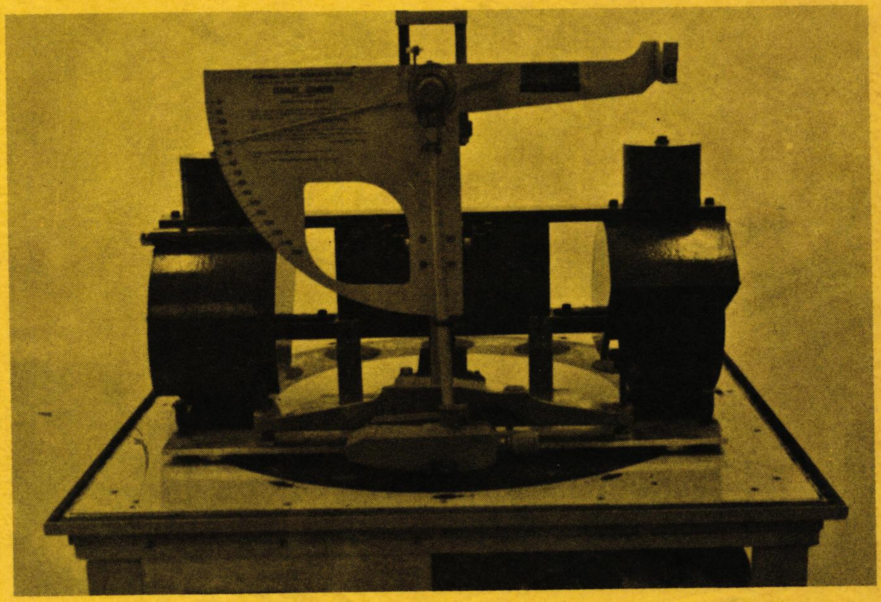


ASPHALT SURFACE DURABILITY AND SKID RESISTANCE

INVESTIGATION

MILLER C. FORD, JR.
CIVIL ENGINEERING DEPARTMENT
UNIVERSITY OF ARKANSAS
FAYETTEVILLE, ARKANSAS

FINAL REPORT
MARCH, 1978

TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle An Investigation of Asphalt Surface Durability and Skid Resistance		5. Report Date March, 1978 6. Performing Organization Code
7. Author(s) Miller C. Ford, Jr.		8. Performing Organization Report No.
9. Performing Organization Name and Address Civil Engineering Department University of Arkansas Fayetteville, Arkansas 72701		10. Work Unit No. 11. Contract or Grant No. HRC 38
12. Sponsoring Agency Name and Address Arkansas State Highway and Transportation Dept. P. O. Box 2261 Little Rock, Arkansas 72203		13. Type of Report and Period Covered Final Report 14. Sponsoring Agency Code

15. Supplementary Notes

This study was conducted in cooperation with the U.S. Department of Transportation, Federal Highway Administration

16. Abstract

This report presents the results of a four-year investigation of the durability and skid resistance of asphalt pavements in Arkansas. The study included field and laboratory evaluation of the polish characteristics and stripping resistance of 18 most commonly used mineral aggregates in Arkansas. The method of polish was accomplished on a small-scale circular wear track that was designed to polish 12 Marshall size specimens at one time, with in-place specimen frictional measurement by the BPT. The laboratory work included tests of aggregate cast in polyester, Marshall specimens, and pavement cores. The asphalt mixtures were evaluated for their Marshall stability and immersion-compression retained strength. The principal findings are summarized below:

1) The field test results indicated a good correlation between SN40 and BPN values.
2) The laboratory tests gave a good correlation between the Texas polish value from the British Wheel and the Arkansas polish value from the accelerated circular track.
3) A good correlation was obtained between the BPN (field) value and the Arkansas polish value (laboratory).
4) Polish tests on cores indicated that most cores had a higher polish value than was shown from the parent coarse aggregate in the Marshall polish test. The difference in these values was attributed to the fine aggregate that was used in the actual paving mixture.
5) The data indicated that the Arkansas accelerated polish test results on asphalt mixtures can be used to estimate minimum SN40 values of the mix in the pavement.

17. Key Words	18. Distribution Statement
Asphalt pavement, Marshall polish value and stability, aggregates, asphalt mixes, pavement cores, polish value, stripping, immersion-compression, circular wear track, SN40 and Arkansas Polish Value.	NO RESTRICTIONS

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
UNCLASSIFIED	UNCLASSIFIED		

Form DOT F 1700.7 (8-69)

AN INVESTIGATION OF ASPHALT SURFACE DURABILITY AND SKID RESISTANCE

by

Miller C. Ford, Jr.

FINAL REPORT

HIGHWAY RESEARCH PROJECT NO. 38

conducted for

The Arkansas State Highway and Transportation Department

in cooperation with

The U.S. Department of Transportation

Federal Highway Administration

MARCH, 1978

ABSTRACT

This report presents the results of a four-year investigation of the durability and skid resistance of asphalt pavements in Arkansas. The study included field and laboratory evaluation of the polish characteristics of the 18 most commonly used mineral aggregates in the state. Field tests on 34 different pavements sections, totaling 212 miles in length, were performed with the locked-wheel skid trailer (SN40) and the British Portable Tester (BPT). The British Pendulum Number (BPN) value was obtained at 60 sites along the study roads. Texture depth at the BPT sites was measured by the sand patch test method. Pavement cores were taken at 32 of the BPT sites for polish and evaluation in the laboratory.

