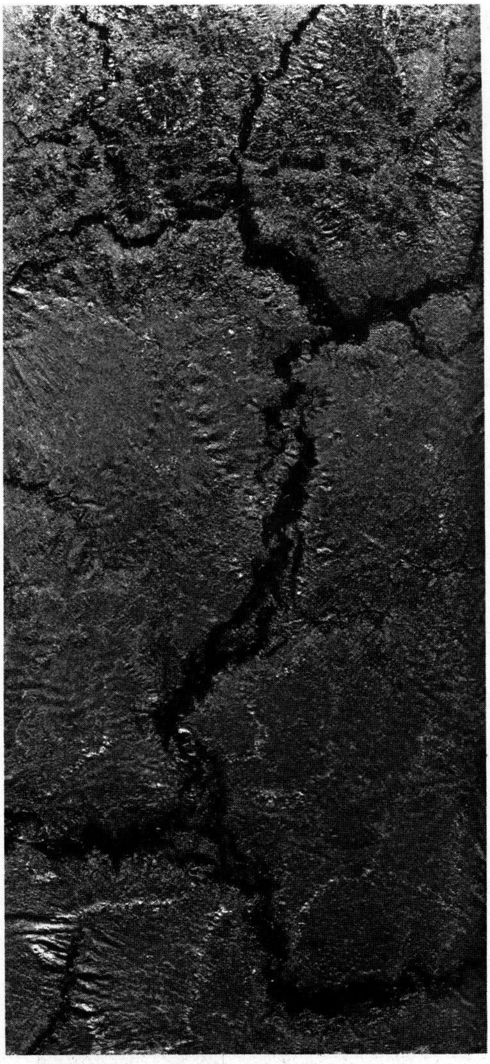


Figure 9. Thin film (0.010 mil) of 82 penetration asphalt without mineral filler after 100 hr of accelerated weathering.

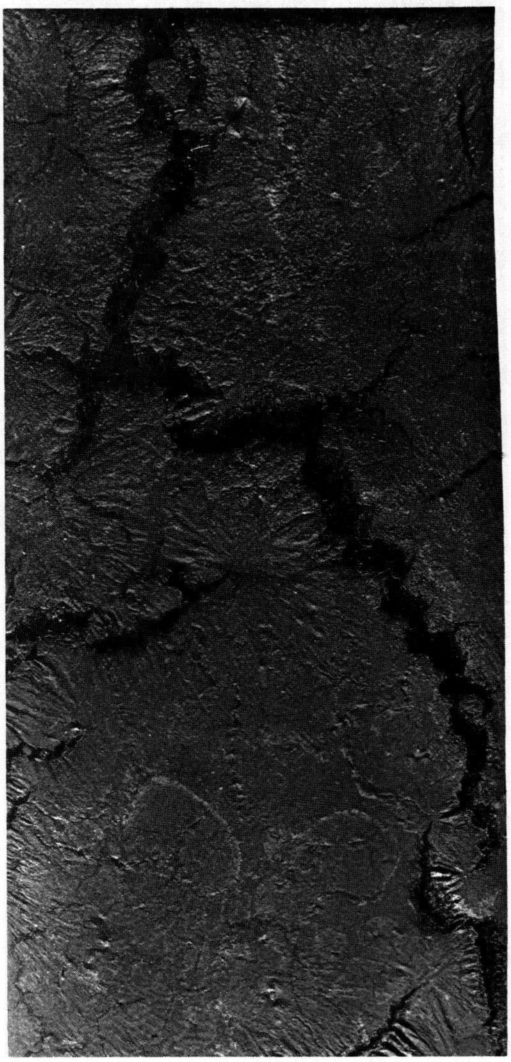


Figure 10. Film (0.011 mil) of 82 penetration asphalt with 50 percent by weight limestone filler (-200M) after 100 hr of accelerated weathering.

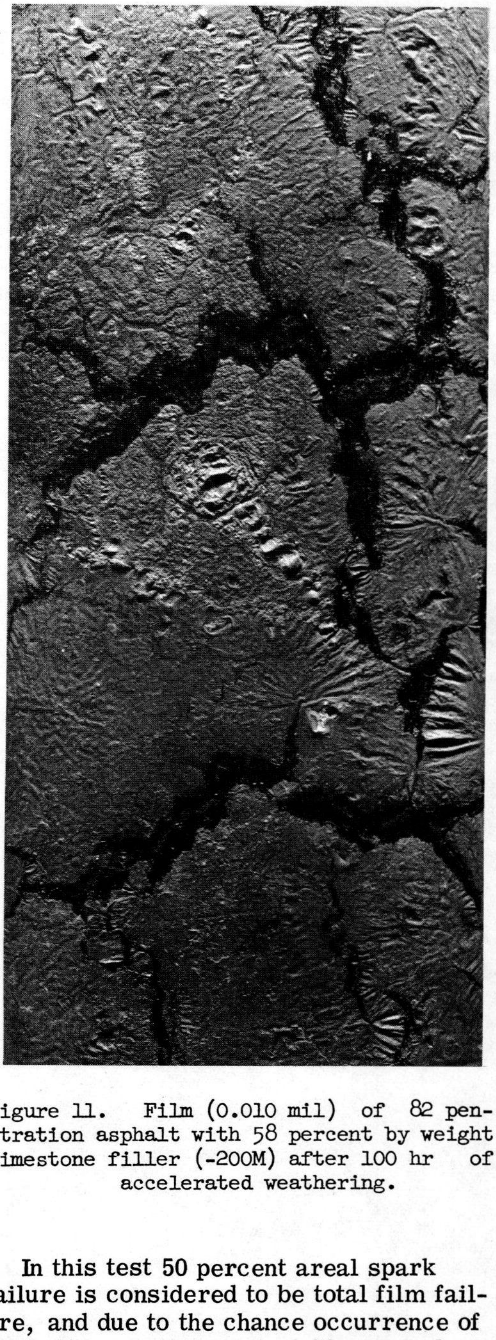


Figure 11. Film (0.010 mil) of 82 penetration asphalt with 58 percent by weight limestone filler (-200M) after 100 hr of accelerated weathering.

In this test 50 percent areal spark failure is considered to be total film failure, and due to the chance occurrence of air bubbles in the original film, areal spark failure of less than 5 percent is not considered significant.

Film failure in these tests is apparently the result of repeated exposure to cyclic high and low temperatures combined with the chemical or physical changes resulting from the combined effects of air, deionized water and ultra- a mixture of 50 percent and 57 percent -200 mesh limestone filler, respectively, and 82 penetration asphalt. Figure 12 shows films in which 20 percent by weight of limestone was replaced by an equal weight of 7M asbestos.

The results of exposure of these films in standard weatherometer machines, the intermittent use of a spark test to detect microscopic holes, and a photographic trace of these holes to measure film failure quantitatively is shown in Figure 13.

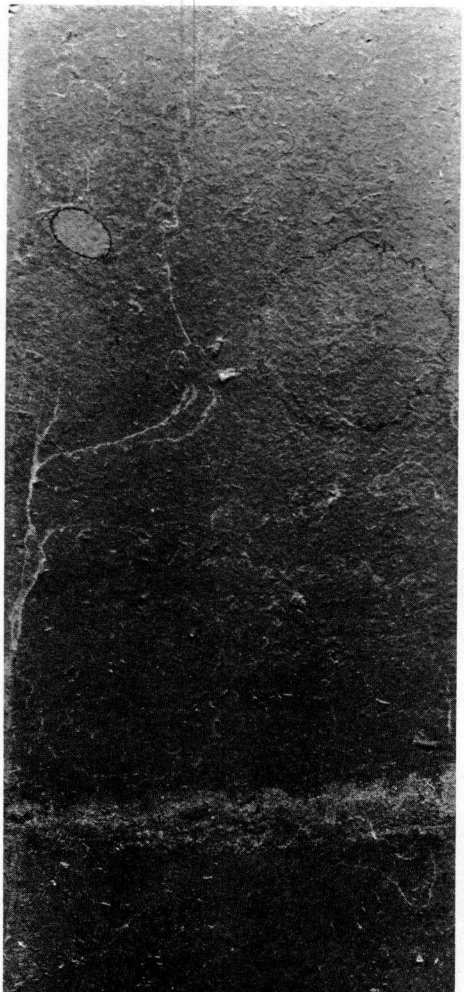


Figure 12. Film (0.012 mil) of 82 penetration asphalt with 30 percent limestone and 20 percent asbestos after 100 hr of accelerated weathering (rough surface is original film surface).

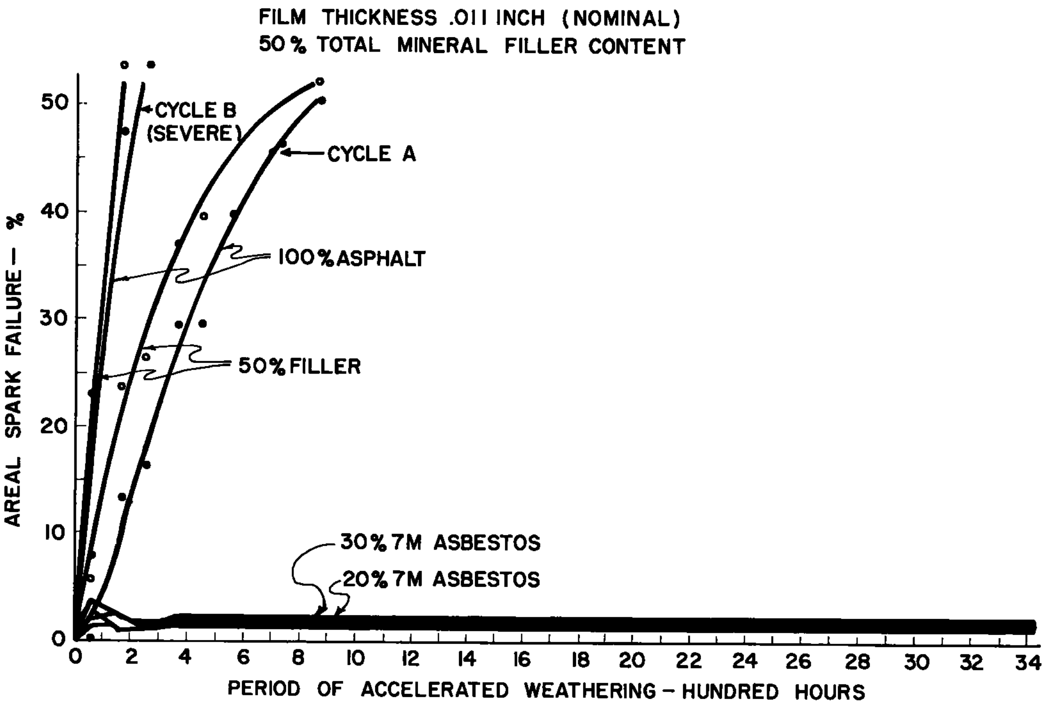


Figure 13. Weatherability of thin films of 60 penetration asphalt (weatherometer cycles A and B).

violet rays which simulate weathering conditions. The test measures the reaction of each film to the cumulative internal stresses which result from these conditions.

Films of pure 60 penetration asphalt of 11-mil thickness showed rapid film failure with a progressively decreasing rate of failure. Total film failure (50 percent areal spark failure) was at 900 hr of accelerated weathering under the moderate weathering cycle (A) and at 200 hr under the extreme weathering cycle (B).

Films of 60 penetration asphalt with 50 percent limestone filler (-200 mesh) showed no significant improvement in over-all rate of film failure at either weathering cycle.

In films of asphalt with or without 50 percent limestone filler, the rate of film failure under severe weathering was approximately six times faster than identical films tested under the less severe weatherometer cycle.

Films containing 20 percent and 30 percent short asbestos fiber showed no measureable film failure through to the end of the test period (3,400 hr).

Severity of accelerated weathering appeared to have no effect on the ability of the fiber to maintain structural integrity of the film.

DISCUSSION OF RESULTS

Asphalt Content

It is generally agreed that except for the effect on plastic strength, an increase in asphalt content in standard asphaltic concrete mixes would be desirable.

If the static concentrated compressive strength is a valid measure of resistance to plastic deformation, then, based on previously described data, use of 3 percent 7M asbestos allows an increase in asphalt content of 2 percent total weight of mix. For a significant increase in static compressive strength, asphalt content should be increased by at least 1 percent above the standard mix.

However, if maximum cohesional (tensile) strength were used as the criterion for optimum asphalt content, asphalt content of the mix with 3 percent asbestos would be kept the same as the standard mix or decreased slightly, depending on the type of test (static or dynamic) and temperature of the test.

The question which logically follows is why increase asphalt content? Most of the reasons suggested for increased asphalt content seem to be related to flexibility properties. (One other reason for increasing asphalt content, independent of flexibility, is increased weather resistance (of surface courses).

Flexibility

Although resistance to plastic deformation appears to be considered by paving engineers to be the most important strength characteristic of bituminous pavement at elevated service temperatures, flexibility is generally considered to be the main strength property of importance at normal or low ambient temperatures.

Flexibility of an asphalt pavement has been defined by Monismith in two ways which reflect the viscoelastic nature of asphalt paving mixes: the ability "to conform to variations in the base and subgrade elevations of the pavement structure and the ability to bend repeatedly without fracture (6)," or simply conformability and flexural fatigue.

According to Monismith ..."the greater the amount (of asphalt)..... the more able should be the mixture to conform to base and subgrade variations. From the standpoint of fatigue resistance, it would also appear that the greater the amount of asphalt, the better able would be the mixture to withstand repeated deflections since the asphalt would exist in thicker films (6)."

The first point has apparently been established from field and laboratory evaluations. In general, the static load extensibility and penetration (1) of the control mix in the Johns-Manville Research Center tests bears out this conclusion that extensibility at failure (a measure of conformability) varies with asphalt content. However, although extensibility at failure of the control mix under static loading at 75 F showed a progressive increase as asphalt content was increased up to 6 percent, it decreased from 6 to 7 percent asphalt content. In addition, the extensibility of the control mix under dynamic and static loading at 140 F appears to be approaching a maximum at 7 percent asphalt content.

In the asbestos-asphalt mixes, the data show that extensibility under static loading at 75 F increased continuously as asphalt content was increased up to 8 percent asphalt (the maximum asphalt content tested).

From the standpoint of flexibility, the fact that asbestos allows a large increase in asphalt content of standard paving mixes may be the most important consideration.

Monismith (6) has presented data which demonstrate that flexural fatigue strength at 75 F increased as asphalt content increased. His data also showed a large increase in modulus of rupture (dynamic flexural strength) due to increasing asphalt content in a standard asphaltic concrete.

The fact that a high static load strength was maintained in the asbestos mix as asphalt content was increased up to 8 percent (or greater) and extensibility or penetrability increased progressively suggests that it may be possible to obtain a wide range of flexibility characteristics. On a resilient base, a pavement with a large increase in asphalt content might be desirable. For a thin pavement, with inherent high flexibility, possibly a small increase in asphalt content might conceivably be best.

Asbestos Content

In general, results to date suggest the following. Based on thin-film weatherometer tests, adequate stabilization of the asphalt-filler during weathering would require 1 percent total weight 7M asbestos. For essentially complete stabilization 2 percent 7M would be required. The amount required probably is less for low penetration asphalts and may conceivably be higher for high penetration asphalts.

Although the strength tests described herein pertain mostly to mixes with 3 percent 7M asbestos, limited tests with other fiber contents suggest that strength properties are proportional to fiber content, with compaction characteristics and total mineral filler content controlling this relationship at low and high asphalt contents, respectively.

On a strength basis, the amount of fiber desired would be dependent on how much of any type of strength was desired.

From the standpoint of resistance to plastic deformation alone, it appears possible from static compression tests that as little as 1 percent asbestos might be adequate for some mixes with the asphalt content determined by the test criterion (increased asphalt content for optimum static strength, standard asphalt content—5 to 6 percent—for optimum dynamic strength).

With the possible exception of resistance to plastic deformation, even an approximate idea of how much strength, flexural, impact, etc. would require field evaluation of test strips with varying fiber content, asphalt content, and total mineral filler content.

CONCLUSIONS

Because of the difficulties involved in extrapolation of small-scale laboratory test results to pavements under service conditions, the following tentative conclusions do not necessarily apply generally to all pavement mixes or to pavement performance which can only be determined by field evaluation.

The conclusions relating to pavement strength at high temperatures are based on the assumptions that plastic strength of asphalt pavement is of critical importance rather than elastic strength and that a static load test is a valid, if somewhat rigorous, measure of plastic strength.

The test data suggest that asbestos fibers may greatly increase plastic strength of asphalt mixes while allowing a relatively wide range of asphalt contents, which should permit a corresponding range of flexibility characteristics as desired.

Based on the test data, the use of dynamic tension (cohesion) tests to determine optimum asphalt content for maximum plastic compressive strength would be justified for the control (standard) mix, but not for the asphalt mixes with asbestos included.

Relatively large increase in dynamic tensile strength at 75 F and 140 F of the control mix and in repeated blow impact strength at 0 F were obtained by the addition of chrysotile (asbestos).

Thin-film accelerated weathering tests suggest that inclusion of short asbestos fibers effectively maintains structural integrity of the asphalt exposed at the surface for an indefinite period of time.

ACKNOWLEDGMENTS

The author would like to thank George Clarvoe and his staff for their advice and assistance throughout the evaluation. Special thanks should be given to Charles Bloom for his help in the weatherability tests. Frank Hunt and J.J. Scanlan were indispensable in carrying out the laboratory evaluation. Foster Hulsizer of the Mechanical Section designed the mechanization of the Texas gyratory compaction apparatus.

John Griffith, Ben Kallas, and Harry Thompson of the Asphalt Institute offered valuable advice during the survey which preceded the laboratory investigation.

The advice and criticism of paving technicians throughout the country are gratefully acknowledged.

REFERENCES

1. ASTM Standards 1958, Part 4, Method C-190-58; Tensile Strength of Hydraulic Cement Mortars.
2. "The Design of Asphaltic Concrete Paving Mixtures." Texas Highway Dept. Const. Bul. C-14, Part I.
3. ASTM Standards 1958, Part 4, Compressive Strength of Bituminous Mixture (Tentative).
4. ASTM Standards 1958, Part 4, Method of Test for Toughness of Rock.
5. Benson, F.J., "Appraisal of Several Methods of Testing Asphaltic Concrete." Texas Eng. Exp. Sta. Bul. No. 126 (June 1952).
6. Monismith, C.L., "Flexibility Characteristics of Asphaltic Pavement Mixes." AAPT Proc., Vol. 27 (1958).

7. Asphalt Institute, Specifications and Construction Methods for Hot-mix Asphalt Paving, Specification Series No. 1, reprinted August 1957.

Appendix A

TABLE 4

Quebec Stand. Classif.		Minimum Wt.[1] (Oz)			
Group	Sample	½ Mesh	4 Mesh	10 Mesh	Pan
3:	3R	2	8	4	2
4:	4K	0	4	9	3
5:	5R	0	0	10	6
6:	6D	0	0	7	9
7:	7D	0	0	5	11
	7F	0	0	4	12
	7H	0	0	3	13
	7K	0	0	2	14
	7M	0	0	1	15
	7R	0	0	0	16

[1]Quebec Standard test, Canadian Chrysotile Asbestos Classification.

TABLE 5

PHYSICAL PROPERTIES OF CANADIAN CHRYSOTILE ASBESTOS

Specific gravity[1]	2.55
Fiber diameter[2] (in.)	0.000000706 to 0.00000118
Fibrils[1] in 1 in. (no.)	850,000 to 1,400,000
Tensile strength[1,2] (psi)	100,000 to 355,000

[1]Badollet, M.S., Canadian Mining and Metall. Bul., Trans., 54:151-160 (Apr. 1951).
[2]Zukowski, R., and Gaze, R., Nature, 183:35-51 (Jan. 1959).