For correlation purposes, the ultimate polish of each study aggregate was determined with the British Wheel test by the Texas Highway Department. The Arkansas method of polish was accomplished on a small-scale circular wear track that was designed and built for the study. This device (APD) was designed to polish 12 Marshall size specimens at one time, the geometry of the device permitting the in-place specimen frictional measurement with the BPT. This laboratory polishing work included testing aggregates cast in polyester, Marshall specimens, and pavement cores.

The asphalt mixtures used were designed to conform to the specifications for the Arkansas Highway Department ACHMSC type 2. Only the coarse aggregate fraction of the mixtures was varied, the fine aggregate fraction (47%) consisted of limestone screenings and a small amount of river sand. The asphalt mixtures all were evaluated for their Marshall stability and immersion-compression retained strength (after vacuum saturation). Each aggregate sample was tested for its resistance to film stripping by the static (140F) and dynamic immersion tests, and the surface reaction test was used for quantitative measurement of surface area stripped. The air void content of the compacted asphalt mixture was found to influence greatly the retained strength as measured by the immersion-compression test.

The polish tests indicated that when sandstone, novaculite, or synthetic aggregate was used in the asphalt mixtures, the APV after polish was less than the APV obtained for the pure aggregate. When limestone, gravel, or syenite aggregate was placed in an asphalt mixture, the resulting APV was higher than that obtained for the pure aggregate. Thus, the APV obtained from tests on aggregate specimens did not indicate the actual polish resistance that was obtained when the material was placed into an asphalt mixture. Laboratory polish tests on pavement cores indicated that for most cores, a higher APV was indicated than their Marshall APV. The difference in these polish values was attributed to the fine aggregate that was used in the actual paving mixture. The data of this study indicate that the results of the Arkansas APD test on asphalt mixtures can be used to estimate the minimum SN40 that would be obtained when the mix is used in a pavement.

GAINS, FINDINGS, AND CONCLUSIONS

The data obtained from this investigation provide sufficient information for adequate study and evaluation of the polishing characteristics of aggregates, asphalt mixtures, and pavement cores. The Arkansas Polish Value obtained in the laboratory cna be used to establish the minimum skid number that will be provided by an asphalt mixture. In conjunction with the minimum desired skid value, SN_{40}, specifications can be established for the polish values of asphalt mixtures or mineral aggregates for use in Arkansas pavements.

The Arkansas Accelerated Polishing Device is an excellent apparatus for polishing test specimens to their minimum friction value. Therefore, the device and associated test procedures are a valuable aid in predicting the wearing properties of aggregates and asphalt mixtures for use in highway pavements.

The polish tests indicate that when sandstone, novaculite, or synthetic aggregate was used in asphalt mixtures, the polish value after polishing was less than the value obtained for the pure aggregate. However, when limestone, gravel, or syenite aggregate was incorporated in an asphalt mixture, the resulting polish value was higher than that obtained for the pure aggregate. The polish value obtained from tests on aggregate specimens does not indicate the actual polish resistance that is obtained when the materials are placed into a highway pavement.

IMPLEMENTATION STATEMENT

The results of this investigation demonstrate the capability of evaluating in the laboratory the ultimate polish value of an aggregate or asphalt mixture in terms of the locked-wheel skid trailer SN_{40} value. The minimum desired pavement frictional resistance in terms of SN_{40} for Arkansas traffic conditions should be determined and the results of this investigation should be implemented by specifying the asphalt mixture components to attain the required skid resistance.

All types and gradations of asphalt mixtures used or to be used in Arkansas pavements should be evaluated by the Arkansas Accelerated Polishing Device test procedure as reported herein. The implementation of this suggested procedure will ensure that the asphalt mixtures used are the best available and that they will be satisfactory for the purpose for which they are intended.

The practical application of the results of this investigation would be enhanced by an evaluation of the relationship between traffic and the number of revolutions of the Arkansas Accelerated Polishing Device. The relationship between the hours of laboratory polish and traffic (total number of wheel passes or AADT) would enable the design engineer to specify the pavement materials necessary to provide the skid resistance for that particular highway traffic. The Arkansas Accelerated Polishing Device and test procedure then can be used to pre-evaluate the asphalt mixture proposed for use to ascertain that the desired long-term skid resistance is provided.

ACKNOWLEDGEMENTS

The author expresses his thanks to Professor J. R. Bissett, who was co-principal investigator until his retirement in 1975, for his guidance in this investigation. Acknowledgement is also given to the many Civil Engineering undergraduate students who helped in performing the laboratory work, and to the following graduate students who assisted in carrying out this work: Wendy Jacks, Paul Gunn, Bert Parker and Jeff Richards.

Appreciation is extended to the Arkansas Highway Department personnel who assisted in the field work, some of whom are: Jake Clement, Bob Kessinger, Jerry Rogers, Jerry Westerman, Larry Spencer, Jessie Meabon, Grady Williams, Sam Smith, Calvin Peevy, Charles Mauk, Ken Harris, D. M. Green, James Earnhart, Norman Cobb, Bobby Hughes, Bill Wood, J. L. Owens, Frank McCollough and Mack Neal.