TABLE 6

COMPOSITION OF TYPE IVa DENSE-GRADED SURFACE MIX[1]

	Percent Passing by Weight[2]	
Sieve Sizes	Asphalt Institute Spec.	Fine Aggregate Mix
½ in.	100	-
⅜ in.	80-100	-
#4	55-75	-
#8	35-50	40
#30	18-39	21
#50	13-23	15
#100	8-16	9
#200	4-10	4

[1]See Ref. 1. Bitumen content for Asphalt Inst. Spec., 3.5 to 7.0 percent; for fine aggregate mix, 3 to 8 percent.
[2]For the Johns-Manville Research Center laboratory evaluation all aggregate retained on a #8 screen was excluded, increasing the weight proportions of the other constituents—asphalt, filler, andfine aggregate fractions so as to obtain the same relative proportions which would exist in a mix with the coarse aggregate included.

TABLE 7

PHYSICAL PROPERTIES OF ASPHALT[1] USED FOR IVa FINE AGGREGATE LABORATORY MIXES AND THIN FILM WEATHEROMETER SAMPLES

Penetration Grade	60	82
Penetration, mm/10 100 g for 5 sec	60	82
Specific gravity	1.023	1.023
Heat susceptibility (5 hr at 325 F)		
Loss on heating (%)	0.025	0.039
Penetration, mm/10	55	69

[1]From Venezuelan crude at Curacao refinery, Shell Oil Co.

TABLE 8

BUFFALO CRUSHED STONE CORPORATION NEW YORK STATE SPECIFICATION 2A FOR TOP COURSE OF BITUMINOUS PAVEMENT[1]: ROTAREX EXTRACTION OF 500-g SAMPLE

Passing	Retained	Control Mix (%)	Mix A (%)	Mix B (%)	Mix C (%)
1 in.	½ in.	None	None	None	None
½ in.	¼ in.	6.8	4.6	6.6	5.8
¼ in.	⅛ in.	24.2	26.4	21.8	24.8
⅛ in.	40	20.0	17.7	25.3	22.9
40	80	23.4	21.7	20.2	20.8
80	200	15.0	16.3	11.0	11.2
200	-	5.0	4.2	4.2	4.6
Asbestos		None	1.3	2.5	1.3
Asphalt cement		7.6	7.8	8.4	8.6
		102.0	100.0	100.0	100.0

Note: All aggregates were limestone.
[1]Placed on the Scajacada Creek Expressway, August 1959 as part of Physical Research Project No. 11 of the New York State DPW, Bureau of Physical Research.

Appendix B

DESCRIPTION OF WEATHEROMETER CYCLES USED IN THE THIN FILM WEATHERABILITY TESTS

Cycle A

Below are shown the successive steps in one 21-hr cycle which was repeated each day for five successive days per week:

Step	Time Period	Condition
1	1.0 hr	Cold water spray (deionized water at 40 F).
2	1.5 hr	Radiation from single carbon arc ultraviolet lamp. Black bulb temperature 140 to 145 F. Air temperature 115 to 120 F.
3	2.0 hr	Cold water spray (as in Step 1 above).
4	16.5 hr	Light exposure (as in Step 2 above).

<u>Cycle B</u>

Continuous ultraviolet radiation for 21 hr (as described under Cycle A).

Every 51 min, a 9-min duration of cold water spray (40 F, deionized).

Specimens were exposed in the horizontal position 9 in. below the carbon arc with the length of the plate pointing toward the (vertical) axis of machine rotation and the inside end of the plate 8 in. from the axis.

Appendix C

TENSILE TEST SAMPLE PREPARATION

The following procedure was used to prepare the tensile test samples of the IVa fine aggregate mixtures. Each operation is listed in the chronological order. Only one sample (130 gs) was prepared from each batch (300 gs) of mix:

1. Aggregate and mineral filler fractions were combined and preheated at 350 F.
2. The asphalt was heated to a temperature of 300 F with constant stirring.
3. The aggregate, mineral filler, and asbestos (asbestos was added purposely without preheating or drying) were mixed in a preheated 5-qt stainless steel bowl for 30 sec.
4. The preheated asphalt was added to the aggregate mixture and mixed by hand for 30 sec.
5. 120 gs of the mixture was then placed by hand into a briquet gang mold which had been preheated to 250 F and wiped with a rag saturated with silicone lubricant.
6. With the mortar mold clamped rigidly onto a ½-in. steel plate (also preheated) the mixture was compacted at a load of 3,000 psi for a period of 2 min with a top plunge preheated at 250 F with the same configuration as the mold, allowing for 0.005-in. clearance between the plunge and the inside dimensions of the mold all the way around.
7. With the top plunger bearing against a spherically based compression block, a load of 3,000 psi was applied to the mixture in the mold for a period of 2 min.
8. The sample was removed from the mold after cooling for 2 min. in cold water.

The following procedure describes the methods used to determine density and void contents:

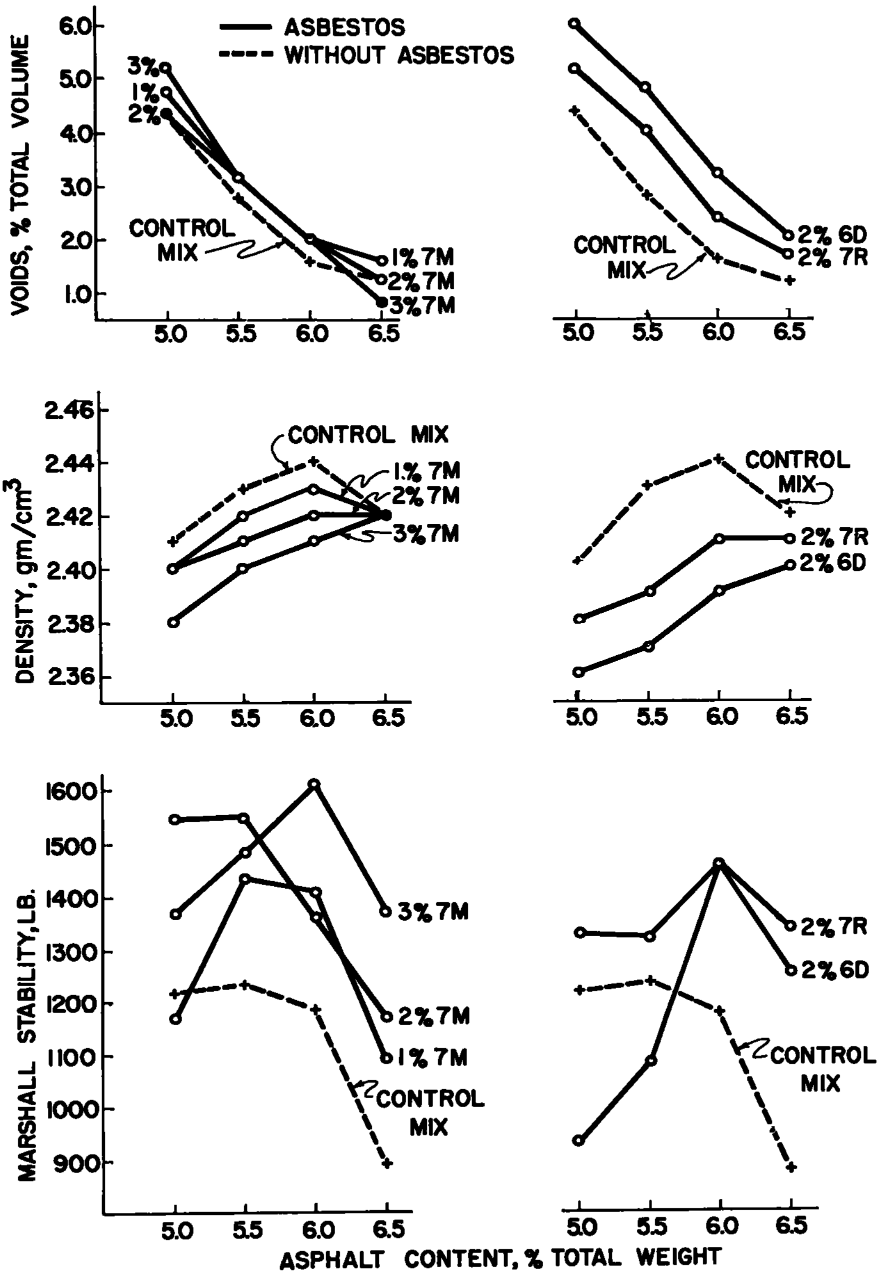
1. After curing at room temperature for 48 hr, the dry weight and weight under water after 1-min immersion were taken. The difference in weight was taken as the bulk volume.
2. The density was determined by dividing the dry weight by the bulk weight.
3. The absolute volume of each sample was determined by summing up the product of the weight percentage multiplied by the dry weight of the sample divided by the specific gravity for each constituent in the mix:

$$\Sigma \left(\frac{\text{wt } \%}{\text{Sp. G}} \text{ x dry sample wt} \right)$$

4. The approximate void content in percent by volume was calculated by subtracting the absolute volume from the bulk volume and dividing by the bulk volume.

Appendix D

MARSHALL TEST DATA BY THE CHICAGO TESTING LAB.
ON ASPHALTIC CONCRETE



Determination of Age Hardening Tendencies And Water Susceptibility of Paving Asphalt By the Sonic Method

FRANKLIN P. ABBOTT and WILLIS G. CRAIG, The Lubrizol Corp., Cleveland, Ohio

This paper presents data on (a) progressive hardening and embrittlement of the asphalt cement with aging; (b) loss of adhesion of the asphalt cement to the aggregate with the resultant lower compressive strengths due to water displacement of the asphalt binder at the asphalt-aggregate interface; (c) progressive loss of water resistance of the asphalt cement as the asphalt hardens; and (d) progressive loss of the ability of the asphalt binder to re-adhere to the aggregate after displacement by water.

The action of effective anti-stripping additives in ameliorating these causes of road deterioration is shown to be in the direction of reducing the magnitude of the last three of these deteriorative factors.

● THE DETERIORATING effect of water on bituminous paving mixes has long been recognized as a serious cause of service failure, but there is still a wide divergence of opinion regarding the causes of this deterioration and, more particularly, the means by which this deterioration can be evaluated in accelerated tests. In a previous paper (1), W.G. Craig pointed out the major variables of static water immersion tests and has shown that the asphalt is as great a variant as the aggregate. It was also shown that the utility of adhesion improving additives is limited by their temperature stability and/or their degree of reactivity with the asphalt under test. The test methods used to demonstrate these variables were largely based on the Massachusetts State Highway Department stripping test (2) which includes 24-hr elevated temperature (350 F) storage of the asphalt-additive blend prior to aggregate coating, curing and immersion in water.

EFFECT OF PROLONGED HEATED STORAGE OF ASPHALT-ADDITIVE BLENDS

At the time this earlier paper was presented, a question was raised regarding the validity of the method in determining the long-term effectiveness of additives at road temperatures by means of this short-term, high temperature test. Because of this question, a long-term, moderate temperature test was started. The test procedure was as follows:

1. The four additives evaluated in the previous paper were blended with two geographically different paving asphalt cements at 0.5 percent by weight in the manner prescribed by the Massachusetts stripping test (2). This level of additive treatment was sufficient to allow all four additives to pass the stripping test without a heating period prior to coating and immersion.
2. Immediately after blending, a portion of each blend was tested according to the procedure of the Massachusetts stripping test.
3. The balance of eight samples was placed in tightly sealed cans and stored in a constant temperature oven held at 140 F.
4. After 1, 3, 6, and 12 months of oven storage, the samples were removed and the stripping tests repeated. The results of this test are shown in Figure 1.

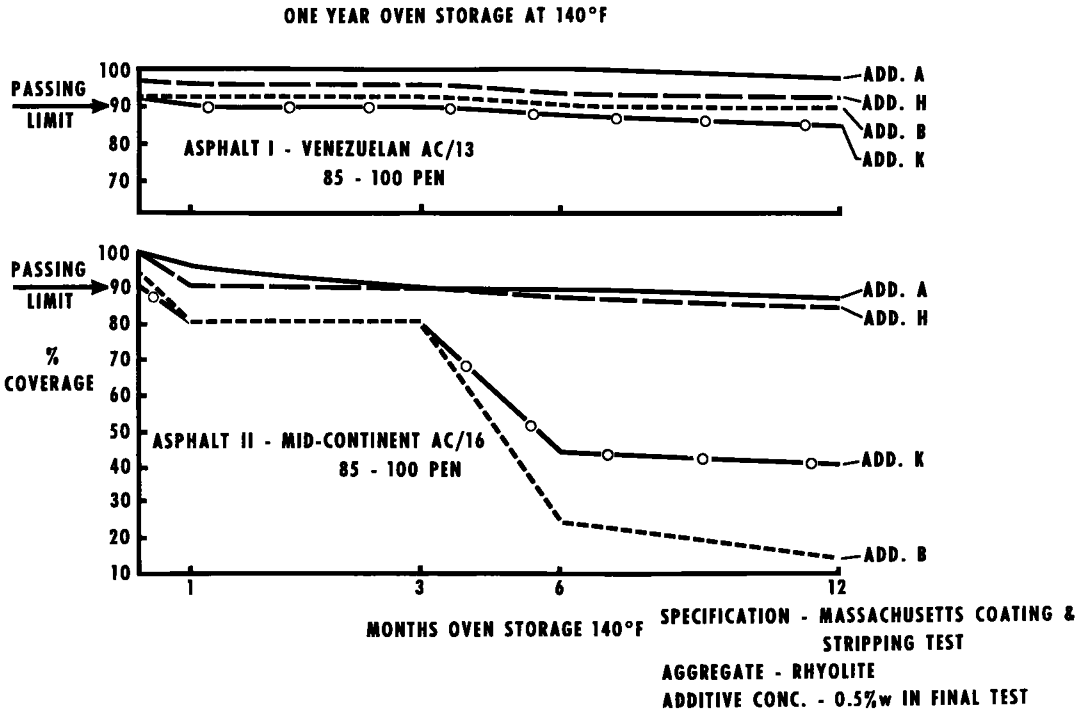


Figure 1. Variation of additive effectiveness in two geographically different asphalts.

Asphalt I has excellent susceptibility to treatment with anti-stripping agents. The data show that in this long-term, moderate temperature test, Additive A is most effective, Additive H somewhat less, Additive B is borderline, and Additive K fails between three and six months. This is exactly the same relative performance of these four additives in this asphalt as was obtained in the standard Massachusetts, 350 F, 24-hr heat test. The Massachusetts test, however, caused a greater degree of deterioration in Additives H, B, and K than is shown here.

Asphalt II is much more difficult to treat. Additives A and H were still performing at close to specification levels at the end of the year, while B and K were well below specification level after only one month of oven exposure. This is also in line with our previous results obtained from the Massachusetts heat stability test with this asphalt and these additives, although again, the regular Massachusetts test is somewhat more severe than the test reported here.

The data clearly indicate that the performance of adhesion improving additives is markedly affected by the asphalts with which they are blended, and that with unstable additives the stripping resistance may be lost during summer road exposure as well as during refinery handling and storage at elevated temperatures, as is demonstrated by the conditions of the standard Massachusetts test.

EVALUATION OF STRIPPING CHARACTERISTICS BY MEANS OF THE SONIC METHOD

Inasmuch as none of the static methods of determining stripping of cutbacks from aggregates uses the wide gradation of aggregate found in a typical paving mix, it was decided to determine if the sonic method (3) was applicable to open gradation aggregate-asphalt test beams prepared with cutback asphalt.

Test beams, 12 by 2½ by 2½ in., were prepared by vibratory compaction of an aggregate mix containing 6.5 percent by weight binder as either RC-2 or MC-3 cutbacks.

The mix design and method of compaction resulted in test samples with substantially uniform voids content. This open mix design was chosen to allow rapid water penetration into the test beams. (See Appendix B for details of mix design and method of compaction.)

Freshly prepared beams were much too plastic to yield reliable data by the sonic test method. Attempts to cure the test specimens at 140 F resulted in slumping of the specimens at edges and ends. Because of this, test specimens were allowed to cure at room temperature in the laboratory and their loss of weight as the solvent evaporated was closely followed by periodic weighing. After each weighing the fundamental frequency of the test beam was determined ana the sonic modulus calculated. The results are shown in Figure 2 where the sonic modulus and the solvent weight loss of the test beams are plotted against curing time.

The loss of solvent from both the RC and MC bars was very slow. After 12 weeks the rate of solvent loss dropped to practically zero, although there was a very slow increase in the sonic modulus of the samples between 12 and 21 weeks. The sonic modulus at the end of this period was still well below that of a fresh sample prepared from molten asphalt, which served as a baseline, and it was decided to accelerate the cure by heating to 140 F inasmuch as the samples now appeared to be firm enough to withstand this temperature without slumping. One week of exposure at 140 F caused enough additional solvent to evaporate from the RC specimens to bring their sonic moduli into the range of similar beams prepared from the same asphalt by hot-mixing, although weighings showed that the aged specimens still retained approximately 35 percent of their original solvent content.

The MC series, as would be expected, did not respond as readily to the forced drying. After one week at 140 F about 50 percent of the original solvent content still remained in these beams, and their sonic moduli were not as high as those of the RC specimens.

The data in Figure 2 clearly show that even in these specimens of small cross-section which were open to the air on three sides, the loss of solvent was very slow. In a road section of comparable depth ($2\frac{1}{2}$ in.), evaporation could take place only at the surface and therefore solvent escape would be even slower.

The increase in sonic modulus in comparison to the solvent evaporation rate is interesting. Apparently there is a marked increase in the "body" of the asphalt which is not entirely attributible to solvent loss, inasmuch as the RC beams with 35 percent

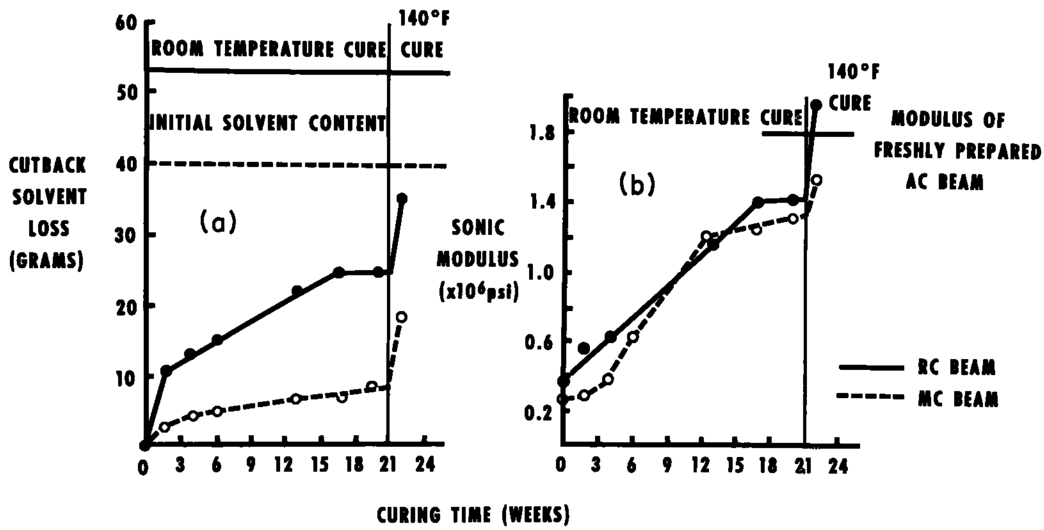


Figure 2. Changes during curing of RC and MC cutback beams showing (A) rate of cutback solvent loss and (B) sonic modulus increase.

of their initial solvent content remaining were as resilient as a freshly prepared AC beam. This can only mean that, during the long drying period, changes in the asphalt have occurred which have hardened it to such an extent that, although the asphalt is partially solvated, these beams have reached the elasticity of a standard hot-mix specimen.

To determine the effect of water and adhesion-improving additives on the sonic moduli, the cured RC and MC beams were immersed in water at 140 F, as recommended by Goetz (4), for 15 days and their decrease in sonic modulus was followed.

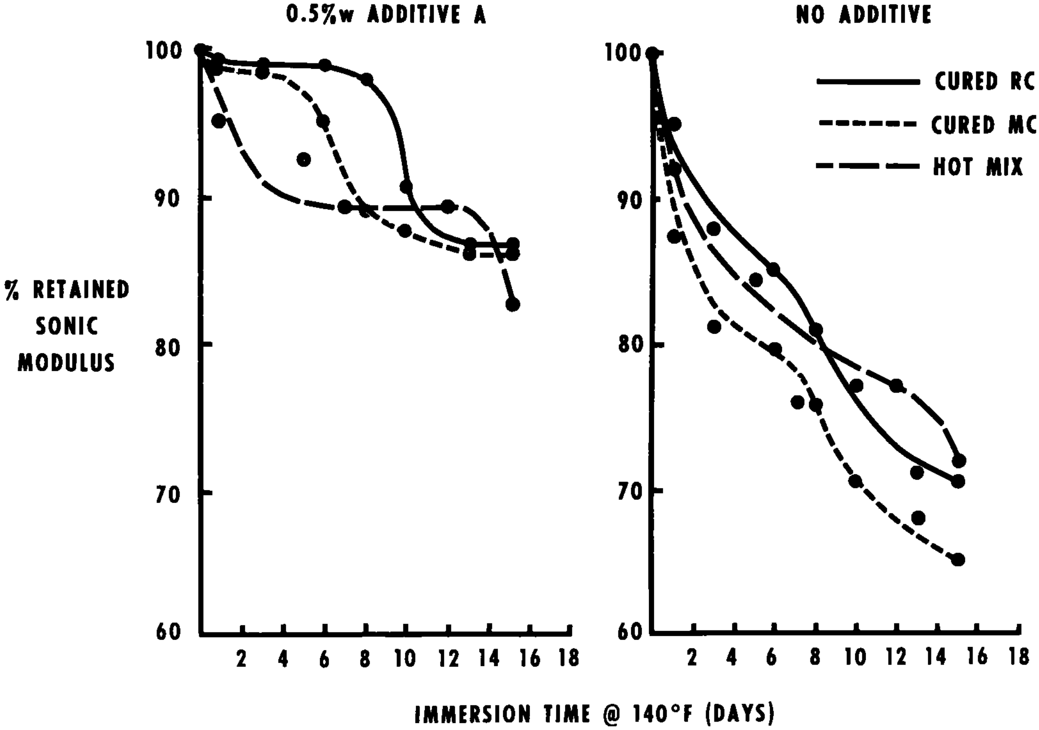


Figure 3. Effect of additive on sonic modulus during water immersion of cutback and hot mix beams.

An additional set of test specimens was prepared from the same mix design by hot-mixing with the same asphalt cement as was used in preparing the cutback samples, and immersed and tested in the same manner. The results are shown in Figure 3.

From Figure 3 it can be seen that there is a marked reduction in sonic modulus at the end of the first day of immersion, followed by a somewhat slower decline throughout the test. In the case of the untreated test samples, the loss of modulus on extended water immersion is very pronounced, and is especially severe with the untreated MC specimens which are the most plastic of the group. The left-hand group of curves in this illustration demonstrates the effectiveness of this particular additive (Additive A) in retarding the change in modulus during water immersion.

Figure 3 shows that the water resistance of non-additive cutback paving mixes and of similar paving mixes prepared from asphalt cement was substantially the same after the cutback mix had reached the same elasticity (as shown by equivalent sonic moduli) as the AC. However, the time required for an RC mix to reach this state in actual service is very long, and it is thus susceptible to severe stripping during this period.

Effective additive treatment, however, will protect against this deterioration and, in the cutback mixes, will markedly improve their stripping resistance over that which can be obtained with a treated AC. Just why the additive should be more effective in protecting the samples prepared from cutback asphalt than it is with the AC samples is not known. It may well be that this increase in effectiveness is caused by the lower viscosity of the cutback asphalt at the time of mixing of the samples, as the degree of effectiveness of the additive is greatest for the lower viscosity RC cutback sample. The lower viscosity of the cutback binders at the time of mixing, together with the surface activity of the additive, may enable them to wet out better on the aggregate and thus provide more complete and uniform coverage than can be obtained with asphalt cement by hot-mixing.

EFFECT OF AGING, WATER IMMERSION, AND DRYING ON COMPACTED BITUMINOUS MIXES

Since it had been shown by means of the Massachusetts test that asphalt-additive blends were deteriorated by long-term storage at 140 F, and that the sonic method was effective in showing differences between untreated and additive-containing asphalt during water immersion of compacted aggregate paving mixtures, the next step was to combine the two in a single test which would introduce the effects of both long-term aging at 140 F and of aggregate gradation on water resistance.

A large number of open-mix sonic test beams, prepared in the same manner as the foregoing, were made from an untreated 85-100 penetration Lloydminster asphalt and from the same asphalt containing 0.5 percent by weight of four commercial, heat stable, adhesion-improving additives. All freshly prepared beams were chilled to 40 F and their sonic values determined. One set was immersed in water immediately and the balance of the test beams were placed in a large, forced draft oven maintained at 140 F. At intervals of 1, 3, and 6 months, beams of each mixture were withdrawn and their fundamental frequencies at 40 F redetermined.

After each particular oven aging cycle, beams of each mixture were immersed in 140 F water for 15 days and the decrease in their sonic moduli was followed. At the end of the 15-day immersion period the test beams were removed from the water and allowed to dry at room temperature. The increase in sonic modulus as the beams dried was followed periodically for 20 days. After this time the beams had returned to nearly their original weight, indicating essentially complete drying.

On completion of sonic evaluation after each aging, immersion, and drying cycle, beams of each mixture were carefully sawed into two, 5-in. long sections, the ends capped with plaster-of-Paris, and the two samples crushed in unconfined compression by means of a Marshall Compression Tester, modified to operate 0.05 in. per min per inch of beam height in accordance with ASTM Specifications (Appendix C). The asphalt from the crushed specimens was recovered by the ASTM D-402-55 method for penetration testing.

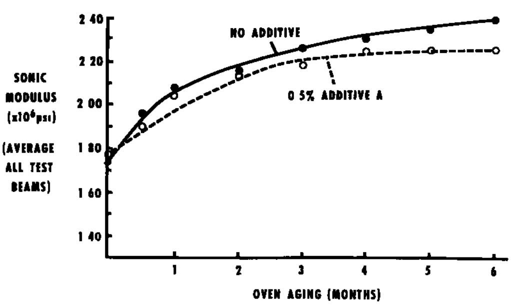


Figure 4. Sonic modulus changes occurring in Lloydminster AC-rhyolite beams during aging at 140 F.

Figure 4 shows the effect of oven aging at 140 F on the sonic modulus of beams prepared from the 85-100 penetration Lloydminster asphalt and from the same asphalt treated with 0.5 percent of a commercial anti-stripping agent. Here it will be seen that there is a fairly rapid increase in the moduli during the first month, followed by a somewhat slower, relatively constant increase during the balance of the test.

Figure 5 shows the results of the aging, water immersion, and drying test for the untreated asphalt sample, and for the sample treated with 0.5 percent of Additive A,

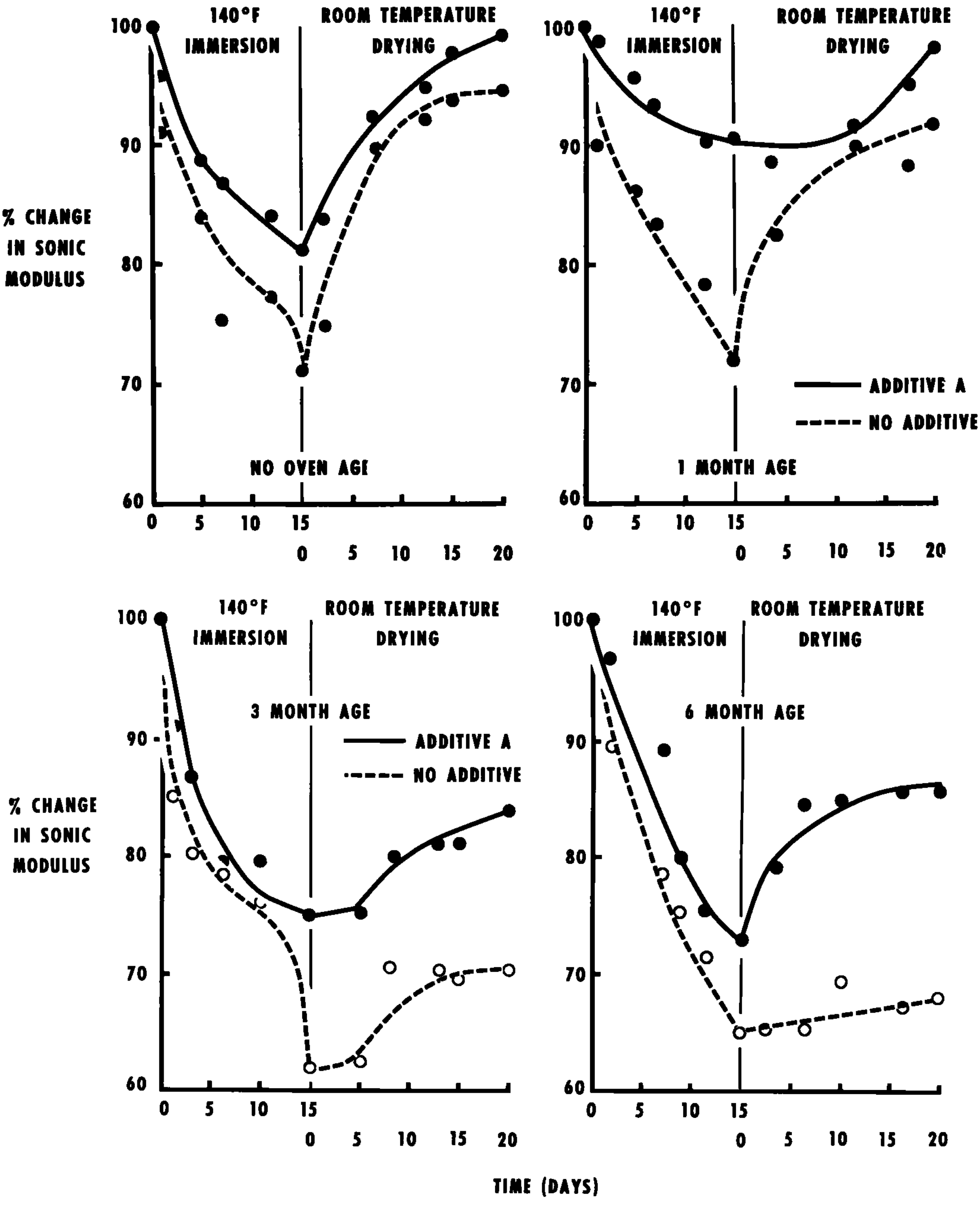


Figure 5. Sonic modulus changes during water immersion and drying of Lloydminster AC-rhyolite beams after various periods of oven aging.

which was the most effective of the four additives. Because oven aging caused the sonic moduli of the test beams to increase with time of exposure, samples removed from the oven after the various test times had different moduli at the time they entered the soaking and drying cycle. For this reason the data have been presented as the "percentage change" of modulus during this cycle. From these curves it can be seen that both the treated and untreated test beams became progressively more susceptible to changes in their sonic moduli during immersion as their test age increased. After the first month of aging at 140 F, the changes in sonic moduli during the soaking and drying cycle were not markedly different from the unaged beams, but after three and six months of aging, pronounced reductions in the retention of sonic moduli during water immersion are shown for both the treated and untreated test beams. The loss of sonic moduli of additive-treated beams is, however, noticeably less than that of the untreated specimens at all periods of aging.

The most interesting information gained from these tests is to be found in the recovery of sonic modulus on drying. After one month of aging, the recovery of the additive-treated beam was substantially 100 percent, and the untreated beam regained almost 95 percent of its original modulus. As aging progressed and the asphalt hardened, the "healing", or re-establishment of bond between asphalt and aggregate as the water evaporated, became less and less. The effect of the additive, while appreciable, was not enough to bring the moduli of the beams back to their original values. This is especially evident after six months of aging. At this time the recovery of sonic modulus of the untreated beam is only 65 percent of its original age-hardened value, whereas the beam containing Additive A showed recovery to 85 percent of its original value. Apparently, during the aging of the test beams, the asphalt has hardened enough so that it is no longer sufficiently plastic to re-establish its bond to the aggregate once it has been displaced by water.

The water immersion test data show little change in the stripping characteristics during the first month of aging, although there has been a rapid increase in sonic modulus during this period (Fig. 4). It is thought that this initial rapid increase in sonic modulus may be due to "structure formation" or gelling of the asphalt rather than to chemical changes due to oxidation and/or polymerization, and that actual age hardening is not apparent until after the first month, after which time this hardening continues at a constant rate depending on the susceptibility of the system.

Because the sonic moduli of the test beams are increasing with the test age of the specimens, it is logical to suspect that the compressive strength of the specimens is also increasing if the increase of sonic moduli is a function of the hardening of the asphalt. Figure 6 confirms this, as it will be noted that when the unconfined compressive strength of the test specimen is plotted against the oven aging time, the general shape of the curves obtained are similar to those of Figure 4. Therefore, a rea-

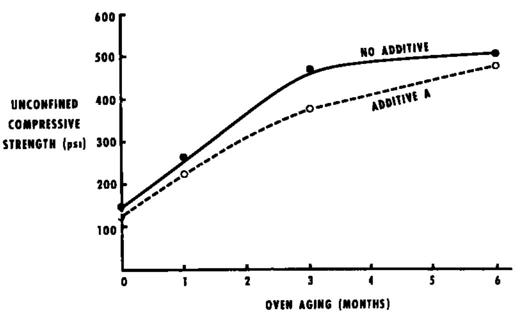


Figure 6. Changes in unconfined compressive strength of Lloydminster-rhyolite beams after various periods of oven aging at 140 F.

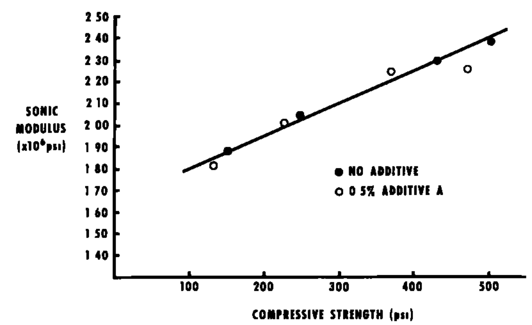


Figure 7. Sonic modulus and strength relationship during 6-month age hardening of Lloydminster AC-rhyolite beams.

sonably good correlation appears to exist between the unconfined compressive strength and the sonic moduli of these specimens. This correlation is confirmed by Figure 7, where sonic modulus is plotted against compressive strength. This gives a straight-line plot with only modest scatter of the data. The exact relationship of sonic modulus to unconfined compressive strength shown in this figure is, of course, applicable only to this particular mix design and asphalt.

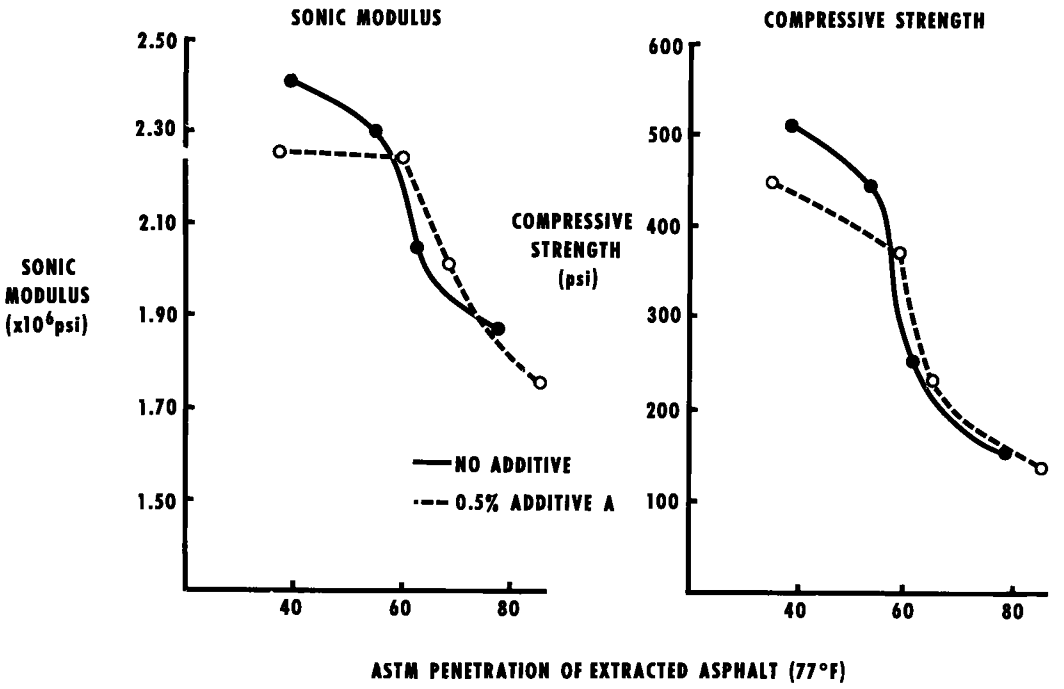


Figure 8. Relationship of penetration of extracted asphalt with sonic modulus and compressive strength of beams during oven aging cycle.

The relationship of the sonic modulus and the unconfined compressive strength of the test specimens to the penetration of the asphalt extracted from them, is shown in Figure 8. Starting from the lower right, each point on a curve is the value obtained at 0, 1, 3, and 6 months of oven aging, respectively. The curves show a uniform increase of both sonic modulus and strength, with decreasing penetration up to the three months' aging period when the asphalt has hardened from its original 78-87 penetration range to a 55-60 penetration range. After this point neither the sonic modulus nor the compressive strength change as rapidly as the penetration. This would seem to indicate that between 60 and 40 penetration, changes take place in the asphalt structure which markedly affect the ASTM penetration value without adding greatly to the elasticity or the strength of the test specimen as measured by these tests.

This anomaly can be explained by assuming that a high degree of "structure formation" or gelation occurs in the asphalt during the first few months of aging, and that this "structure," once formed, does not change appreciably as the asphalt hardens further. This "structure" could contribute appreciably both to the elasticity (sonic modulus) and the unconfined compressive strength of the specimen. It would be broken during the extraction procedure used to recover the asphalt for the penetration tests, and therefore the penetration values would not be a true indication of the rheological characteristics of the asphalt as it was when in situ during the sonic and compression tests.

It may well be, however, that when the asphalt reaches a hardness near 60 penetration, there is a reduction in the interfacial forces of adhesion between the asphalt and the aggregate which affects both the elasticity and the strength values of the specimen in these tests so that their increase is not proportional to the hardness of the asphalt. This supposition is reinforced by the data on water immersion of aged bars, where it was shown (Fig. 5) that the aged bars were more subject to the deterioration of sonic values on water immersion than were freshly prepared bars. This would indicate that the asphalt, during aging, gradually loses adhesion to the aggregate and is now more readily displaced from the surface of the aggregate by water penetration at the interface.

The data derived from test beams that were aged, immersed in water for 15 days, and then dried at room temperature for 20 days, are shown in Figure 9 (right). Freshly prepared beams regained a substantial portion of their original moduli and strength on drying, with the additive treated samples showing almost complete recovery. The difference between the fresh samples and those aged for one month is somewhat more pronounced than in the immersion test, and the benefit of the additive in promoting rehealing shows up strongly in the strength recovery. After three and six months, however, the "healing," particularly in regard to regaining strength lost during water immersion, is very minimal.

From these data it is apparent that the prolonged effect of water on aged bituminous concrete pavement is very deleterious, much more so than on a fresh pavement, and that once such a pavement is so exposed the damage is permanent.