Larry Walker and Tom Patty of the Texas Highway Department helped in performing the correlation work with the British Wheel tests, and Willard McCasland of the Oklahoma Highway Department provided help in the aggregate casting and tips on the British Portable Tester operation.

Special acknowledgement is given to Cathy Morrow and Sylvia Glezen for their help in typing the report.

TABLE OF CONTENTS

Chapter | Page

Chapter Page

LIST OF TABLES

Table		Page

LIST OF FIGURES

Figure | Page

Figure | Page

Figure		Page

Figure | Page

Figure		Page

CHAPTER I

INTRODUCTION

The purposes of this investigation are to measure the rate of wear of Arkansas aggregates and asphalt mixtures by use of an accelerated polishing machine; to determine the feasibility and economy of using some locally available aggregates with imported aggregates of high quality to produce hot-mix asphalt concrete mixtures; and to improve skid resistance by blending high quality, polish resistant, coarse aggregates with local, less polish resistant aggregates to produce hot-mix asphalt concrete pavements. Further, because obtaining a good skid resistant asphalt pavement has no merit unless the mixture has sufficient durability to give satisfactory performance, the stripping resistance of the aggregate and asphalt mixtures is evaluated.

The skid resistance of highway pavements is one of the greatest problems facing today's highway engineers and researchers. The early detection of low skid resistance areas, combined with a program to pre-evaluate a pavement's polishing characteristics before actual construction, is desirable if engineers are to maintain feasible and safe operational highways.

Because of increased legal duties imposed on public entities by the judiciary system, highway departments have been subject to liability for accidents that result from what once were considered purely weather-related causes. Unfortunately, nearly all pavement surfaces that are economically feasible to construct will lose their initial high skid resistance with exposure to heavy traffic.

Skidding accidents can be caused by vehicle factors, the driver variables, or the pavement frictional properties. Though many factors contribute to skidding accidents, this investigation is concerned with those primary factors affecting asphalt pavement friction in Arkansas. Most accidents attributed to low pavement friction are observed to occur on wet pavements. Thus, accidents that occur during wet weather can be prevented if a good skid resistant pavement is provided. Further, accidents that occur under dry weather conditions also may be related to low pavement friction.

The problem of pavement skid resistance has been under investigation for several years. Several different devices and techniques for measuring the skid resistance of a pavement under wet conditions have been established. Two of the machines being used more widely each year are the locked-wheel trailer and the British Portable Tester. Various laboratory techniques have been developed to accelerate and measure the wearing and polishing characteristics of different pavement materials.

Field and laboratory tests have led to a variety of approaches to the solution of the skid resistance problem. Some of the solutions that have been proposed are: 1) specifying a minimum polish value for the aggregate based on laboratory tests, 2) mixing hard and soft aggregate together, 3) using construction techniques that give the pavement surface a coarse texture, and 4) not using polish susceptible aggregate.

It is an accepted fact that in an asphalt pavement, the larger aggregate in the mix mainly controls the skidding frictional resistance of the pavement. The extent to which the frictional characteristics are changed by weather, traffic, or other mixture ingredients must be determined in order to provide skid resistant pavements.

A research project was designed to investigate the materials commonly incorporated into asphalt pavements by the Arkansas Highway Department (AHD). Because the coarse aggregate fraction of an asphalt mixture contributes most of the pavement's friction, the study is limited to evaluation of this size material. Further, because most pavement mixtures used by the AHD are classified as asphalt concrete hot-mix surface coarse type 2, only this particular gradation is studied. To make the fine aggregate contribution to the skid resistance minimal and constant for all mixtures, the screening from a local limestone quarry along with available river sand is used. The asphalt cement used in the laboratory mixtures represents that which is predominantly used in Arkansas. The same asphalt cement sample is used in all the specimen preparation. The foregoing procedures ensure that any change in skid resistance or durability properties of different mixtures is attributable to the coarse aggregate.

To accomplish the stated purposes, the project work was scheduled to be performed as follows: 1) determine the sources and obtain samples representative of aggregate used in Arkansas pavements, 2) develop laboratory methods and procure equipment needed to perform tests to evaluate the durability and friction characteristics of the aggregates, both as separate particles and when used in asphalt mixtures, 3) evaluate the actual field performance of the aggregates by testing existing pavements constructed from these aggregates, 4) relate the laboratory test results to the actual field performance results obtained in step 3, and 5) evaluate combinations of polish susceptible aggregates and polish resistant aggregates to provide mixtures with more enduring skid resistance.

The initial proposal to accomplish the purposes of the investigation was modified to gain approval of the funding agency. The proposed method of polishing was changed from use of an accelerated circular track device to utilization of a British Wheel. After more than 12 months of delay due to nondelivery of the British Wheel, the project work plan was revised to permit local design and construction of the accelerated polishing machine.

A limitation of $2000 was placed in the maximum cost of the accelerated polishing machine by the Arkansas Highway Department. It was further stipulated that the polish test results to be obtained with the accelerated polishing machine had to be correlated with test results obtained by polish on a British Wheel.

The investigative work required close cooperation with AHD personnel. The initial work consisted of selecting the mineral aggregate samples that were representative of Arkansas aggregate. A total of 17 aggregate sources were chosed. They are all approved sources of aggregate that had been used in asphalt pavement construction by the Arkansas Highway Department. The general classification and number of each type of aggregate are: limestone (7), sandstone (5), gravel (3), syenite (1) and novaculite (1).