CONCLUSIONS

From the data presented, the following conclusions can be drawn:

1. The long-term storage of asphalt-additive blends at 140 F (a temperature which often occurs in roads) confirms the results previously published (1) regarding the variability in results which can be obtained with a series of asphalt-additive blends. The previous work was based on an accelerated heat test with 24-hr storage of the blends at 350 F. Present data confirm the supposition that the same differences would be shown by long, low temperature storage as is obtained in the accelerated test. It also indicates that deterioration of the effectiveness of anti-stripping agents can occur over long periods in the road as well as at elevated temperature in asphalt processing, and shows that an accelerated heat stability test is desirable in specifying anti-strip additives which will be effective in the particular asphalt to be used for construction.

2. The sonic test method is not an effective tool for evaluating the stripping characteristics of cutback paving mixes because of the long time required for the sample to reach an elastic state by evaporation of solvent. The data show, however, that cold paving mixes prepared from RC cutback will eventually reach the same elastic state as freshly prepared AC mixes even though 35 percent of their initial solvent content still remains in the mix. This can be explained by the increasing hardness of the asphalt due to oxidation and/or polymerization during the long drying period.

   The water resistance of cold-mix and hot-mix pavements is substantially the same after the cold-mix paving has evaporated enough of its solvent to approach the viscosity of the hot mix. Prior to this time, cold-mix pavements are much more susceptible to stripping. MC cold-mix pavements require much longer cure times to approach the nature of AC paving than do RC mixes, and hence are susceptible to severe stripping for a longer time. Anti-stripping additives are very effective in preventing stripping of cold-mix pavements during their long period of curing, and once a cutback paving containing anti-strip additive has cured, it is more resistant to water deterioration than a hot-mix paving containing the same additive.

3. Aging compacted bituminous paving mixtures in a forced circulation hot air oven results in hardening of the paving. This hardening can be readily followed by the increase in the sonic moduli of the test specimens. The increase in sonic modulus parallels the increase in unconfined compressive strength. For a given mix design, and a given asphalt, a straight-line plot of sonic modulus versus compressive strength can be obtained.

4. Water susceptibility of hot-mix pavements can be readily determined by the sonic method and the effectiveness of adhesion promoting additives evaluated. Hardening of asphalts during aging results in lessened resistance to stripping, a phenomenon which cannot be demonstrated by the usual static stripping tests. In addition, the method enables an evaluation of the degree of re-adhesion of the asphalt to the aggregate on drying—a factor which may be of great importance to road durability.

5. Sonic modulus, unconfined compressive strength, and the ASTM penetration of the asphalt recovered from the test specimens are parallel functions up to a point. After about three months of oven aging, the penetration value of the asphalt changes more rapidly than either the compressive strength or the modulus. The more rapid

increase of strength and sonic modulus in relation to the decrease in ASTM penetration during the first three months of oven exposure is thought to be due, in part, to development of "structure" or gelation within the asphalt.

6. Measurement of the unconfined compressive strength of compacted test samples after aging and soaking, confirms the loss of adhesion shown by the decrease in sonic modulus. On very long soaking, the decrease in the compressive strength is greater than the decrease in sonic modulus.

7. Adhesion improving additives are effective in reducing the magnitude of change in both sonic modulus and in unconfined compressive strength during water immersion of fresh and aged paving samples. They are also effective in promoting the "healing" or regaining of these values as the sample dries.

8. A major cause of road deterioration lies in the hardening of the asphalt during aging and the additives become less effective as the age and hardness increases. In order to further improve the quality of asphaltic paving cements, additive utilization and efficiency, research should be directed toward devising means of retarding the hardening of the asphalt during exposure.

REFERENCES

1. Craig, W.G., "Variables of the Static Asphalt Stripping Test." The Lubrizol Corp., Cleveland, Ohio.
2. "Specification for Anti-Strip Additive." Commonwealth of Massachusetts, Dept. of Pub. Works (March 1955).
3. Goetz, W.H., "Sonic Testing of Bituminous Mixtures." AAPT Proc., Vol. 25 (1955).
4. Andersland, O.B., and Goetz, W.H., "Sonic Test for Evaluation of Stripping Resistance in Compacted Bituminous Mixtures." AAPT Proc., Vol. 25 (1956).
5. Bawa, K.S., "Sonic Studies of Bituminous Mixtures." M.S. Thesis, Purdue Univ. (Jan. 1953).
6. Yong, R.N.U., "The Physical Significance of Sonic Test on Bituminous Mixtures." M.S. Thesis, Purdue Univ. (June 1954).

Appendix A

MODIFIED[1] MASSACHUSETTS HIGHWAY SPECIFICATION

Test Procedure

1. Additives tested were blended into the two representative asphalt cements (85-100 pen.) at a concentration of 0.5 percent w at a temperature of 200-250 F and stored for a period of one year at 140 F in tightly sealed ½ pint paint cans. After 1, 3, 6, and 12 months of storage, samples were removed for anti-strip evaluation.

2. An RC-2 cutback was prepared by diluting 75 parts of each sample with 25 parts of Varnish Makers and Painters Naphtha.

3. Rhyolite stone was graded so that 100 percent passes a ⅜-in. sieve and is retained on a No. 4 sieve. The aggregate was washed in distilled (ph 6-7) water and oven dried at 270-300 F.

Coating and Stripping Test

One hundred grams of washed and dried Rhyolite aggregate was wetted with two grams distilled water and coated with six grams of the prepared cutback by thoroughly mixing for 5 min. The coated aggregate was then air-cured for 1 hr at room temperature. Initial coverage was noted.

The coated aggregate was then immersed in distilled water at room temperature and, after 24 hr, inspected for percent coverage. (At least 90 percent coverage after the immersion period is necessary to pass this test.)

[1]Extended heat storage at 140 F.

ASPHALT SPECIFICATIONS

Asphalt Type:	Lloydminster	Asphalt I	Asphalt II
Specific gravity at 77 F	1.0351	1.019[a]	0.9945
Softening point (R and B)	115 F	113 F	120 F
Pen. at 77 F (100g, 5 sec)	97	92	94
Saybolt furol vis, 210 F	1220	-	-
Ductility, 5cm/min at 77 F	150+	-	150+
Loss on htg. 325 F, 50g, 5 hr	0.07%	0.7%	0.005
Pen. at 77 F on residue from loss on heating test	88	87	95
Flash, C.O.C., deg F	535	535	595
% CCl_4 soluble	99.92	-	99.85
% CS_2 soluble	99.94	-	99.90
% 86 deg Baume naph insoluble	23.1	-	18.00
% Sulphur	5.2	-	-
Oliensis spot test	Neg	Neg	Neg

[a]Specific gravity at 60 F.

Appendix B

MIX DESIGN

Standard open-gradation aggregate mix was used in the preparation of all sonic beams. This design was so chosen to produce specimens of relatively high voidage to enhance both permeability of air during oven aging, thereby promoting hardening, and to accelerate the effect of water to promote stripping. Mixed aggregates—rhyolite as coarse, Ohio bank sand as fine, and either limestone dust or hydrated lime as filler—and 6.5% w binder (based on total weight) composed the mix design.

Rhyolite aggregate (as received) was washed thoroughly with tap water to remove dust and finally with distilled water (pH 6-7), then dried overnight in a forced air oven at 300-325 F. Ohio bank sand and inorganic fillers were used as received, omitting the washing step, but were oven dried prior to screening. The aggregates were then individually sieved and the various sized components stored for batch blending.

The asphalt for test was received in 5-gal. pails, and to appropriately sample, the pails and contents were carefully heated to 200-225 F in a forced air oven with their lids intact so as to exclude excess air. Samples of asphalt were transferred to one-pint cans, in an amount sufficient to prepare two test beams. Additive, at a concentration of 0.5% w based on AC, was mixed with the samples of asphalt at 175-200 C for 1-2 min. Blends were then set aside in closed cans until used in beam preparation.

Cutback asphalt blends (RC-2 and MC-3 types) were prepared in a similar manner, except that pint samples of cutback were first prepared using either V.M. and P. Naphtha (RC solvent) or Bronoco 529 (MC solvent), and then mixed with additive at 120-140 F.

PREPARATION OF SONIC TEST BEAMS PREPARED FROM HOT MIXES

All sonic beams prepared in this program were compacted from individual 2,500 gram batches of bituminous mix. The blended aggregates and container of prepared asphalt were brought to a temperature of 275 ± 5 F in a forced air oven prior to mixing together. A stainless steel mixing bowl, trowel, and beam mold were simultaneously heated to this temperature. After heating, the nescessary amount (6.5% w) of asphalt was added to the hot aggregate mix and stirred vigorously by hand for approximately 2 min or until a visually homogeneous mix was obtained. Temperature of the mix after this time was 255-260 F. The hot mix was then immediately troweled into the preheated, specially-constructed portable beam mold, tamped slightly to level

SIEVE ANALYSIS OF AGGREGATES

% Passing

Sieve Size:	1/2	3/8	# 4	# 8	# 50	# 100	# 200
Rhyolite	100	99	70	5	0	0	0
Ohio Sand	100	100	95	81	20	3.5	1.5
Limestone Dust	100	100	100	100	100	100	85
Hydrated Lime	100	100	100	100	100	100	100
Median of Spec	100	95	70	32	7.5	4.5	3.5

ADJUSTED MIX DESIGN (BLEND PROPORTIONS)
(For Hot Mix Beams*)

% Passing

Sieve Size		1/2	3/8	# 4	# 8	# 50	# 100	# 200
Rhyolite	64.5%	64.5	64.5	45.3	3.2	-	-	-
Ohio Sand	32.0%	32.0	32.0	31.0	27.0	6.4	1.0	0.5
Limestone Dust	3.5%	3.5	3.5	3.5	3.5	3.5	3.5	3.0
Total		100	100	79.8	33.7	9.9	4.5	3.5
Median of Spec		100	95	70	32	7.5	4.5	3.5

BATCH WEIGHTS (% OF TOTAL MIX)

Hot Mix

Rhyolite (60%) - 1500g
Ohio Sand (30%) - 750g
Limestone Dust (3.5%) - 87.5g
Asphalt (6.5%) - 162.5g

Cold Mix

Rhyolite (60%) - 1500g
Ohio Sand (30%) - 750g
Limestone Dust (1.5%) - 37.5g
Hydrated Lime (2.0%) - 50.0g
AC (as Cutback) (6.5%) - 162.5g

Total
2500g

* Adjusted mix design for cutback beams uses 1.5%w Limestone Dust and 2.0%w Hydrated Lime.

RHYOLITE-LLOYDMINSTER RC-2 AND MC-3 BEAMS

Physical Properties

Beam No.		Weight After Cure Gms	Weight After Cure Lbs	Overall Solv Loss Gms	Overall Solv Loss (%)	Theo. Solv. Content (gms) Before Cure	Beam Dimensions (Length = 12") Breadth Inch	Beam Dimensions (Length = 12") Thickness Inch	Density*	%v Solids	%v Voids	Theo. Vol. Air Voids cc
RC-2	(1)	2468	5.46	34	(64.4)	53.3	2.45	2.53	2.025	86.8	13.2	161
	(2)	2471	5.46	36	(68.2)	53.5	2.49	2.53	1.995	85.5	14.5	180
MC-3	(3)	2502	5.54	19	(47.3)	40.2	2.47	2.52	2.045	87.8	12.2	149
	(4)	2501	5.54	23	(57.2)	40.2	2.49	2.52	2.027	86.8	13.2	163

* Max theo. Specific Gravity of Mix (after Cure): RC-2 Beams, 2.333
MC-3 Beams, 2.33 (3), 2.336 (4)

RHYOLITE-LLOYDMINSTER AC BEAMS

Physical Properties

(1) Beams Containing No Additive

Beam No.	Oven Dry Wt. Gms	Oven Dry Wt. Lbs	Dimensions (Length = 12") Breadth Inch	Dimensions (Length = 12") Thickness Inch	Vol. cc	Density*	%v Solids	%v Voids	Theo.Vol, Air Voids (cc)
1	2479	5.46	2.55	2.52	1263	1.962	82.7	17.3	218
2	2478	5.46	2.54	2.52	1259	1.969	83.0	17.0	214
3	2477	5.46	2.54	2.52	1259	1.968	83.0	17.0	214
4	2475	5.46	2.55	2.53	1268	1.951	82.3	17.7	224
5									
6	2482	5.47	2.55	2.53	1268	1.957	82.5	17.5	222
7	2482	5.47	2.53	2.53	1258	1.972	83.1	16.9	213
8									
9	2482	5.47	2.54	2.52	1258	1.972	83.1	16.9	213
10	2480	5.47	2.53	2.52	1253	1.978	83.4	16.6	208
11	2477	5.46	2.53	2.52	1253	1.976	83.3	16.7	209
12	2482	5.47	2.55	2.52	1263	1.964	82.8	17.2	217

* Max. Theo. Sp. Gr. of Mix = 2.372 (based on densities of aggregates and binder)

Physical Properties

(2) Beams Containing Additive "A"

Beam No.	Oven Dry Wt. Gms	Oven Dry Wt. Lbs	Dimensions (Length = 12") Breadth Inch	Dimensions (Length = 12") Thickness Inch	Vol. cc	Density*	%v Solids	%v Voids	Theo.Vol, Air Voids (cc)
13	2480	5.47	2.54	2.53	1263	1.963	82.8	17.2	217
14	2484	5.47	2.55	2.52	1263	1.966	82.9	17.1	216
15	2483	5.47	2.54	2.53	1263	1.965	82.8	17.2	217
16	2480	5.46	2.54	2.53	1263	1.963	82.8	17.2	217
17	2479	5.46	2.54	2.52	1258	1.970	83.1	16.9	212
18									
19	2492	5.49	2.54	2.52	1258	1.980	83.5	16.5	207
20	2486	5.48	2.54	2.53	1263	1.968	83.0	17.0	214
21									
22	2488	5.48	2.55	2.53	1268	1.962	82.7	17.3	219
23	2488	5.48	2.55	2.53	1268	1.962	82.7	17.3	219
24	2485	5.48	2.54	2.53	1263	1.967	82.9	17.1	216

* Max. Theo. Sp. Gr. of Mix = 2.372

the mix, and compacted by means of an air-driven, hand vibrator containing a steel base which fit loosely into the mold. The temperature immediately prior to compaction was 225-235 F. Control of degree of compaction of the mixes was easily afforded by applying vibrational force until the mix level reached a measured depth in the mold. Specimens compacted in this manner had a variable breadth dimension of 2.50-2.55 in., the thickness and length dimensions being fixed at 2.5 in. and 12 in., respectively.

After compaction, which normally took about 3 min, the beam mold was cooled by a stream of cold water, disassembled immediately, and the beam removed and marked with suitable identification. Beams were then oven-dried for 24 hr at 140 F and their breadth and thickness dimensions determined prior to sonic testing.

PREPARATION OF SONIC BEAMS FROM MIXES CONTAINING CUTBACK ASPHALT

Cold-mix bituminous beams containing RC and MC cutback asphalt were prepared using the batch method described in the section on "Preparation of Sonic Test Beams Prepared from Hot Mixes" with the exception that preheating of the aggregates and cutback was at a lower temperature to prevent solvent loss during the over-all operation. Preheating of the mix ingredients and implements, including beam mold, did not exceed 175 F and the temperature at time of compaction was between 140-150 F. After compaction, beams were removed from the mold without water-cooling applied, and carefully set aside to cure at room temperature after proper measurement to determine breadth and thickness dimensions.

Appendix C

DETAILS OF THE SONIC METHOD

Determination of the Fundamental Frequency of Vibration

The first step in calculating the sonic modulus of the beam was determining its fundamental frequency of vibration at one specific temperature, 40 F. This was accomplished by chilling each beam to 40 ± 1 F in either a melting ice bath for specimens undergoing water immersion or in a cold chest when dry beams necessary for oven aging were desired. At least 2 hr chilling time was required to reduce beams to the desired 40 F.

The sonic apparatus used in this study consisted of a Type 208-B Dumont cathoderay oscillograph, a Model 655, Jackson audio oscillator, a modified ST-104 Jensen Speaker used as a driver, and a VM-1 Brush Vibromike used as a signal pick-up. For obtaining frequencies of a beam at 40 F, it was placed horizontally on the driver stand with one side in contact with the driver, and with the pick-up placed on the approximate center of the beam, the output frequency of the oscillator was varied until a peak of maximum amplitude was registered on the oscilloscope. The frequency at which this condition exists is denoted as the fundamental frequency.

To insure that the frequency determined was the true fundamental frequency, a quick check was made for existence of nodal points. These nodes should be located a distance of 0.224 L from either end of the beam with length L, vibrating transversely in a free-free condition. Nodal point check was made by feeding the oscillator signal directly to the X-plates of the oscilloscope as well as to the driver. The signal detected by the pick-up was fed to the Y-plates. When these signals were of the same frequency and in phase, a Lissajous circle was seen on the tube screen. At the point of maximum trace height, zero amplitude of vibration was shown on the Y-plates after moving the pick-up to the theoretical location of the nodal points. When the pick-up was moved to either left or right of the nodal point, the Lissajous circle showed inclinations in opposite directions indicating reversal of signal phasing.

SONIC EVALUATION OF HOT-MIX AND CUTBACK BEAMS

After determination of physical properties of hot-mix beams, those beams which were chosen to undergo an oven-age cycle were placed on a flat steel tray containing perforations to allow circulation of air and this set-up put into a forced draft oven regulated at 140 ± 2 F.

Periodically, beams were removed from the oven, chilled to 40 F in a cold chest, and their fundamental frequencies obtained and recorded. Beams subjected to oven storage were tested in this manner for periods of one, three and six months.

After completion of each aging cycle, the appropriate set of beams (usually two of each asphalt blend composition) underwent a water immersion cycle in a constant temperature water bath at 140 ± 2 F for a 15-day period. Frequencies of vibration were again obtained after intervals by removing beams from the water bath and placing them in a melting ice bath adjusted at 40 F for a 2-hr period prior to frequency determination. A 20-day drying cycle at room temperature followed, with similar periodic frequency measurement.

For cutback specimens, frequencies were similarly determined during a 5½ month cure period. This served to follow transition of beams from a plastic to a progressively more elastic state. Solvent loss was also followed. Approximately five months' cure was conducted at room temperature, after which time solvent loss and frequency of vibration ceased to change. Inasmuch as both RC and MC beams possessed plastic character after this time, it was decided to hasten solvent loss by curing at 140 F.

After one week of additional curing at 140 F, both RC and MC beams lost considerably more solvent and showed greater elastic properties (rise of frequency values).

These "cured" beams then underwent a 15-day water immersion cycle similar to hot-mix specimens.

Calculation of Sonic Modulus of Elasticity

The following equation, derived by Bawa (5) and Yong (6), was used in the computation of the modulus of elasticity:

$$E = CWn^2$$

$$C = 0.00323\ l^3/bt^3$$

in which E = Young's Modulus of Elasticity in psi;

C = a constant dependent on beam dimensions in inches;

W = weight of each beam in pounds;

n = fundamental frequency in cps at 40 F;

l = length of beam (inch);

b = breadth of beam (inch); and

t = thickness of beam (inch).

Sample Calculation of Sonic Modulus

Example cited herein used data obtained from Sonic beam No. 10.

Beam weight	5.47 lb
Breadth (width)	2.53 in.
Thickness	2.52 in.
Length	12.0 in.
Fundamental frequency (n)	1460 cps

$$C = 0.00323 \times \frac{(12)^3}{2.53 \times (2.52)^3} = 0.1379$$

$$E = CWn^2$$

$$E = 0.137 \times 5.47 \text{ lb} \times (1460)^2$$

$$E = 1.61 \times 10^6 \text{ psi}$$

Reliability of Fundamental Frequency Measurements and Sonic Modulus Data

The major factor governing the confidence limits of the reported fundamental frequency values is the temperature of the beams at the time of measurement. Because 40 F is the point determined by Goetz (3) at which elastic properties of asphalt beams exist, it is very important that this temperature does not significantly vary during the time required to measure the frequency. This depends wholly on technique of the experimenter, and with a little practice reasonably accurate frequencies can be obtained within 30 to 45 sec. Throughout this program, care has been taken to carefully regulate the temperature of ice baths and cold boxes to 40 ± 1 F and allow beams to reach an equilibrium before testing. Thermocouples were imbedded in the beams to determine the length of time required to reach 40 F. It was found that 1.75 to 2.0 hr cooling was necessary to reduce a beam from 140 F to 40 F, and approximately 0.75 hr to cool from room temperature to 40 F.