One additional aggregate was selected later for a total of 18 aggregates studied. It is a synthetic aggregate made from expanded clay. This synthetic material had only been used successfully for surface treatments at that time, but it was deemed appropriate for study because AHD skid trailer tests indicated that it provided a high skid resistance surface.

The standard test methods employed were those used by the AHD Division of Materials and Tests or the standard methods published by the American Society for Testing and Materials (ASTM). Special tests were devised on the basis of the experience of other researchers. The pavement frictional resistance was evaluated with the AHD locked-wheel skid trailer and a British Portable Tester, which is a device that was satisfactory for both field and laboratory friction measurements.

The British Wheel tests were performed on all of the study aggregates by the Materials and Test Division of the Texas Highway Department in their Austin, Texas, laboratory. As part of this correlation work, the Texas Highway Department also furnished British Wheel specimens with known polish values for checking the operation of the British Portable Tester.

The laboratory polishing tests of aggregate and asphalt mixtures were accomplished on a small-scale circular wear track. This device was designed to polish 12 Marshall size specimens at one time, the geometry of the device permitting the in-place specimen frictional resistance measurement with the British Portable Tester. The actual asphalt mixture, as designed by the Marshall method, was then tested for its polishing characteristics, as well as its stability, flow, and immersion-compression strength.

In addition to immersion-compression tests of Marshall specimens, each aggregate was tested for its resistance to film stripping by the static immersion and dynamic immersion tests. The quantitative evaluation of the amount of film stripping was measured by the surface reaction test.

Cores from selected pavement sections were taken and polished for correlation with the results of the laboratory polish test on the Marshall specimens. In the last phase of the project, different types of aggregates were blended together to determine whether they would provide a more polish resistant mixture.

The work planned included correlating the test results from this investigation with those obtained by the AHD from their large circular track testing apparatus (HRP No. 40). No correlation was possible because no test results were available from HRP No. 40 prior to the completion of this report.

The results of the laboratory and field investigation will be a valuable asset for pre-evaluating the wear and skid resistance properties of the different types of aggregates and their asphalt mixtures that are used in Arkansas. The results can provide new insight on the design of asphalt pavements by allowing the highway engineer to be aware of the different aggregate-asphalt mix characteristics in relation to their skid resistance and durability properties.

CHAPTER II

REVIEW OF LITERATURE

In 1853, during the early years of American road-making, Professor Gillespie (1)[1] presented criteria for the quality of stone to be used in roads. In part he said:

> Flint or quartz rocks, and all pure silicious materials are improper for use, since though hard, they are brittle, and deficient in toughness. Granite is generally bad, being composed of three heterogeneous materials, quartz, feldspar, and mica; the first of which is brittle, the second liable to decomposition, and the third laminated. The sienitic granites, however, which contain hornblende in the place of feldspar, are good, and better in proportion to their darkness of color. Gneiss is still inferior to granites, and mica-slate wholly inadmissible. The argillaceous slates make a smooth road, but one which decays very rapidly when wet. The sandstones are too soft. The limestones of the carboniferous and transition formations are very good; but other limestones, though they will make a smooth road very quickly, having a peculiar readiness in 'binding', are too weak for heavy loads, and wear out very rapidly. In wet weather they are also liable to be slippery. It is generally better economy to bring good materials from a distance than to employ inferior ones obtained close at hand.

The major consideration in road building at that time was to reduce the rolling friction of the wagon wheels moving over the various types of surfaces. Gillespie also noted that the friction on all types of road surfaces was different, and could only be determined by experiment. He described a "Dynamometer" as the instrument used for measuring friction at that time.

The present-day highway engineer is still concerned with the problem of pavement friction and durability and needs to know which

---

[1]The number in parentheses corresponds to the listing of the literature cited in the Reference section.

material will perform satisfactorily. The fact that "limestones" are likely to be slippery when wet has been known for more than one hundred years. Some limestones are better than others, however, and researchers are still trying to determine which stone will provide the best pavement, when the economy factor is to be considered.

Skid Resistance

Historical Perspective

The evaluation and corrective procedures associated with skid resistance have concerned highway engineers for many years. In the United States, horse-drawn vehicles were used as early as 1896 by Captain Green to aid in observing and evaluating different types of pavement (2). "Slipperiness", as it was called, was of significance both to the foothold capability of horses and the traction associated with the increasing motor-driven traffic (3).

Most sheet pavements, as well as wood-block and to some extent stone-block pavements, became slick and dangerous when wet or frosty. Soon it became obvious that storm water runoff was a major design parameter of hardened pavements, and engineers began "crowning" roads to aid in the proper disposition of storm water. As this art was developed, the sloped pavements of turns and intersections were designed to combat frictional loss due to the vehicles' forward inertia while turning (3).

Research began in the mid-1920s to investigate possibilities of providing all-weather skid-safe travel for the motoring public. Agg, and later Moyer, were the forefathers of this research. Their goal was expected to be accomplished within a short period of time (2).

Five decades have since passed, however, and because of the continuing evolution and changing maneuverability of the automobile, the goal of skid-safe roads is more elusive than ever.