Reproducibility of frequency measurements for any given set of beams was very good considering minor differences in size of the beams, a normal variation inherent in the method of vibratory compaction.

For the most part, frequency data obtained was reliable to within about 1.5 percent. It was found that occasional beams having identical frequencies had slightly different sonic moduli. This is caused by the slight variations in the breadth and thickness dimensions which affect the constant, C, in the calculation of sonic modulus. Apparently, there is a limit in the accuracy of the instruments or method itself to depict the differences in frequencies of beams of slightly different size. Variation in beam weights of from 5.45 to 5.48 lb does not significantly change the sonic modulus at a given frequency and is therefore not a factor affecting the accuracy of test data.

Determination of Unconfined Compressive Strength of Sonic Beams

After completion of a particular cycle (aging, immersion, drying) of sonic testing, beams were sawed into two pieces, each 5 in. in length, and a cap of plaster-of-Paris, approximately 1/16 in. thick carefully applied to each end to form flat, parallel bearing surfaces. Unconfined strength of each piece was then determined according to ASTM Procedure Des. D-1074-55T (77 F, rate of vertical deformation, 0.05 in./min/ in. of specimen height). A motor-driven Marshall Stability Testing Machine with variable speed gear reducer installed, operated in accordance with ASTM rate, was used to determine strengths reported. Variation of strength between the two sections of the beams did not vary by more than 5 psi units, and an average of the two values was reported for each beam.

Extraction of Asphalt From Sonic Beams for ASTM Penetration Determination

Immediately after strength data were obtained, one section of each beam, with plaster caps removed, was immersed in water-white commercial xylol, and after 2 hr the resulting xylol-asphalt solution decanted from the aggregates and filtered to remove included fines. The solution was then stripped of xylol and the asphalt recovered according to ASTM Des. D-402-55. Approximately 60 grams of recovered asphalt was obtained, and penetration (ASTM Des. D-5-52) at two temperatures, 40 F and 77 F, determined. Penetration values reported herein are those determined at 77 F.

Rheology of Bitumens and the Parallel Plate Microviscometer

PAUL R. WOOD and HERMAN C. MILLER, Naugatuck Chemical Division, United States Rubber Co., Naugatuck, Conn.

The viscoelastic behavior of bitumens varies with film thickness. Therefore, for correlation of laboratory testing with field performance in bituminous pavements, it is important to match the geometry of testing to geometry of end use. The parallel plate microviscometer is well suited for this work, but the total displacement suggested by previous authors for most paving asphalts is too low.

Rotating parallel plates have been used to extend the useful range of the microviscometer.

● IT was introduced to the bituminous field in 1954 by Labout and Van Oort (2, 4, 5) Griffin et al. (1) simplified the thin film preparation.

The Naugatuck Chemical Division's work began before a commercial instrument was available so the Division designed and built its own. Inasmuch as the heart of the instrument, the glass plate assembly, is identical with that of previous authors, the instrument is described only as an example of an inexpensive unit which is both versatile and precise.

The parallel plate microviscometer is an important contribution to the bituminous field and especially to the paving field, not only because it is a fundamental instrument, but because it is perhaps the only instrument in which thin films, duplicating the binder thickness in a pavement, can be studied. In bituminous concrete pavements, the film thickness of the asphalt binder is of the order of 5-10 microns (3). This is vastly different from the film thickness encountered in most instruments used for viscosity measurement.

It has been reported by the previous authors that the physical behavior of bitumens does not change with film thickness which would make the latter advantage of the microviscometer unimportant. However, the authors have found that viscoelastic behavior does change with film thickness and herein lies the real advantage of the microviscometer. In addition, the authors have found that for most paving asphalts, the suggested displacement of 200 microns (1) of the upper plate for each shearing stress is insufficient.

EXPERIMENTAL DETAILS

Apparatus

The microviscometer is shown in Figure 1. Movement of the upper plate is followed with a microscope, 600 X, focused on a stage micrometer 2 mm in length, graduated in units of 0.01 mm. The level of the water bath is adjusted to be above the surface of the top plate and below the string.

Film Preparation

The film preparation step which has already been described by previous authors requires considerable practice, but is made easier by using the film preparation unit shown in Figure 2. To prepare a uniform film, the hot glass plates-binder sandwich is placed in a template on the unit and pressed to the desired thickness with the aid of a ³⁄₈- x 2- x 2-in. piece of glass over the top

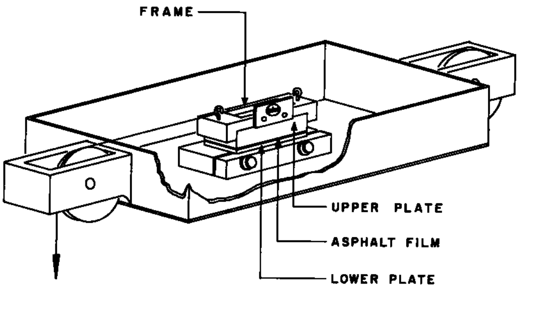


Figure 1. Microviscometer unit.

glass plate. This piece of glass keeps the fingers out of the way for viewing and from being burnt.

Application

Displacement of the top glass plate is plotted with respect to time on rectangular graph paper. Viscosity is calculated as follows:
Viscosity is calculated as follows:

$$\text{Viscosity in poises} = \frac{\text{Shearing Stress}}{\text{Rate of Shear}} = \frac{\dfrac{\text{Load in grams x 980}}{\text{Area in cm}^2}}{\dfrac{\text{Displacement in cm}}{\text{Film thickness in cm x seconds}}}$$

DISCUSSION OF DATA

Curves obtained using a 127 penetration Californian asphalt are shown in Figure 3. This Californian asphalt is showing Newtonian behavior, constant viscosity at varying rates of shear, under these conditions of testing as shown in Figure 4 (Curve 1).

All the curves in Figure 3 are straight lines passing through the origin which is typical for Newtonian fluids. If it had been known in advance that this asphalt had Newtonian behavior under these conditions, it would only have been necessary to obtain one curve in Figure 3 in order to plot the curve in Figure 4. The instrument is sensitive enough to obtain an accurate curve for Newtonian materials after only 25 to 50 microns displacement. Curves No. 2 and 3 (Fig. 3) then could have been completed in 2 and 10 min, respectively.

Results obtained using an 85 penetration Venezuelan asphalt are shown in Figure 5. In a viscoelastic material, the rate of shear under a given load may be rapid initially, but then decreases to a constant value. This condition is visible in Curves 5 and 6 (Fig. 5). The steady state viscosity is calculated from the straight-line portion of these curves. These results plotted as viscosities in Figure 4 (Curve 2) indicate that this asphalt has viscoelastic behavior, viscosity decreasing with increasing rate of shear.

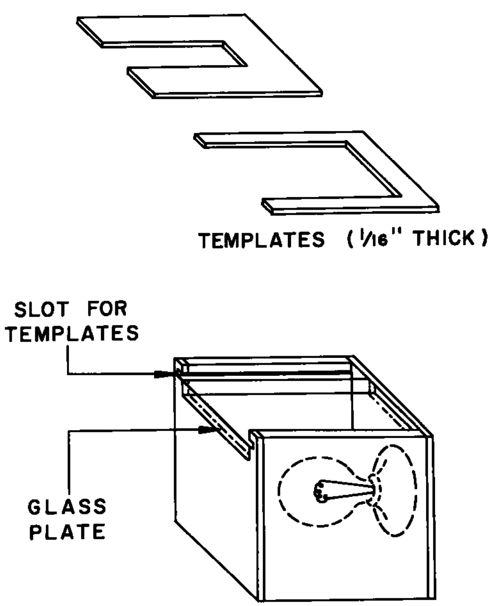


Figure 2. Film preparation unit.

For the Californian asphalt, a displacement of only 50 microns was sufficient. For the Venezuelan asphalt, the straight-line portion of Curve 5 (Fig 5), was reached at a displacement of 80 microns and after 4 min of testing.

Data obtained with the same Venezuelan asphalt at 140 F are shown in Figure 6. In Curve 9, the straight-line portion was reached in 25 min at a displacement of 400 microns. In Curve 10, it was reached in 45 min at 300 microns. The true viscosity rate of shear curve for this asphalt at 140 F is shown in Figure 4 (Curve 3). A flase viscosity curve is also shown in Figure 4 (Curve 4) which results from plotting the curves in Figure 6 after only 100 microns of displacement. The initial 100 micron portions of these curves appear as straight lines.

These three curves were taken to 0.15 cm displacement to be sure that no further change in rate of shear occurred.

Nine paving grade Venezuelan asphalts obtained from the same supplier and which met identical specifications (that is, penetration, softening point, ductility, etc.) were studied over a wide range of test conditions.

None were close enough to Newtonian behavior over a practical temperature and rate of shear range to allow the true steady state viscosity behavior to be determined from curves of less than 400 microns of displacement.

After similar studies on many other types of paving asphalts including Smackover,

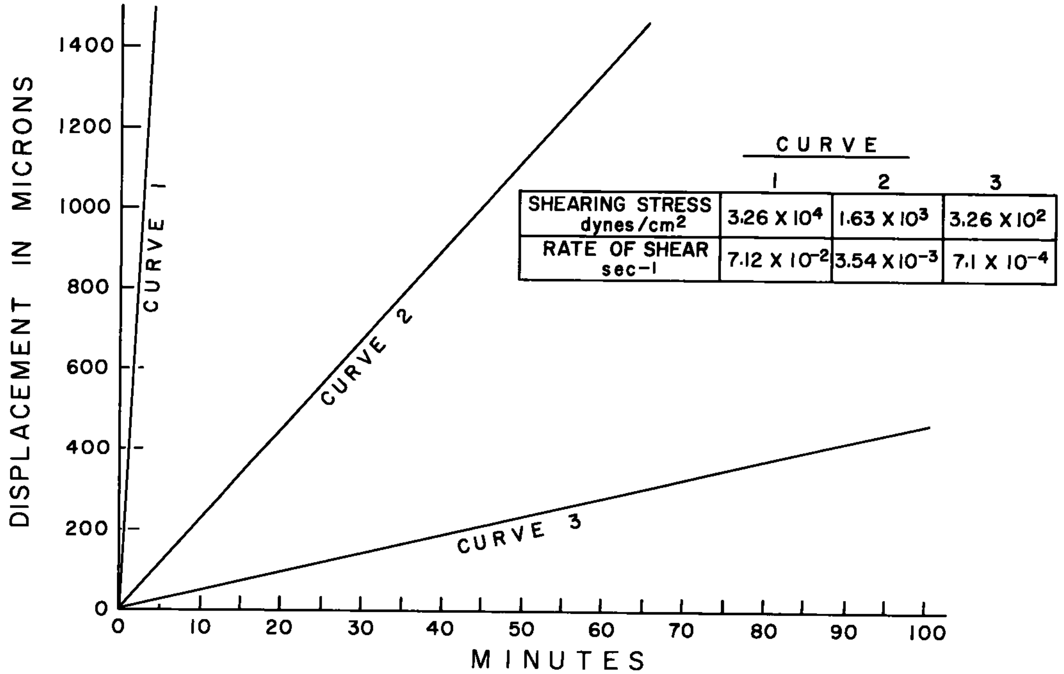


Figure 3. Displacement-time curves at 77 F for Californian asphalt, 127 penetration, 108 micron film.

Wyoming, Arkansas, Santa Maria, Martinez, Midcontinent, Texas, two from Canada, and others, the authors conclude that probably less than 10 percent of the paving asphalts should be evaluated with a displacement less than 400 microns.

Asphalts which deviate widely from Newtonian behavior present more of a problem. An example is the air blown Midcontinent asphalt, 135 pen. 127 F softening point, for which data are shown in Figure 7. In Curves 11, 12 and 13, the initial rate of shear is high, then it decreases as though it would reach a constant value; but instead, due to structural breakdown of the rheological units in the asphalt, it begins to increase again. In Curve 13, the rate of shear becomes constant in about 1 hr after 400 microns of displacement. In Curve 12, a constant rate of shear was not obtained until after 700 microns. In Curve 11, the initial bending due to elastic response and the opposite bending due to structural breakdown has all blended into a straight line. In Curve 14, no upward bending of the curve occurred after the straight-line portion was reached.

The curve obtained using a rubberized asphalt joint sealing material is shown in Figure 8. In this case no straight line portion has been reached even at 1,800 microns.

The same precautions must be followed when studying tars.

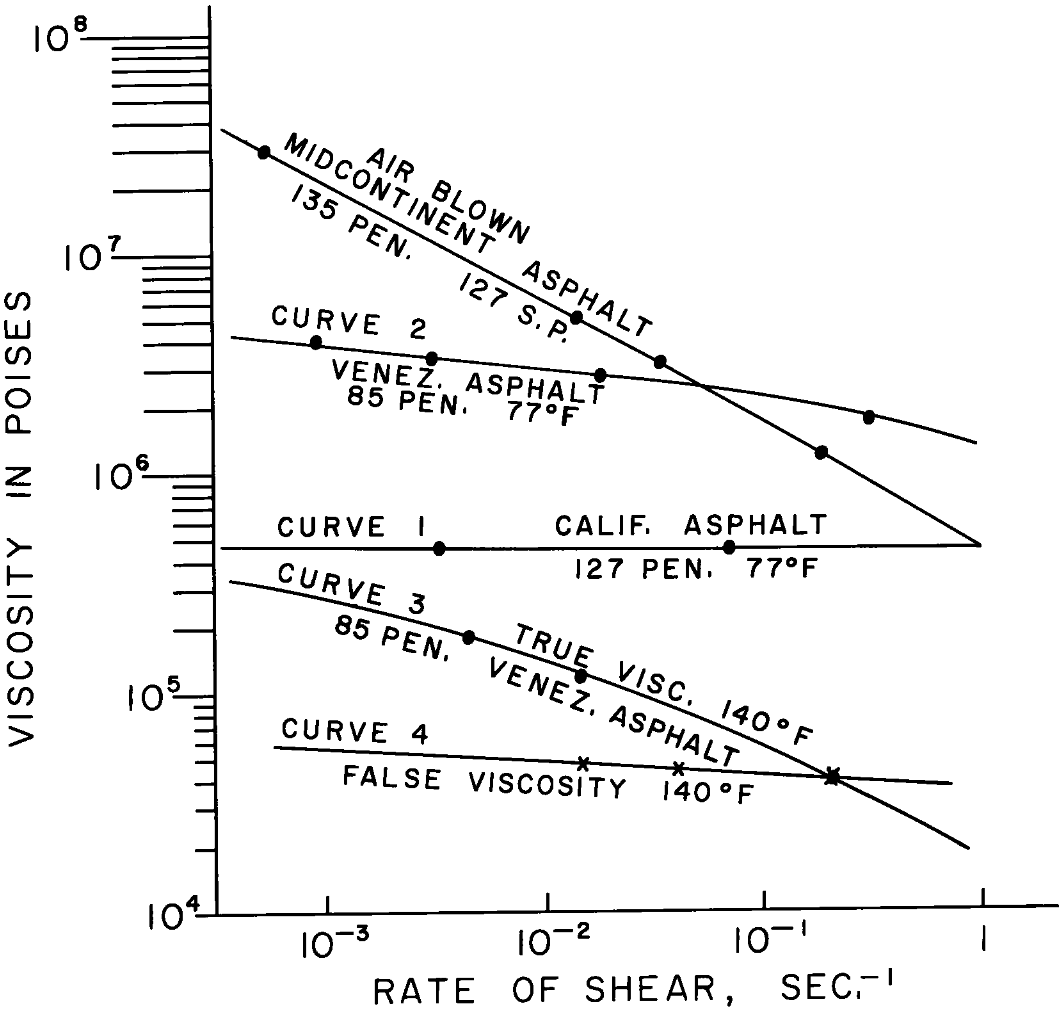


Figure 4. Viscosity at varying rates of shear.

Variation of Viscoelastic Behavior with Film Thickness

Previous authors have indicated that the viscoelastic behavior of bitumens could be studied in the parallel plate microviscometer without particular attention to film thickness within the range of the instrument (that is, 10 to 100 microns), because the behavior was not dependent on film thickness.

The authors find that viscoelastic properties change with film thickness and have reported it elsewhere (7).

The reason for the two points of view on this subject may be due to the displacement-time factor.

Typical displacement-time curves for various film thicknesses are shown in Figure 9. Notice that more than 100 microns of displacement was necessary to reach the straight-line portion in Curves 16, 17 and 18. A total testing time of about 3 hr was necessary to be sure that the straight line portion of Curve 18 was reached.

When the slopes 1, 2, 3 and 4 taken from the straight-line portion of each curve were used to calculate the viscosity, Curve 1 in Figure 10 was obtained which clearly shows the dependency of viscosity on film thickness.

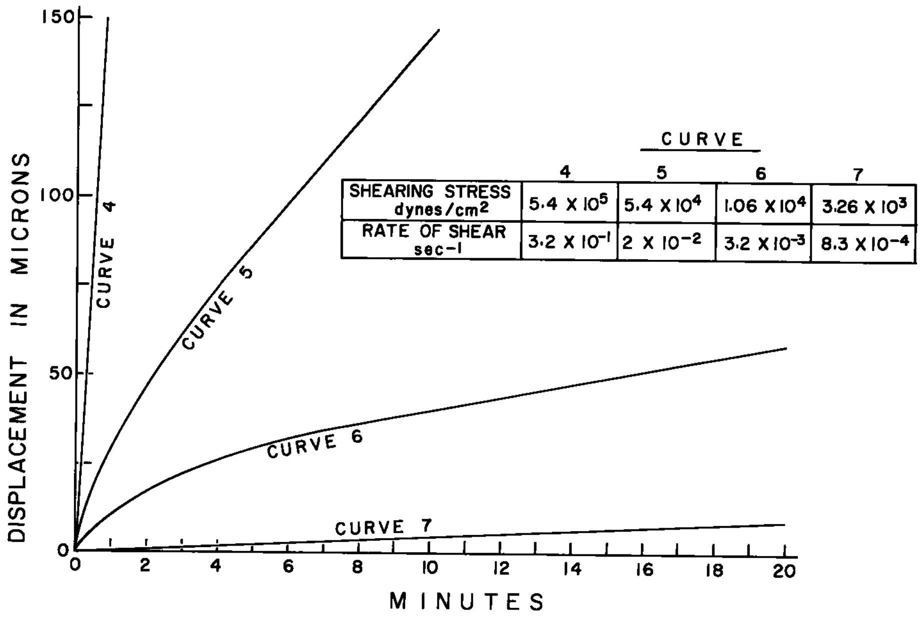


Figure 5. Displacement-time curves at 77 F for Venezuelan asphalt, 85 penetration, 10 micron film.