It was not until the 1950s that the causes of slipperiness, its measurement, and its effect on the safety of vehicular traffic were recognized as matters of great concern (4). The highway engineer, having little or no control over driver education or tire manufacturers, concentrated his efforts on the measurement of skid resistance and the development of corrective measures for inadequate pavement surfaces (2).

For 10 years between 1948 and 1958 Dillard (5), of the Virginia Department of Highways, used the latest model cars and tires in the stopping-distance method of skid resistance measurement. With each new car and tires used, Dillard found that he could not correlate the old data with the new data.

The Michigan State Highway Department (6) started its skid testing also in 1947-48 with a skid trailer pulled behind a dump truck equipped with water tanks for wetting the pavement. They too found a lack of correlation with each new set of new improved tires.

Of particular significance in the history of skid resistance research was the First International Skid Prevention Conference in September 1958. The objectives of the conference were broad in scope. They included the correlation of stopping-distances for vehicles, the dissemination of technical knowledge, and the determination of gaps in research. Probably the most significant findings at the conference were the lack of aggreement among and between the data obtained from skid trailers and the influence of the water film thickness on the skid numbers measured in the field (2).

The Tappahannock Skid Correlation study (2) in the fall of 1967 also showed that the problem with correlation between skid measuring devices was due to water film thickness and the use of new improved tires. From work accomplished at Purdue University, six out of every seven accidents in wet weather were shown to be due to skidding. Thus, the skid resistance measuring equipment and procedures were critically appraised, evaluated, and standardized as a result of these first conferences. The solution to the problem of skidding accidents was not found, however, and it remains one of the major goals of the highway engineer today.

Nature of Skid Resistance

Lack of skid resistance today is due mainly to wet pavement. Dry pavements nearly always have a high skid resistance.

Two components of friction develop between the tire and pavement: adhesion and hysteresis. The adhesion component of friction has been shown to be speed dependent, whereas the hysteresis component is relatively independent of speed except at very high speeds when it decreases as the tire gains heat. The adhesion component generally is considered to be the product of the interface shear strength and the actual contact area of the tire and pavement. The hysteresis component is caused by damping losses within the rubber when the tire rubber is passing over and around the mineral particles. Mullen (7) found that the adhesion component is the result of molecular forces (shear) developed at the tire-pavement interface, whereas the hysteresis component is a function of the energy losses within the tire rubber as it is deformed by the asperities of the pavement surface.

Kummer and Meyer (4) show pictorially how these components are determined in Figure 1.

The effective skid resistance force is the sum of the adhesion force, F_a, and hysteresis force, F_h. It is customary, though, to present the measured friction force (adhesion and hysteresis components) as a dimensionless "coefficient" defined as the ratio of force components opposing the sliding of the rubber-surface contact area (4). Hence,

$$f = \frac{F}{L} = \frac{F_a + F_h}{L} = f_a + f_h$$

where:

f = effective measured friction coefficient

F = measured friction force

L = slider or wheel vertical load

F_a = friction due to adhesion component

F_h = friction due to hysteresis component

f_a = friction coefficient due to adhesion

f_h = friction coefficient due to hysteresis

Parameters

To examine properly the problem of skid resistance, one must have a good understanding of the primary parameters that influence skidding. The question of exactly how many factors affect pavement friction is pointless, because their number can be greatly increased by adding affects of the effects. Csathy et al. (8) list the three primary parameters as:

(a) those associated with the pavement surface aggregate characteristics and surface texture,

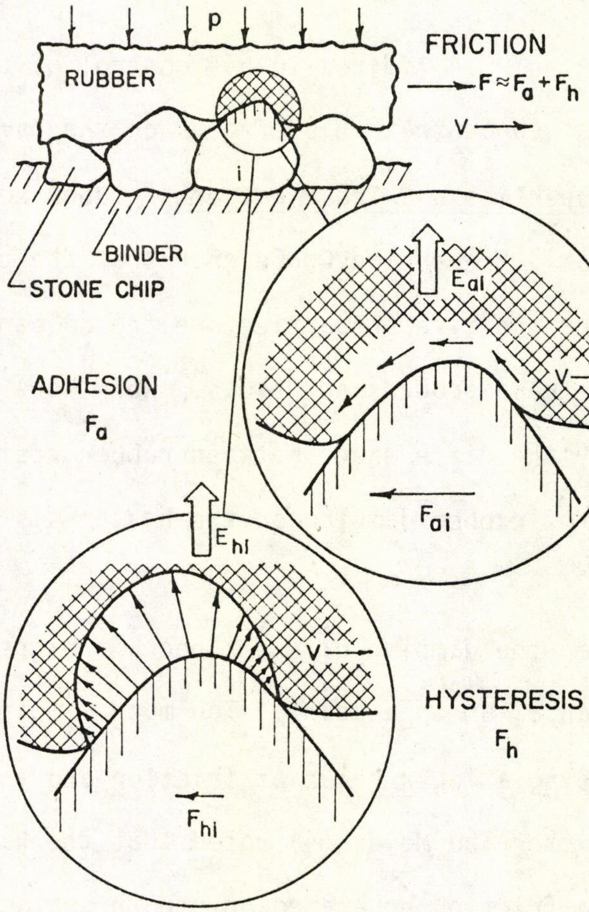


Figure 1. The Two Principal Components of Rubber Friction: Adhesion and Hysteresis (4).

(b) the vehicle-rubber properties and tread pattern, and

(c) those caused by the operating conditions - vehicle speed and temperature.

The highway engineer is limited in his control of the last two parameters, and must concentrate his efforts on the pavement surface.

Rubber Properties and Tread Pattern. Tabor reported to the First International Skid Prevention Conference that the primary factors involved in rubber and tire properties are the rubber damping and elasticity and the tread grooves and slots. Work done after World War II helped identify the differences between rubber resilience and friction and the effect of rubber damping on the hysteresis component of friction.

The greater the damping of the rubber, the greater the increase in skid resistance, up to a point. Too much damping may generate too much heat, causing a lower value of friction and making the rubber seem stiff. Kummer and Meyer (4) noted that the harder the rubber, the greater the friction generated on smooth surfaces but the lower the friction value on textured surfaces. The change in slip and skid resistance due to an increase in rubber hardness and damping and the addition of tread grooves and slots varies with different surface types.

Tread design is important in skid resistance, especially when the pavement becomes wet. Tread designs are constantly being upgraded by the tire manufacturers to remove the layer of water between the tire and pavement more quickly. Many states require, by law, that a vehicle tire must have a certain amount of tread.

Vehicle Speed and Operative Conditions. Speed is the most variable parameter under the driver's control. Now that highway speeds have been reduced to 55 mph to conserve energy, the number of high-speed accidents has decreased slightly. Wet skid accidents are considerably reduced with a reduction in speed. Unfortunately, lack of adequate detailed information from the authorities investigating traffic accidents precludes a quantitative expression of this effect.

Aggregate Characteristics

The primary function of the pavement surface in providing skid resistant properties is to maintain effective instantaneous drainage in the contact region (adhesion) and to provoke high hysteresis losses in the tire rubber (deformation). The aggregate in a paving mix is crucial in developing these forces, and must maintain the properties of being textured, angular, hard, and polish-resistant under wear (9). Research by Burnett et al. (10) has shown that the type coarse aggregate and gradation in a bituminous mix is most influential in contributing to skid resistance. They also state that the coarse aggregate should be angular and resist wear so the pavement can have good water drainage and hysteresis effects or should contain adequate microtexture to replace any loss of angularity.

Some states (11, 12) have had problems with adequate skid resistance on pavements where a high carbonate aggregate, such as limestone, was used. Sherwood and Mahone (11) concluded from insoluble residue tests of carbonate aggregates (limestones) that the amount of acid-insoluble mineral grains contained in limestone is primarily responsible for their variable wearing characteristics. The insoluble residue test is a procedure used to separate the aggregate into a carbonate fraction and a non-carbonate fraction. The test is based on the chemical reaction that occurs when dilute hydrochloric acid is allowed to react with the carbonate portion of the aggregate. A leaching process dissolves the carbonate fraction in the form of a residue.

The results of Sherwood and Mahone's (11) tests, which compared the average frictional values obtained with a skid trailer going 40 mph (SN_{40}) with the percentage of the aggregate's insoluble residue, are shown in Table I. They classified SN_{40} values from good to critical as the skid number decreases from 53 to 33. The results indicate that as the percentage of insoluble residue increases in aggregate, the skid resistance increases. Grambling (12) and Underwood et al. (13) concluded from laboratory research that relatively pure carbonate aggregates polish uniformly and become slippery, but as the amount of insoluble sand-size material within the aggregate particles increases, the skid resistance properties increase.

Mullen et al. (14) show that the skid resistance of a certain aggregate increases with the aggregate hardness up to a certain point as in Figure 2. Then, as the aggregate becomes harder, the skid resistance decreases. Other factors they found to improve skid resistance within an individual aggregate particle are mineral grain shape, size, and distribution. Beaton (9) and Mullen et al. (14) agree that if an aggregate is to retain non-skid properties under prolonged traffic, the aggregate should contain at least two mineral constituents that are considerably different in their resistance to wear.

More recent research by Dahir and Meyer (15) indicates that aggregates with coarse grained rock surfaces require more polishing effort than those with fine grained surfaces before maximum polishing is achieved. This effect is believed to be due to the flattening or

Table I

Rating of Insoluble Residue and

Coefficient of Friction Values (11).

Rating	Average SN_{40}[1]	Average IR[2]%
Good	53	37
Fair	44	13
Poor	41	9
Unacceptable	38	8
Critical	33	5

[1] SN_{40} - Skid Trailer results at 40 mph.

[2] Insoluble Residue Results.