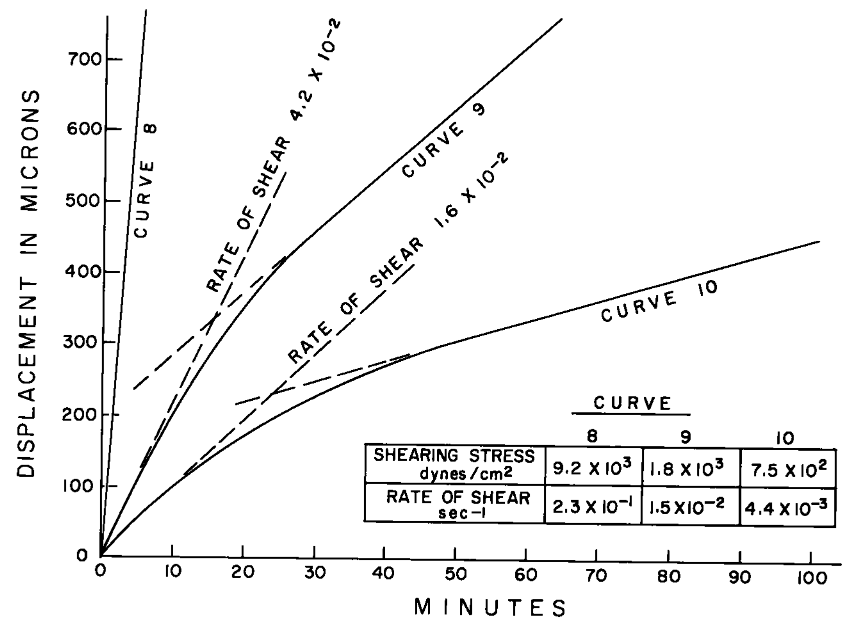


Figure 6. Displacement-time curves at 140 F for Venezuelan asphalt, 85 penetration, 10 micron film.

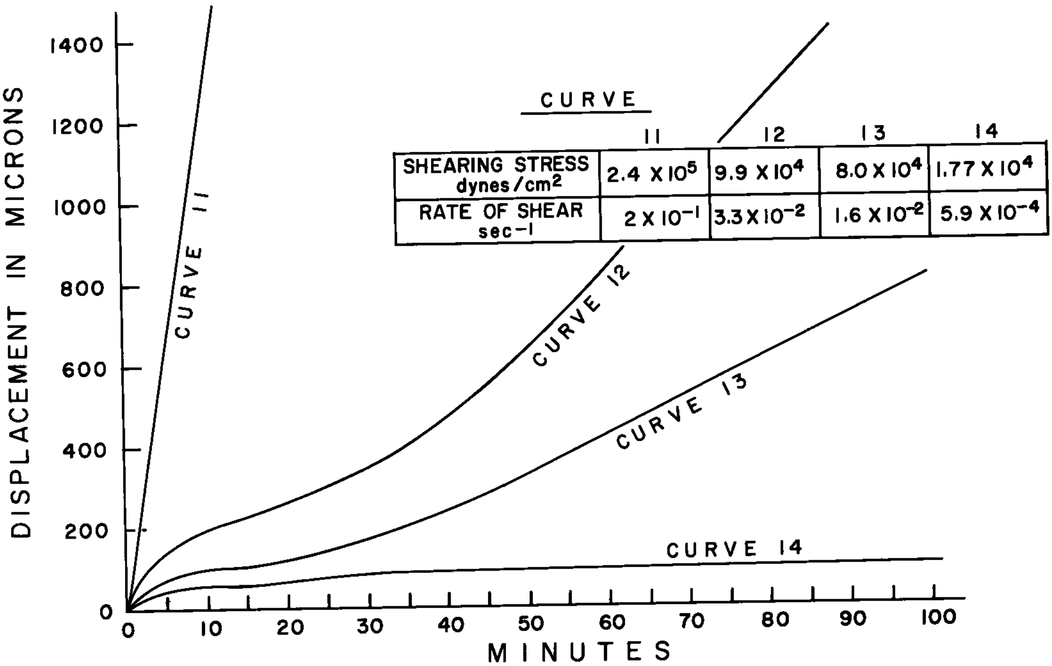


Figure 7. Displacement-time curves at 77 F for Midcontinent air blown asphalt, 135 penetration, 127 soft point, 10 micron film.

When slopes 1, 5, 6 and 7, taken from the first 100 micron displacement and which also appear to be straight lines, were used to calculate the viscosities, Curve 2 in Figure 10 was obtained which indicates that viscosity does not change with film thickness.

It is not surprising that the viscosity under shear increases with decrease in film thickness because it has been known for some time that tensile increases in the same manner. This was recently reported by Mack (3) and Wood (7). This work can be easily duplicated using the same 2-cm x 3-cm x 1/4-in. glass plates used in the parallel plate microviscometer.

A circular film of any desired thickness is prepared between crossed plates as shown in Figure 11. For films under 10 microns thickness, the diameter of the circle should not be greater than 1 cm. The advantage of using glass plates instead of metal is that uniformity of the film (that is, thickness and lack of cavities) can be insured by observation. A jig to hold the tensile specimen is also shown in Figure 11. The jig can be placed in a temperature controlled bath on the tensile machine. A compression head is used on the tensile machine because the jig reverses the force to one of tensile on the specimen.

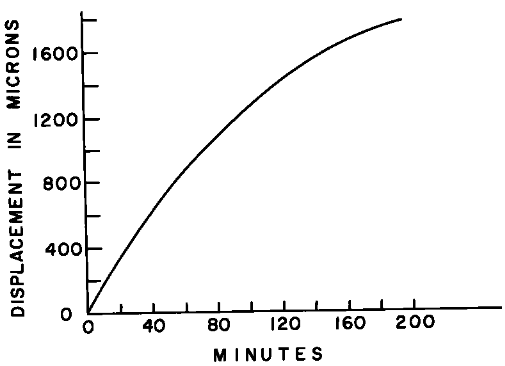


Figure 8. Displacement-time curve at 77 F for rubberized asphalt joint sealer, 10 micron film.

The effect of film thickness on tensile for the same 110 penetration asphalt is shown in Figure 12.

Notice that in Figures 10 and 12 this asphalt under these conditions of testing has what can be called an infinite thickness region, above which the viscosity or tensile does not change.

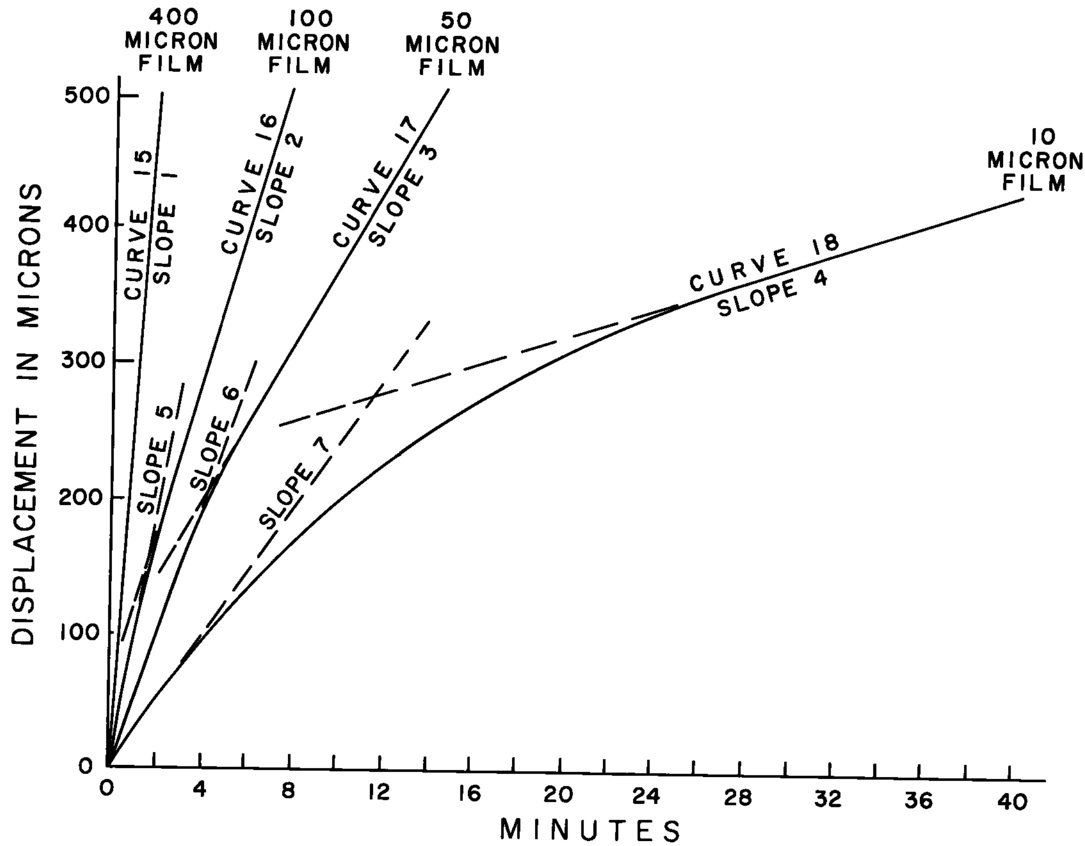


Figure 9. Displacement-time curves at 77 F and various film thicknesses for Venezuelan asphalt, 110 penetration.

Viscosity measurements on this asphalt were made in many types of fundamental viscometers such as capillary, rotational cylinder and disc, falling concentric cylinder, etc., up to asphalt thicknesses of 1/4 in. (6,350 microns) and within experimental error, the viscosity was the same as the viscosity in the infinitely thick region in Figure 10.

This also applies to tensile data on films 1/4 in. thick.

It is unlikely that the change in properties with film thickness in films greater than one micron thick can be attributed to long range London-vander Waal forces (6).

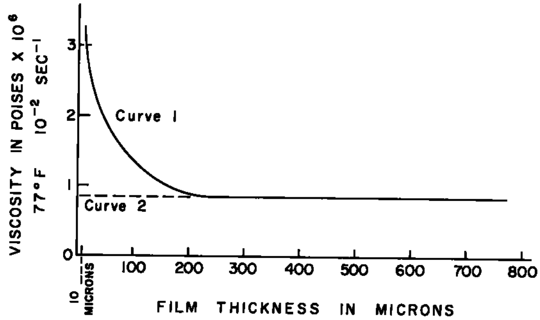
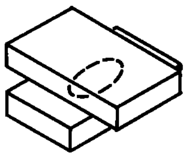


Figure 10. Effect of film thickness on viscosity at 77 F of Venezuelan asphalt, 110 penetration.

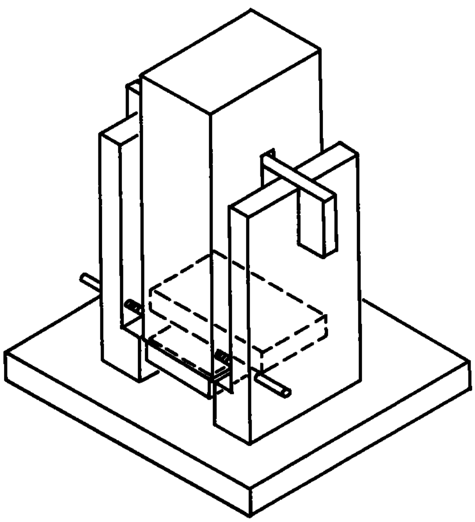
The answer probably lies in the degree and rate of structural breakdown in the bitumen. Any colloidal material which exhibits thixotropy must pass through intermediate stages of breakdown.

The straight-line portion, constant rate of shear, of the displacement-time curves represents an equilibrium between structural breakdown and reformation.

It has been shown that in thin films, the elastic effect on flow properties in many cases is not overcome until the displacement is many times the thickness. The true steady state viscosity of a 10



TENSILE SPECIMEN



TENSILE SPECIMEN HOLDER

Figure 11. Tensile specimen and holder.

micron film of rubberized asphalt (Fig. 8) was not reached in 1,800 microns of displacement.

In thick films such as occur in many viscometers, limitations of the particular type of viscometer may not allow the necessary amount of rate of displacement to take place to completely overcome initial elastic effects on flow or to reach the upward swing of displacement-time curve if one is present.

Rotational Parallel Plate Microviscometer

The present rectangular glass plates limit the amount of displacement to 0.15 cm, 5 percent of the total length. Actually the limit is 0.30 cm if the test is started with the top plate already off-set 0.15 cm. Even 0.30 cm is not sufficient for highly viscoelastic materials or thick films of even moderately viscoelastic materials under certain conditions of testing. The authors have overcome this by using circular glass plates which are rotated. In many ways the rotational movement lends itself to easier ocular or automatic measurement.

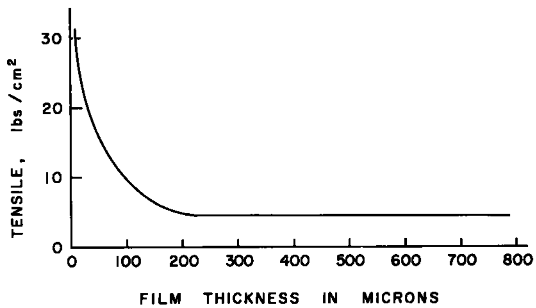


Figure 12. Effect of film thickness on tensile strength at 77 F of Venezuelan asphalt, 110 penetration.

CONCLUSIONS

The viscoelastic behavior of bitumens has been shown to vary with thickness when very thin films are involved. This must be taken into account especially in pavement work. The geometry of testing must match the geometry of end use.

The parallel plate microviscometer is an important contribution to the bituminous paving field because fundamental rheological data can be obtained on very thin films comparable to those obtained in practice.

Care must be taken in the use of the microviscometer that the total displacement is sufficient to reach a steady state condition of flow. This amount of displacement may be many times that suggested by previous authors.

The use of circular plates extends the useful range of the microviscometer.

REFERENCES

1. Griffin, R. L., Miles, T. K., and Penther, C. T., AAPT Proc., Vol. 24, pp. 31-53 (1955).
2. Labout, J. W. A., and Oort, W. P. van, "Micromethod for Determining Viscosity of High Viscosity Materials." Anal. Chem., 28:7, 1147-1151 (July 1956).
3. Mack, C., "Physical Properties of Asphalts in Thin Films." Ind. Eng. Chem., 49, pp. 422-427 (1957).
4. Oort, W. P. van, "A Study of the Aging of Asphaltic Bitumen." Drukkerij Boeijinga-Apeldoorn (1954).
5. Oort, W. P. van, Ind. Eng. Chem., 48 (July 1956).
6. Overbeek, J. Th. G., and Sparnaay, M. J., Discuss. Faraday Soc., 18, p. 12 (1954).
7. Wood, P. R., "Rheology of Asphalts and Its Relation to Behavior of Paving Mixtures." HRB Bul. 192, pp. 20-25 (1958).

Design and Construction of Epoxy Asphalt Concrete Pavements

W. C. SIMPSON, R. L. GRIFFIN, H. J. SOMMER, and T. K. MILES, Shell Development Company, Emeryville, Calif.

A new paving material developed specifically to withstand high temperature jet blast and fuel spillage and to provide the high load-carrying ability demanded by modern airfield and highway traffic is described. Epoxy asphalt concrete (EAC) is a combination of graded mineral aggregate and an asphaltic binder containing an epoxy resin which is converted into a polymer with unusual solvent and heat resistance.

The mechanical properties of EAC as well as its resistance to heat and solvents are summarized. Various aspects of mix and thickness design in relation to the production of EAC pavements are discussed and preferred mix plant and construction practices which have been developed from field experience are outlined.

The minor additions required for handling the binder in conventional hot-mix paving plants are mentioned and the control of polymerization rate by choice of aggregate temperature is described. Importance of aggregate gradation and binder content in achieving dense, impermeable pavements is demonstrated, and a preferred range of dense gradings compatible with good mix workability and aggregate availability is shown.

Spreading of the mix should be accomplished before the binder viscosity exceeds about 60 poises, and the influence of this fact on working time and choice of mix temperature is discussed. From compaction studies of epoxy asphalt concrete, preferred practices have been developed, and the results of these studies were found to be in good agreement with the theory of compaction of asphaltic concrete mixes.

Thickness design of EAC pavements using some of the existing methods of flexible pavement design has been considered. Some examples are shown and adjustments in the design procedures are suggested.

● FOREMOST among the problems facing the paving industry are those created by the steadily increasing loads imposed on airfields and highways. On airfields the structural problems are further complicated by the occurrence of damage to pavements by fuel and solvent spillage and by high temperature jet blast associated with modern military and commercial jet aircraft.

As a result of years of laboratory experimentation and field development work, a new paving material, epoxy asphalt concrete (EAC), has come to the fore as a product of great interest. Because of its combination of unusual mechanical properties and chemical resistance, EAC offers a simultaneous solution to the problems associated with heavy loads, high temperature blast and fuel spillage.

In a previous paper (1) the mechanical properties, heat resistance, and solvent resistance of epoxy asphalt concrete were discussed in detail and demonstrated by laboratory tests and by examples of field performance under conditions of actual use. The purpose of the present paper is to discuss some of the most important aspects of construction with EAC and to indicate in a preliminary way how the mechanical properties of the material may be used in thickness design of epoxy asphalt concrete pavements.

PRODUCTION OF EPOXY ASPHALT CONCRETE (EAC)

This new material of construction is produced in conventional hot-mix plants normally employed for preparation of asphaltic concrete mixes. Graded mineral aggregates are dried, screened and proportioned in the normal fashion and mixed in the pugmill with EPON (Trademark Shell Oil Company) asphalt binder, which initially has appearance and properties similar to asphalt. A chemical reaction which takes place in the binder during mixing, hauling, spreading, and compaction, and for some time thereafter, converts the binder into a thermoset plastic which is flexible, extensible and has high tensile strength. The mix is spread by conventional, self-propelled paving machines or by hand-raking and is compacted with steel and rubber-tired rollers of the normal type. Construction techniques and specifications for the successful application of this new product have been established in cooperation with the Products Application and Research Departments of Shell Oil Company in a series of field installations conducted during the past four years using commercial hot-mix plants and paving equipment. The finished pavement has the appearance of asphaltic concrete but its resistance to attack by solvents and fuels, its strength retention under high temperature jet blast, and its high load-carrying capacity represent great improvements over the properties of normal asphaltic concrete.

HANDLING THE BINDER IN THE MIX PLANT

The binder used in the production of EAC consists of a blend of a liquid epoxy resin and a paving grade asphalt containing an additive which functions as a flexibilizing coreactant. The asphalt containing the additive is stored and transferred in the existing facilities of the hot-mix plant and a minor addition to the plant is required to provide for pumping of the epoxy resin to the weigh platform where blending of the binder components is accomplished in the weigh bucket with brief stirring.

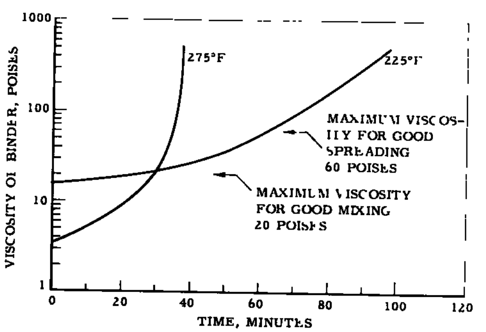


Figure 1. Influence of temperature and time on viscosity of the binder.

Influence of Temperature

The chemical reaction in the binder system which begins with the addition of the epoxy resin causes the visosity of the binder to increase steadily with the passing of time. Two examples are shown in Figure 1. The rates of reaction and of viscosity increase are readily controlled by the choice of temperature, an increase in temperature resulting in a higher rate of viscosity rise. A detailed knowledge of the influence of temperature on reaction velocity and viscosity has been obtained in studies of the reaction kinetics. As in normal plant practice, the binder is transferred promptly from the weigh bucket to the pugmill for mixing with the aggregate, and the temperature of the binder is determined thereafter by the temperature of the aggregate which constitutes over 90 weight percent of the mix. For good mixing with aggregate in the pugmill in a period of less than 1 min, a binder viscosity of less than about 20 poises is advisable, as indicated in Figure 1. Although the binder and aggregate can be successfully mixed at temperatures as low as 215 F, the aggregate may not be sufficiently dry at this low level and a somewhat higher minimum aggregate temperature is normally used for this reason. The maximum aggregate temperature which may be used is determined by the amount of time required for hauling, spreading, and compacting the mix. At high temperatures the polymerization reaction proceeds more rapidly, and the viscosity of the binder increases more rapidly than at low temperatures (Fig. 1). Laboratory and field experience have shown that the mixing, hauling and spreading of the mix should be accomplished by the time the binder viscosity has reached 60 poises in order to be able to obtain the desired degree of compaction readily with conventional rollers. The aggregate temperature to be used in a particular construction job will be determined largely by the hauling time involved because the mixing time in the plant and the time required for spreading the mix are essentially constants. Where the hauling time is short, relatively high temperatures can be used, but for long hauls lower temperatures are chosen. The practical range of temperatures for handling the mix falls within the normal operating range of hot-mix paving plants.

CHARACTERISTICS OF THE CURED BINDER

If desired, the binder may be cured in the form of sheets or other shapes for laboratory testing. Test methods devised for the characterization of elastomers and plastics are particularly applicable. Tensile tests of dumbbell-shaped specimens according to ASTM method D-412-51T at a rate of 20 in. per min at room temperature show that the binder has a tensile strength of 1,000 psi and an elongation at break of 200 to 300 percent. Correction of the tensile test results for the change in cross-section area during the test, gives a true tensile strength of 3,000 psi.

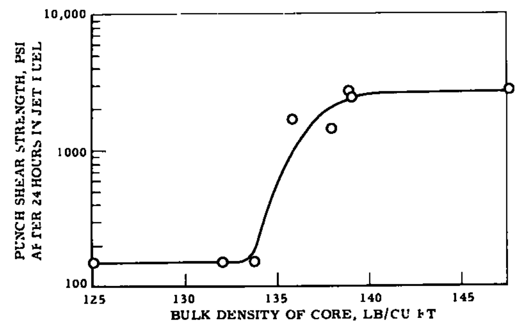


Figure 2. Influence of density on solvent resistance and shear strength of epoxy asphalt concrete.

The suitability of this material for use as a binder at high temperatures is shown by the fact that it has no melting point and holds its shape at temperatures as high as 800 F. Heating to a high temperature does not destroy the good low temperature flexibility of the binder as indicated by the results of the Fraass test. In this test (Institute of Petroleum method 80/53) a 0.5-mm thick layer of asphalt or other product is placed on a spring-steel strip and subjected to periodic bending which produces a maximum tensile strain of 3 percent in a period of 11 sec. The temperature is gradually lowered until the specimen fractures under this strain. A 60 penetration paving asphalt fractured at +16 F in this test whereas epoxy asphalt binder failed to fracture at -31 F. After exposure to a temperature of 760 F for 5 min the Fraass breaking point of the asphalt was raised to +25 F whereas the epoxy asphalt still failed to fracture at -31 F. This binder resists attack by fuels and solvents which rapidly dissolve normal paving grade asphalts.

MIX DESIGN

In many of the applications of EAC the material has been used as an overlay pavement to protect underlying asphaltic concrete from damage by solvents and fuel spillage. Where this is the major function of the overlay, it is desirable to produce a dense pavement, impermeable to these liquids. Laboratory and field investigations have also shown that the retention of shear strength of the EAC itself in contact with jet fuels is best when the pavement density is high. This is shown in Figure 2 for a group of cores taken from a series of field installations all employing the same aggregate. A punch shear test was used to measure the strength of ½-in. thick specimens of the overlays after 24 hr immersion in jet fuel. The test consists of placing the specimen over a ½-in. diameter hole in a steel plate and driving a 7/16-in. diameter cylindrical rod against the specimen with a testing machine at a rate of 0.2 in. per min until failure by shear results. In Figure 2 a transition from low to high shear strength is shown to occur as the density of the pavement increases over a fairly narrow range. The air void content decreases from about 6 percent to 3 percent (basis saturated surface dry aggregate density) as the shear strength after soaking in jet fuel increases from about 160 to 2,500 psi (Fig. 2). On the basis of data of this type, supplemented by permeability measurements on field installations, an air void content of 4 percent (based on saturated surface dry specific gravity of the aggregate) has been set as a goal in the construction of overlays where solvent resistance is important.

This low air void content is achieved by the use of a dense graded aggregate, high binder content in the mix, and by the use of heavy rollers in the compaction operation.

The Aggregate

For overlay pavements ¾ to 1 in. thick, a maximum aggregate size of ¼ in. is used, and in general a maximum size less than one-half the thickness of the pavement is recommended. Aggregates complying with conventional quality and gradation specifications are used. In order to provide impermeable pavements with high strength, dense graded aggregates are preferred. Andreasen and Anderson have shown (2) that aggregate grading curves (with the exception of skip graded systems) can generally be represented fairly well by equations of the form

$$P = CK^q \text{ or } \log P = q \log K + C^1$$

in which P = % of the aggregate passing a sieve of size K;
q = slope of the plot of log % passing each sieve versus log of sieve size; and
C = a constant, $C^1 = \log C$.

Using this type of representation of aggregate grading, Nijboer (3) has developed a simple graphical method of determining the voids in mineral aggregate from the grading curve. He also showed that the aggregate gradation which leads to minimum voids in the mineral aggregate or maximum density is a gradation in which the slope (q) of the plot of log percent passing various sieves versus log of sieve size is 0.45. Varying the slope, q, of the grading curve over the range 0.3 to 0.6 causes a variation of voids in the mineral aggregate of about 2 percent, indicating that the gradation may be varied within the fairly wide limits shown in Figure 3 without departing greatly from the maximum density. The various maximum density gradings proposed by Fuller, Campen (4), and Hveem (5) all fall within the range of gradings defined by the q values of 0.3 to 0.6. However, aggregate with ¼-in. maximum size and a q value of 0.3 contains more than 20 percent material passing the 200 mesh screen and paving mixes made with such gradings have been found to be extremely difficult to lay smoothly. Aggregate gradations falling anywhere within the boundaries defined by q values of 0.45 to 0.6 (Fig. 3) have been found by field experience with EAC to be the most practical of the dense graded mixes from the point of view of availability and workability.

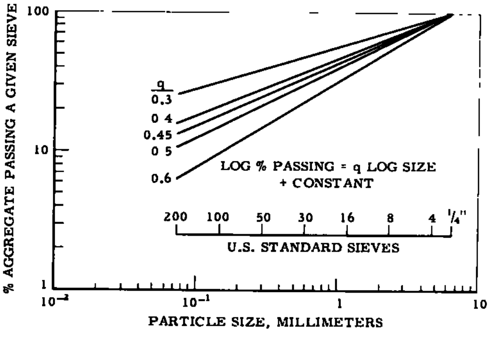


Figure 3. Aggregate gradations for dense, impermeable pavements.

Binder Content

The binder content to be used in the mix may be estimated from the voids in the mineral aggregate (VMA) after deciding on the acceptable level of voids in the compacted mix (VCM), as follows:

% VMA - % VCM = V% Binder Required.

For example, if the voids in the mineral aggregate amount to 21.5 percent and it is desired to have 4 percent voids in the compacted mix then 17.5 percent volume binder will be required in the mix. If the apparent specific gravity of the aggregate is 2.5, in this example, the binder will constitute 8.2 percent weight of the total mix or 8.9 percent of the weight of the mineral aggregate.

The Marshall method of mix design has been used successfully for EAC with stability of the cured mix reaching values as high as 20,000 lb at 140 F. In general, the binder content used in practice is higher than the binder content giving maximum Marshall stability. Field studies and commercial installations with a variety of mixes have shown 7 to 10 percent weight of binder basis aggregate to be the preferred range for ease of workability in the preparation of dense, impermeable EAC desired for good solvent resistance. These high binder contents can be used without producing the unacceptably low stability usually experienced with asphalt because of the high strength and heat resistance of the epoxy asphalt binder.

PLACEMENT AND COMPACTION OF THE MIX

Surface Preparation

When the epoxy asphalt concrete is used as an overlay on existing pavements, the surface should be swept free of dust and dirt and a suitable tack coat should be applied. Tack coats of emulsified asphalt, cutback asphalt and penetration grade asphalt have been used successfully in many installations. Where the pavement is subjected to normal pretakeoff conditions by military or commercial jet aircraft, an asphalt tack coat appears adequate. Where the pavement is subjected to prolonged high temperature blast, such as in maintenance and engine overhaul areas, a tack coat of the epoxy asphalt binder may be required.

Synchronizing Plant and Field Operations

As mentioned earlier, the temperature used at the mix plant is chosen so that sufficient time is allowed for mixing, hauling, and spreading of the mix before the binder viscosity reaches about 60 poises. The mix is produced at such a rate that it can be spread by the paving machine at a convenient speed. This simple scheduling of mix plant and laying operations avoids undesirable holding time for the mix in trucks at the job site or periodic starving of the paving machine. The mix has been spread successfully by hand-raking and by several types of self-propelled finishing machines commonly used in hot-mix paving.

Compacting the Mix

Rolling studies have been conducted on EAC mixes at field installations with steel rollers ranging from 3.5 to 12 tons, and with rubber-tired rollers. The degree of compaction obtained under a variety of conditions was assessed by determining density and air void content of cores taken from the pavements after compaction. The results obtained in these studies with EAC are in general agreement with experience in compaction of asphaltic concrete. By use of a variety of rollers the degree of compaction was found to be directly proportional to roller weight (P), and to the number of passes (N), and inversely proportional to wheel width (W). Degree of compaction was shown to be inversely proportional to the viscosity of the mass (η_m) by varying the binder content of the mix at constant temperature and constant age of the mix. These results agree with those obtained by Nijboer (3) who showed that the degree of compaction or "rolling factor" (R_f) for asphaltic concrete is related to the roller geometry and speed, consistency and thickness of the mix and the number of passes by the following expression

$$R_f = \frac{PN}{WD\eta_m} (h/v)^{0.4}$$

in which

D = roller diameter;

h = pavement thickness; and
v = roller velocity.

For binder contents of 8.5, 9.5, and 10.5 percent in dense graded sand sheet mixes, Nijboer found that an air void content of 4 percent is obtained during compaction when the magnitude of the rolling factor reaches values of 43×10^{-4}, 28×10^{-4}, and 14×10^{-4} inch-pound-seconds, respectively. From the authors' rolling studies with EAC it was concluded that the desired density and low void content can be obtained by making the break-down roll with 10- or 12-ton steel rollers followed by surface finishing with rubber-tired rollers with weights of 2,000 lb per wheel and tire pressures of about 70 psi or higher.

PROPERTIES AND PERFORMANCE OF EPOXY ASPHALT CONCRETE

A program has been undertaken in which the mechanical properties of this new paving material have been investigated in the laboratory, and performance under conditions of actual use has been determined in the field. The results of these investigations, described in detail in a previous paper (1), will be briefly summarized here. The field installations, consisting of ½- to 1-in. thick overlay pavements, have been placed in cooperation with the Products Application and Manufacturing-Research Departments of Shell Oil Company. These studies have been made with mixes employing dense graded aggregates with a maximum particle size of ¼ in.

Stability

By means of the Marshall test the stability of epoxy asphalt concrete has been determined at several levels of binder content. The specimens were prepared with 75 blows on each face and were cured 4 hr at 250 F. The stability of EAC at 140 F is in the range of 10,000 to 20,000 lb as compared with a range of 800 to 3,500 lb for conventional asphaltic concrete made from this dense graded crushed aggregate. There is little apparent damage to the EAC specimens during Marshall testing, and when the load is removed the specimen rebounds, about 60 to 70 percent of the "flow value" actually being an elastic and recoverable deformation (Table 1). Asphaltic concrete does not show this type of recovery but remains permanently deformed after the Marshall test.

TABLE 1

MARSHALL STABILITY AND FLOW VALUES FOR EPOXY ASPHALT CONCRETE AND ASPHALTIC CONCRETE AT 140 F

% Binder Basis Aggregate	Average Marshall Test Values for Duplicates: Stability (lb)	Flow in Inches: Total	Flow in Inches: Rebound	Flow in Inches: Permanent	Bearing Capacity,[1] psi
6	15,900	0.17	0.12	0.05	973
7	19,900	0.18	0.12	0.06	1,135
8	16,100	0.22	0.13	0.09	706
6% Asphalt 85/100	3,560	0.14	0	0.14	275

[1]Tire pressure which can be tolerated at 140 F without producing a compressive strain in excess of 1 percent.

In Table 1 the bearing capacities of epoxy asphalt concrete calculated by the method of Metcalf (6) are shown to be in the range of 700 to 1,100 psi as compared with 275 psi for the asphaltic concrete made with the same aggregate. This represents an improvement which is of particular significance in view of the high tire pressures and wheel loadings which are encountered in modern military aircraft.

The ability of pavements to retain high load carrying capacity after many repeated loadings is particularly important in taxiways and runways where aircraft traffic is channelized. Repeated loading of epoxy asphalt concrete simulating thousands of coverages by heavy bomber traffic has shown that epoxy asphalt concrete retains high stability and low flow values and resists densification.

Bearing Capacity of Field Installations

The chemical reaction which converts the binder from a viscous liquid to a nonmelting plastic is still in progress when the hot mix is spread and compacted and continues from some time in the pavement. The progress of the reaction in the pavement may be followed by determining the bearing capacity of the pavement periodically by means of the 90-deg cone penetrometer test which gives bearing capacity results in good agreement with theory when used on sand sheet mixes of the type involved in epoxy asphalt overlays. Figure 4 shows data taken on an EAC overlay placed at an aircraft maintenance base in the San Francisco area. This overlay supported a truck with a 17,700-lb rear axle load without shoving within 15 min after compaction. One day after placement the pavement could withstand a tire pressure of 200 psi and this increased to 740 psi within one week. A bearing capacity in the range 2,000-3,000 was attained in 30 days.

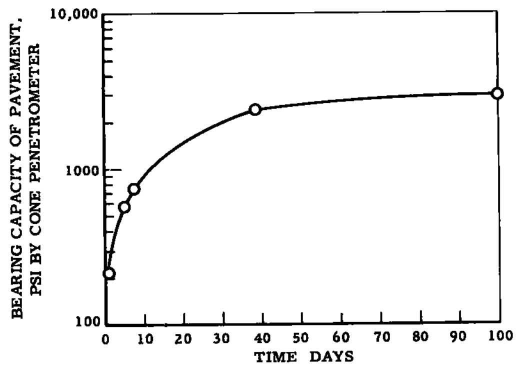


Figure 4. Increase of bearing capacity with time.

These bearing capacity values are far beyond the requirements of aircraft now in use, the highest tire pressures currently encountered being about 500 psi on some aircraft of the U.S. Navy, as described by Hansche (7). A practical example of the need for the high bearing capacity of EAC is found in areas where vehicles such as fork-lift trucks and steel-wheeled dollies carrying heavy loads on small diameter wheels are used. Such a case was encountered in an aircraft assembly plant where large sections of structural members were being transported on trains of steel-wheeled dollies with wheel loads of 800 lb per inch of wheel width. The rough, cracked surface of the portland cement concrete floor was causing damage to the aircraft sections and this condition was alleviated by placement of a ½-in. EAC overlay to serve as a new smooth running surface. The overlay was placed during a weekend and had developed sufficient bearing capacity by the following Monday to support the heavy loads without indention of the pavement.

Fuel and Solvent Resistance

The relative solvent resistance of low air void content asphaltic concrete and epoxy asphalt concrete to jet fuel at 140 F is given in Table 2. Asphaltic concrete disintegrates within 6 hr under these severe conditions while EAC retains a Marshall stability of about 15,000 lb.

The solvent resistance of epoxy asphalt concrete pavements has been tested in a bulk depot where a small amount of spillage and dripping of fuels and lubricants occurs constantly, requiring frequent replacement of asphaltic concrete. A ½-in. overlay of EAC has been effective in protecting the underlying pavement from softening by fuels and lubricants for several years.

An extremely severe condition of solvent and fuel damage was encountered at the maintenance base of a commercial airline in the San Francisco area. As part of the maintenance and overhaul routine, planes are placed on a designated area where engines soiled by oil leaks are washed down with a petroleum solvent (resembling paint thinner) which falls on the pavement. Oil filter changes are made with some spillage; fuel tank drains at 18 locations on the aircraft are opened to remove water and some highly aromatic aviation gasoline as well. Hydraulic fluids frequently accumulate in small amounts on the pavement. As a result of the good performance of an EAC overlay in this severe service the airline elected to protect a three-acre area of asphaltic pavement at a new jet maintenance base with an epoxy asphalt overlay.

Jet Blast Resistance

Epoxy asphalt overlays 1-in. thick have been placed on spalled portland cement concrete slabs and on eroded flexible pavements in overhaul and maintenance areas used for jet planes at several military air bases where thermocouples have been installed in

TABLE 2

SOLVENT RESISTANCE OF EPOXY ASPHALT CONCRETE COMPARED WITH ASPHALTIC CONCRETE

Soaking Time in Jet Fuel at 140 F (hr)	Marshall Stability (lb) Epoxy Asphalt Concrete	Marshall Stability (lb) Asphaltic Concrete
0	16,400	2,650
4	14,400	665
16	15,200	Disintegrated after 6 hr
20	15,200	

the overlays for temperature measurements during blast tests. Some of these tests used cycles which simulated normal pretakeoff operation and consisted of periods of idle power operation and of 100% power or "military" operation. In other tests, prolonged use of the afterburner was also included which resulted in surface pavement temperatures as high as 800 F. The over-all performance of the overlay in these tests, which involved conditions far exceeding the severity and duration expected in normal use by military aircraft, was judged to be excellent. It has been pointed out by the Federal Aviation Agency that the severity of jet blast and temperatures encountered with commerical airline jet planes is considerably less than in the case of militay aircraft.

Tensile Properties

In order to predict the behavior of epoxy asphalt concrete under various conditions of loading at various temperatures, a knowledge of its major mechanical properties is required. The tensile strength has been determined over a temperature range of 32 to 140 F at loading times from about 1 to 10^6 sec. Some results are given in Table 3 for EAC with 8% binder in the mix and for asphaltic concrete with 8 percent of a 60 penetration asphalt as binder. The tensile strength is rather insensitive to changes in loading time and in this respect resembles portland cement concrete much more than asphaltic concrete. Changes in tensile strength with temperature are also considerably less than those found with asphaltic concrete.

The tensile strain of epoxy asphalt concrete at fracture proved to be almost independent of the loading time and averaged about 3 percent. This tensile strain is about three times that obtained with asphaltic concrete and about ten times the tensile strain at fracture of portland cement concrete. From the tensile strength and strain measurements the stress/strain modulus at fracture has been determined over a wide range of long loading times and temperature for EAC. The insensitivity of EAC to changes in loading conditions is shown in Figure 5.

The response of EAC to dynamic loading of the type associated with moving traffic

TABLE 3

TENSILE STRENGTHS OF ASPHALTIC CONCRETE (AC) AND EPOXY ASPHALT CONCRETE (EAC)

Loading Time, (sec)	Tensile Strength (psi) At 32 F AC	At 32 F EAC	At 77 F AC	At 77 F EAC	At 140 F AC	At 140 F EAC
4	850	2,600	130	1,000	6	140
40	570	2,000	60	620	3	120
400	310	1,500	35	470	2	100
4000	210	1,170	20	350	1	90

has been determined by studying the bending of beams of the material at various frequencies corresponding to loading times of 10^{-3} to 1 sec and temperatures from 32 to 140 F. The stress/strain modulus of the material under these short loading times varies from about 3 x 10^6 to about 10^5 psi in the temperature range 32 to 140 F. Under these conditions, epoxy asphalt concrete behaves somewhat like portland cement concrete which has a stress/strain modulus of about 3 x 10^6 psi. In the course of the dynamic testing, beams of EAC have been flexed 100 million times without fracture, indicating that the material has good resistance to failure by fatigue.

Flexural Strength

The flexural strength or modulus of rupture of pavements is an important property which characterizes their behavior under loading. This measurement has been made according to the procedure specified in ASTM methods C293-57T or C78-49 which consists of supporting a beam of standardized dimensions near the ends and producing bending by center loading at a specified rate with a testing machine. From the maximum load and the dimensions of the specimen the modulus of rupture or flexural strength is calculated. As an addition to the ASTM procedure the authors have found it very useful to measure the deflection of the beams at the center during the test by means of a dial gage. This measurement gives a good impression of the relative flexibility of various paving materials.