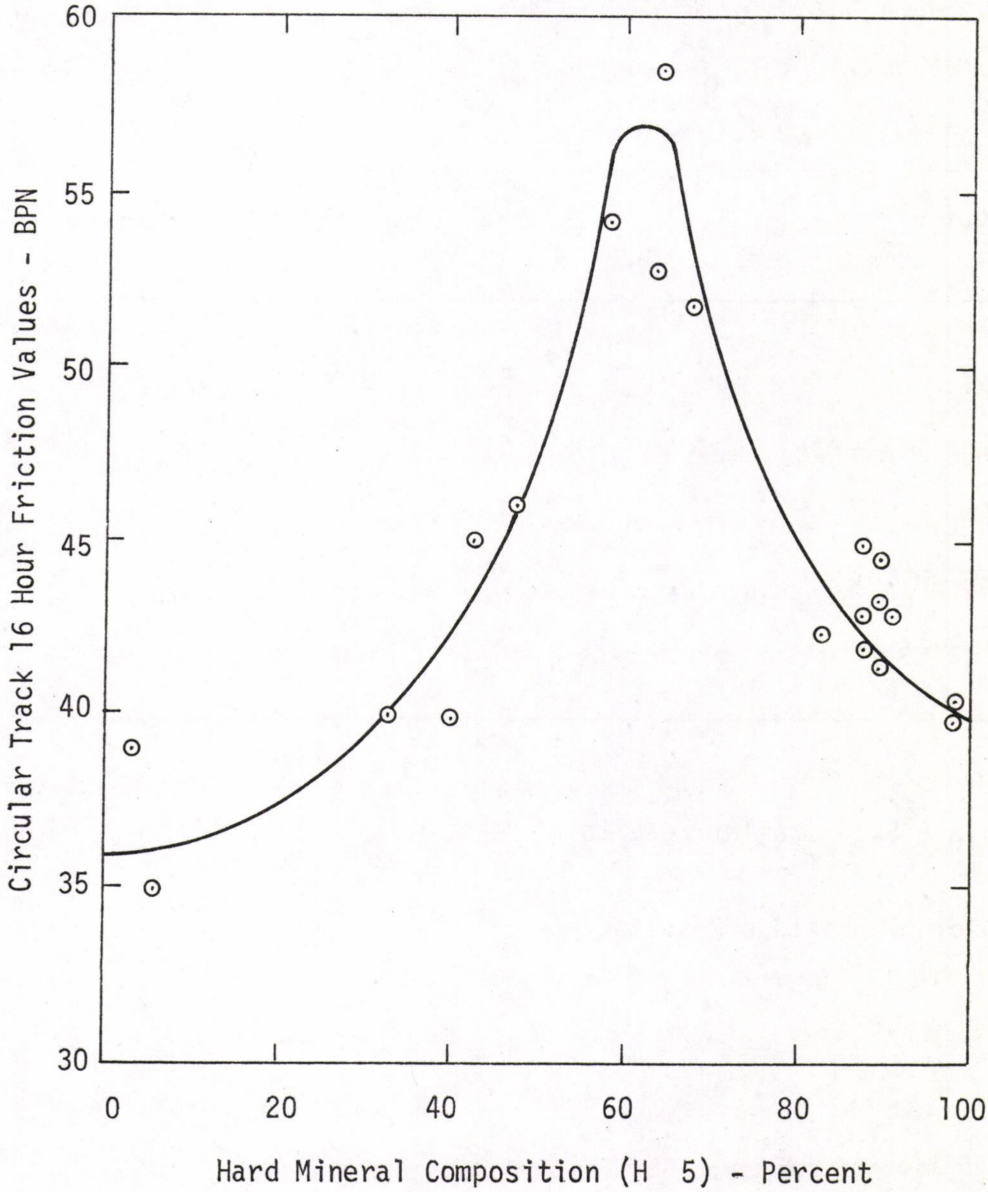


Figure 2. BPN Values Versus Hard Mineral Content (14).

rounding of coarse grains before finer polish commences.

Surface Texture. Surface texture is beneficial to the generation of friction and provides a means by which the water can be channeled off the pavement without a loss of contact between the tire and pavement (hydroplaning) (16). The texture of a pavement cannot be described by a single characteristic. A distinction must be made between macrotexture (profile) and microtexture (degree of sharpness) of the pavement surface (13).

Macrotexture refers to the pavement surface as a whole or the large-scale texture produced by the size of the surface aggregate. The primary function of macrotexture is water drainage from the tire-pavement interface. This effect becomes more pronounced as the vehicle speed increases, tire tread depth decreases, and water depth on the pavement increases. In addition, macrotexture causes tire wrinkling which performs as an energy absorber in rolling, slipping, and sliding. Macrotexture brings into play the hysteresis properties on the tire rubber and increases the ability of the tire tread to remove water dynamically from the tire-pavement interface (17).

Microtexture refers to the fine-scaled roughness contributed by the small individual asperities of the individual aggregate particles (17). The microtexture affects the contact area and shear strength of the adhesion component and hysteresis losses. The microtexture of paved surfaces changes throughout the year from smooth during late fall to rough during early spring. These seasonal variations were noted in skid resistance research conducted by Giles et al (18) and Kessinger and Nielson (19). Microtextures also was found to change during the beginning and after the end of a period of rain, and thus gives rise to daily skid resistance fluctuations as observed by Kummer and Meyer (4).

Grambling (20) reports that in Pennsylvania the pavement skid friction values go through an annual cycle. The skid numbers measured between June 15 and November 15 are considered to be the nominal minimum skid values. The variation in skid numbers approximates a sine curve, with the amplitude dependent on coarse aggregate characteristics and traffic volume.

The critical period for pavement slipperiness is after the surface is wetted. As the vehicle velocity increases, if the water on a pavement surface does not have time to be squeezed away, tire hydroplaning occurs. This hydroplaning or aquaplaning is caused by the fluid pressures that develop at the tire-pavement interface. When these pressures become large and the complete hydrodynamic force on the tire from these pressures equals the total load the tire is carrying, hydroplaning occurs. At this instant, theoretically the tire completely loses contact with the pavement and begins to skim over the surface (21). Beaton (9) explains that the important function of texture is to provide drainage channels by which water can escape from under the tire at faster vehicle speed. This effect allows the tire tread elements to expel any remaining water and make positive contact with the pavement surface.