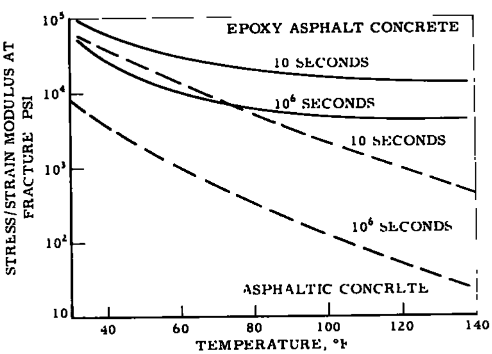


Figure 5. Dependence of stress/strain modulus of AC and EAC on temperature and loading time.

Figure 6 shows modulus of rupture and beam deflection values at maximum load for portland cement concrete, asphaltic concrete, and epoxy asphalt concrete. The beams used were 2 x 3 x 12 in. with a testing span length of 9 in. At the left of the graph the modulus of rupture of portland cement concrete is shown covering a range of about 400 to 800 psi. The center deflection of the beams at maximum load averaged 0.01 in. in these tests. Along the bottom of the graph are the data for asphaltic concrete at three levels of binder content with the modulus of rupture in the range 75 to 100 psi. The beam deflections are very large because of the flexible nature of asphaltic concrete.

The three center curves in Figure 6 show the properties of epoxy asphalt concrete at three levels of binder content; 7, 8, and 12 percent basis aggregate. The circles, triangles, and squares represent the use of three flexibilizing additives which differ in composition. The modulus of rupture of epoxy asphalt concrete may be varied from about 400 psi, which is about the lower level for portland cement concrete, to about 2,400 psi, which is two to three times that obtained with the best portland cement concrete. At the same time, epoxy asphalt concrete may be flexed to the same extent as asphaltic concrete at maximum load. Thus we have a structural material which combines the desirable properties of both rigid and flexible pavements; a strength equal to or greater than that of portland cement concrete and a tolerance for bending equal to that of asphaltic concrete.

FLEXIBLE PAVEMENT DESIGN METHODS APPLIED TO EAC

The use of epoxy asphalt concrete for structural purposes as an overlay on existing pavements or as the major structural element in a new pavement requires a thickness design procedure which takes into account the mechanical properties of the material. Most of the existing procedures for the design of flexible pavements are in reality procedures for thickness design of bases and subbases for flexible pavements, inasmuch as they take little or no account of the properties of the asphalt itself. Conventional asphaltic concrete is recognized as contributing more to the load-carrying capacity of the road than an equal number of inches of crushed rock in several methods of design whereas other design procedures consider the bituminous pavement surfacing only as a safety factor over the required design thickness of base.

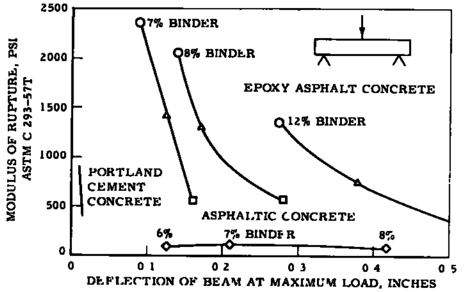


Figure 6. Modulus of rupture and flexibility of epoxy asphalt, portland cement and asphaltic concretes.

Two methods of flexible pavement design in use in this country which take the properties of the pavement surface into account in determining total required pavement thickness are the method proposed by Hveem and Carmany (8) used by the California Division of Highways, and the procedure of Palmer and Barber (9, 10) used by the State of Kansas.

Hveem-Carmany Design Method

In the California method the required thickness of pavement (T) is proportional to the tire pressure (p), the square root of the tire contact area (a), the logarithm of the number of stress repetitions (r), the horizontal pressure (Ph) observed in a Hveem Stabilometer test on the material at a vertical pressure (Pv), and inversely proportional to the fifth root of the cohesion (C) of the bituminous surfacing. The thickness design equation is

$$T = \frac{K p\sqrt{a}\ \log r\ (Ph/Pv - 0.10)}{\sqrt[5]{C}}$$

in which K = 0.0175 is a correlation coefficient bringing the equation into agreement with field experience. For convenience, the tire pressure, contact area, and number of repetitions of load are combined into a traffic index (T.I.), and the properties of the soil determined by the stabilometer test are expressed as a resistance value (R). The equation then becomes $T = 0.095\ (T.I.)\ (90-R) / \sqrt[5]{C}$. The cohesion (C) is determined at 140 F on a specimen of the compacted bituminous surfacing in the Hveem Cohesiometer, the test being roughly analogous to the modulus of rupture. The value of C increases with increasing tensile strength of the pavement surfacing and is said to be approximately equal to 45.4 times the modulus of rupture at 140 F.

In an attempt to obtain a cohesiometer value for epoxy asphalt concrete, a cylindrical core from a field installation was tested in the cohesiometer at 140 F but did not break under the maximum amount of bending which can be obtained in this apparatus. The modulus of rupture of epoxy asphalt concrete beams 2 x 3 x 12 in. containing 8 percent binder determined by ASTM method (293-57T) at 140 F was found to average 600 psi. A value of C of 600 psi x 45.4 or 27,200 is then estimated for EAC.

The thickness design equation may be conveniently solved for a given set of loading and traffic conditions and soil properties using a value of C of 100 which is characteristic of untreated gravel or crushed rock bases. The value of T obtained is the thickness of untreated granular base required over the soil involved and any combination of base and surfacing material equivalent to this thickness of gravel may be used. The gravel equivalents ($\sqrt[5]{C/100}$) for various materials given in Table 4 are those used by the State of California with the exception of the value for epoxy asphalt concrete which was determined as described previously. It will be noted that 1 in. of EAC is equivalent to 3.1 in. of gravel while 1 in. of asphaltic concrete is equivalent to 1.3 in. of gravel. Thus, 1 in. of EAC is equivalent to 2.4 in. of AC.

TABLE 4

COHESIOMETER VALUES AND GRAVEL EQUIVALENTS OF VARIOUS MATERIALS

Material	Cohesiometer Value at 140 F	Gravel Equivalent (Inches of Gravel Equivalent to 1 in. of the Material)
Epoxy asphalt concrete	27,200	3.1
Cement treated base, class A	1,500	1.7
Cement treated base, class B	750	1.5
Asphaltic concrete (85 to 300 pen. asphalt)	400	1.3
Plant mix with grade 4 and 5 cutback	150	1.1
Untreated bases and subbases	100	1.0

As an example of pavement design the authors assume a soil with an R value of 21 and determine the thickness of gravel cover with a cohesiometer value of 100 or its equivalent, required to support 19,200,000 equivalent 5,000-lb wheel loads corresponding to a traffic index of 8.7. From the equation, the thickness is calculated as 23 in. of gravel. The equivalent of 23 in. of gravel in various combinations of asphaltic concrete (AC) surfacing and crushed rock base, and combinations of epoxy asphalt concrete surfacing and crushed rock base are shown in Figure 7. For the particular loading conditions, traffic and soil properties used in the examples, it may be seen that over a 12-in. crushed rock base 3.8 in. of epoxy asphalt concrete would be required as compared with 8.5 in. of asphaltic concrete.

Two aspects of this thickness design method require some adjustment if it is to be used for EAC. First, the physical limitation of the cohesiometer in testing EAC specimens should be removed or another test method substituted for it and secondly, the validity of the correlation coefficient for EAC should be established. It seems likely that a new value of the constant would be required for each new paving material.

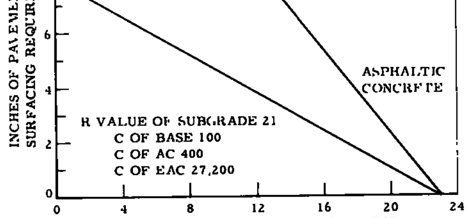


Figure 7. Thickness design of AC and EAC pavements with the Hveem-Carmany method.

Palmer and Barber Design Method

This method of flexible pavement design used by the Kansas State Highway Commission was developed by Palmer and Barber (10, 11) as an extension of the Boussinesq equations which deal with the stresses and deflections due to circular loads for a single layer case. Palmer and Barber extended the equations to include the properties of the pavement surfacing, bases and subbases, introduced coefficients taking into account the traffic volume and degree of saturation of the subgrade and substituted a modulus of deformation for a modulus of elasticity. The thickness design equation follows:

$$T = \sqrt{\frac{3Pmn^2}{2\pi ED} - a^2} \sqrt[3]{\frac{E}{Ep}},$$

in which T = total thickness of pavement required;
P = wheel load;
m = traffic coefficient based on volume of traffic;
n = saturation coefficient of subgrade based on rainfall;
a = radius of a circular area equivalent to the contact area;
D = deflection of the pavement surface permitted;
E = modulus of deformation of subgrade or subbase; and
Ep = modulus of deformation of pavement or surface course.

The equation gives the thickness of the pavement surfacing required if placed directly on the subgrade. If it is desired to introduce some granular base course material between the subgrade and the pavement surfacing, this is covered by

$$t_b = (T - t_p) \sqrt[3]{\frac{Ep}{Eb}}$$

in which t_b = thickness of base course;
T = thickness of pavement surfacing if used alone;
t_p = thickness of pavement surfacing when used with base;
Ep = modulus of deformation of pavement surfacing; and
Eb = modulus of deformation of base material.

If it is desired to use still a third layer of material such as a subbase, another similar calculation is required.

The values of E are determined for subgrade, base, and pavement surfacing from the stress-strain curves of triaxial compression tests. Some examples of modulus of deformation for several subbase, base, and pavement surfacing materials at room temperature as reported by the Kansas Highway Commission are given in Table 5 (HRB Bul. 8). Values of the modulus of deformation for epoxy asphalt concrete determined by the triaxial compression method (Kansas procedure) proved to be about 100,000 psi. The modulus of deformation is derived from the stress versus compression curve obtained in a triaxial compression test where the lateral pressure is maintained constant at a value of 20 psi during the test. This value of lateral pressure applied to the specimen is intended to simulate the lateral support or horizontal resistance which is normally provided by the adjacent similar material in the road. The horizontal resistance in an EAC pavement would be expected to be considerably greater than 20 psi,

TABLE 5

MODULUS OF DEFORMATION FOR VARIOUS MATERIALS FOR USE IN THE THICKNESS DESIGN METHOD OF PALMER AND BARBER

Subbase and Base Materials	Eb or Esb, psi
Coarse sand and gravel bound with soil	7,000
Fine sand and gravel bound with soil	6,000 - 7,000
Crushed limestone bound with lime dust	10,000
Chat bound with silica dust	10,000
Pavement Surfacing Materials	**Ep, psi**
Dense graded surface course, slow curing cutback	15,000
Asphaltic concrete	25,000
Epoxy asphalt concrete (triaxial)	100,000

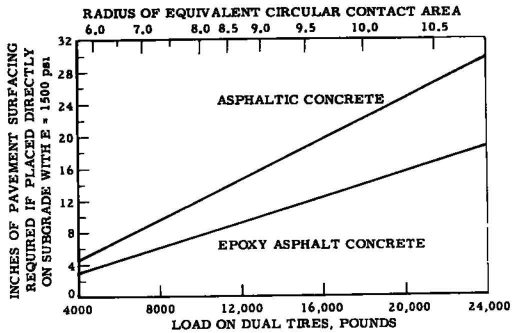


Figure 8. Thickness design for AC and EAC using the method of Palmer and Barber—surfacing placed directly on subgrade.

and a modulus of deformation of 100,000 psi for EAC determined with a 20 psi lateral pressure is thus a conservative value.

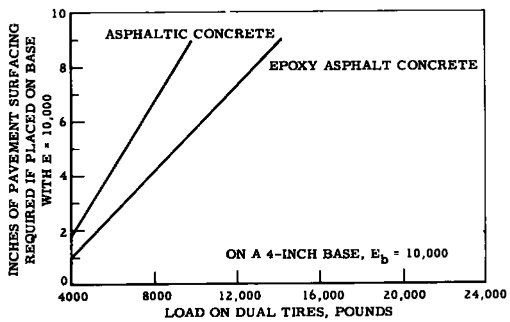
A number of examples of thickness design have been calculated for asphaltic concrete and for epoxy asphalt concrete for wheel loads on dual tires ranging from 4,000 to 24,000 lb, taking into account the variation of contact area with load. In the examples, a clay subgrade with a modulus of deformation of 1,500 psi has been assumed. A saturation coefficient of 1.0 (35 to 40 in. of rainfall per year) is used, and a traffic coefficient of 1.0 (1,200 to 1,800 vehicles per day with the design load) was applied. A deflection of 0.1 in. is permitted as has been customary in the use of this design procedure. The thicknesses of pavement surfacings required if placed directly on the subgrade are shown in Figure 8. The thickness of EAC required is about 0.6 of the thickness of AC needed according to this analysis.

When a crushed rock base of high quality (Eb = 10,000) is introduced between the subgrade and the pavement surfacing, the required thickness of surfacing is substantially reduced. Several examples of design with bases of varying thickness have been calculated, and cases involving 4- and 12-in. crushed rock bases placed on a clay subgrade (E = 1500) are shown in Figure 9 for a range of loads. The thickness requirements are reduced to realistic levels by the addition of the base, and where 4 in. of AC is called for, 2¼ in. of EAC will be required by this design procedure.

The major adjustment in this design procedure which should be made, is the use of a higher, more realistic value of the lateral pressure in the triaxial testing of the EAC specimen. Increasing this pressure would lead to some increase in the modulus of deformation of EAC which would result in a further reduction in thickness requirements.

EAC Overlays

Overlay pavements constitue a particularly appropriate use of epoxy asphalt concrete because of the variety of problems which can be solved simultaneously. From a structural point of view, 1 in. of EAC is as effective as 2 to 2.5 in. of AC and this can be of particular importance in cases where resurfacing is needed but where grade lines and drainage contours are to be held close to existing levels. In resurfacing of structures where added weight must be held to a low level, the EAC overlay gives high strength at relatively low unit weight because of the lower thickness required. When the overlay is to be used to strengthen an existing flexible pavement, the



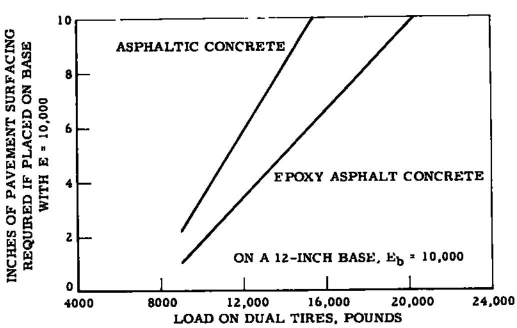


Figure 9. Thickness designs for AC and EAC on a crushed rock base—using the method of Palmer and Barber.

thickness of EAC overlay required may be estimated by the use of existing design methods. For example, the Hveem-Carmany procedure can be used in the following manner. Suppose that the number of 5,000-lb equivalent wheel loads applied to the pavement of Figure 7 is to be increased by a factor of about 10 (from 19,200,000 to 188,300,000) corresponding to an increase in traffic index from 8.7 to 11. To carry this increased load without producing shear failures in the subgrade (R value 21) a thickness of 29 in. of gravel or its equivalent is required by the design method as compared with 23 in. for the traffic index of 8.7. The additional 6 in. of gravel or its equivalent may be added in various ways. Using a gravel equivalent of 3.1 for EAC (Table 4), the job of strengthening the pavement may be accomplished by adding 6/3.1 or approximately 2 in. of epoxy asphalt concrete. Similarly, the strengthening could be accomplished with 6/1.3 or approximately 4.5 to 5 in. of asphaltic concrete. The overlay thickness design could also be accomplished in a similar manner with the method of Palmer and Barber.

Considering all of the properties of epoxy asphalt concrete, the benefits to be gained from the use of an EAC overlay include: (1) added strength and load-carrying capacity with minimum thickness; (2) resistance to shoving, rutting, or indentation under heavy loads; (3) resistance to deterioration by fuels or solvents; (4) excellent tolerance for high temperature jet blast; and (5) a joint-free, smooth riding surface.

ACKNOWLEDGMENTS

The authors wish to acknowledge the valuable contribution of the Manufacturing-Products Application and Research Departments of Shell Oil Company in connection with the field development work. They also wish to acknowledge the assistance of H. Johnson, C.E. Creely, and P.R. Chong in the experimental work.

REFERENCES

1. Simpson, W.C., Sommer, H.J., Griffin, R.L., and Miles, T.K., "Epoxy Asphalt Concrete for Airfield Pavements." Presented at Meeting of Air Transport Division of ASCE, Wash. D.C. (Oct. 23, 1959).
2. Andreason, A.H.M., and Anderson, J., "Concerning the Relation Between Particle Gradation and Interstitial Space in Granular Products." Kolloid Keitschrift, Vol. 51, p. 217 (1930).
3. Nijboer, L.W., "Plasticity as a Factor in the Design of Dense Bituminous Pavements." Elsevier Publishing Company Inc., New York, N.Y. (1948).
4. Campen, H.W., "The Development of a Maximum Density Curve and Its Application to the Grading of Aggregates for Bituminous Mixtures." AAPT Proc., Vol. 11, p. 302 (1940).
5. Hveem, F.N., "Gradation of Mineral Aggregates for Dense Graded Bituminous Mixtures." AAPT Proc., Vol. 11, p. 315 (1940).
6. Metcalf, C.T., "Use of Marshall Stability Test in Asphalt Paving Mix Design." HRB Bul. 234 (1959).
7. Hansche, F.C., "Limitations in the Use of Asphaltic Concrete for High Performance Jet Air Station Pavements." Presented at U.S. Air Force Pavements Conference, Berkeley, California, July 27-31, 1959 (Proceedings to be published).
8. Hveem, F.N., and Carmany, R.M., "The Factors Underlying the Rational Design of Pavements." HRB Proc., Vol. 28, p. 101 (1948).
9. Palmer, L.H., and Barber, E.S., "Soil Displacement Under a Circular Loaded Area." HRB Proc., Vol. 20, p. 279 (1940).
10. Barber, E.S., "Application of Triaxial Test Results to the Calculation of Flexible Pavement Thickness." HRB Proc., Vol. 26, p. 26 (1946).

THE NATIONAL ACADEMY OF SCIENCES—NATIONAL RESEARCH COUNCIL is a private, nonprofit organization of scientists, dedicated to the furtherance of science and to its use for the general welfare. The ACADEMY itself was established in 1863 under a congressional charter signed by President Lincoln. Empowered to provide for all activities appropriate to academies of science, it was also required by its charter to act as an adviser to the federal government in scientific matters. This provision accounts for the close ties that have always existed between the ACADEMY and the government, although the ACADEMY is not a governmental agency.

The NATIONAL RESEARCH COUNCIL was established by the ACADEMY in 1916, at the request of President Wilson, to enable scientists generally to associate their efforts with those of the limited membership of the ACADEMY in service to the nation, to society, and to science at home and abroad. Members of the NATIONAL RESEARCH COUNCIL receive their appointments from the president of the ACADEMY. They include representatives nominated by the major scientific and technical societies, representatives of the federal government, and a number of members at large. In addition, several thousand scientists and engineers take part in the activities of the research council through membership on its various boards and committees.

Receiving funds from both public and private sources, by contribution, grant, or contract, the ACADEMY and its RESEARCH COUNCIL thus work to stimulate research and its applications, to survey the broad possibilities of science, to promote effective utilization of the scientific and technical resources of the country, to serve the government, and to further the general interests of science.

The HIGHWAY RESEARCH BOARD was organized November 11, 1920, as an agency of the Division of Engineering and Industrial Research, one of the eight functional divisions of the NATIONAL RESEARCH COUNCIL. The BOARD is a cooperative organization of the highway technologists of America operating under the auspices of the ACADEMY–COUNCIL and with the support of the several highway departments, the Bureau of Public Roads, and many other organizations interested in the development of highway transportation. The purposes of the BOARD are to encourage research and to provide a national clearinghouse and correlation service for research activities and information on highway administration and technology.