Beaton (9) reported the manner in which pavement surface texture characteristics affect skid properties, as illustrated in Figure 3. The pavement's wet-frictional properties, shown as a skid number, vary with the pavement texture and vehicle speed. The skid number (SN) is discussed in detail elsewhere in this report.

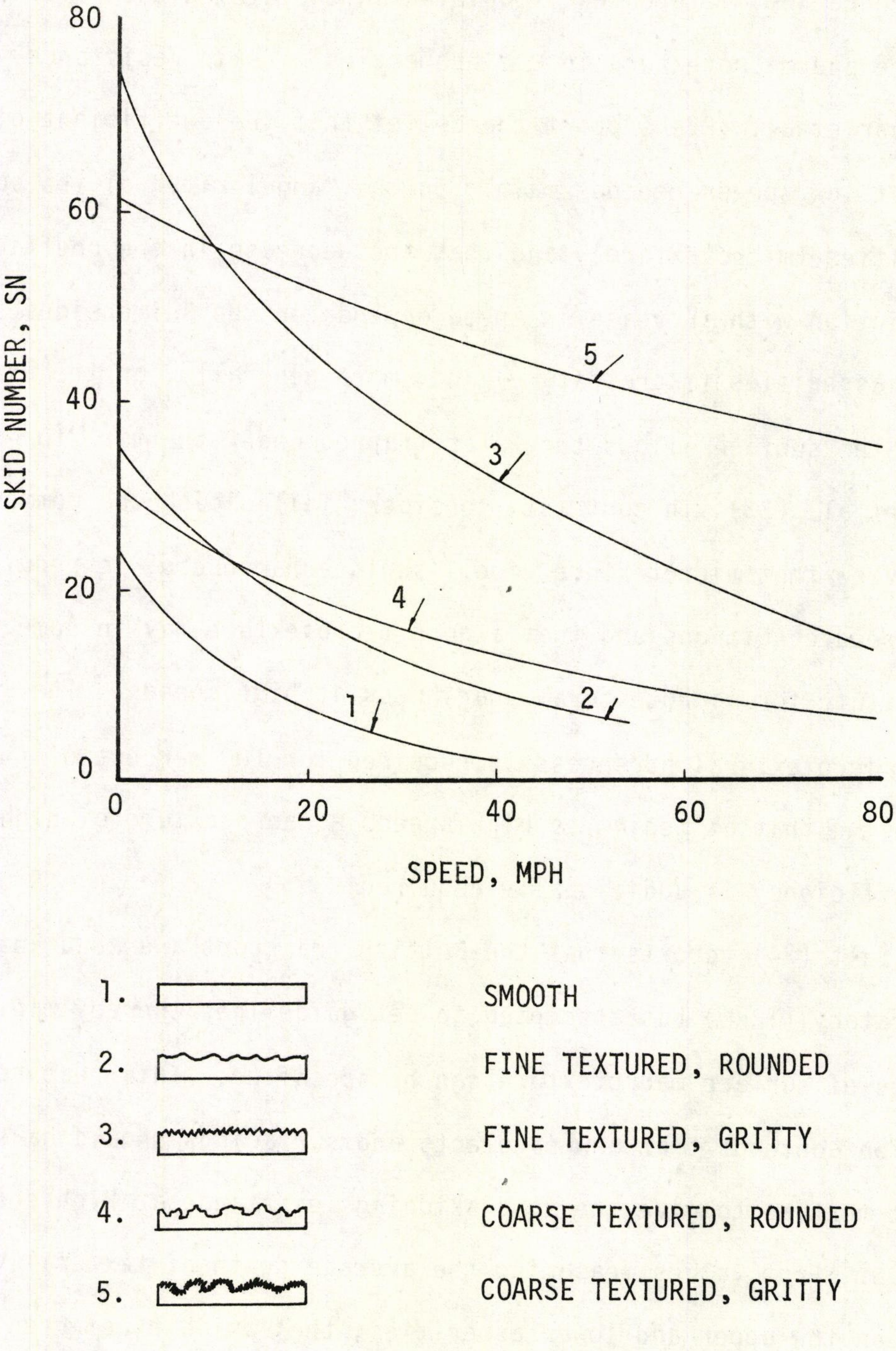


Figure 3. Classification of Pavement Surfaces According to Their Wet Friction and Drainage Properties (9).

There are differences of opinion in explaining the roles of microtexture and macrotexture in the properties of skid resistance. Elsenaar et al. (22) support the belief that the coefficient of friction at low speeds depends mainly on the "angularity" of the surface asperities (microtexture), and that the decrease in the coefficient of friction with increase in speed depends on the "dimensions" of these asperities (macrotexture) and, more generally, on the extent to which the surface allows the water trapped under the tire to escape. Lees et al. (23), in contrast, consider "fallacious" the commonly held view that microtexture, i.e., surface harshness, is required for low speed conditions and that a good macrotexture may in some sense substitute for microtextural harshness at high speeds. They contend that microtextural harshness is required for 100 percent of the speed range, and that at medium to high speeds a macrotexture of high drainage efficiency is additionally required.

Salt (24) reports that the British Transport and Road Research Laboratory (TRRL) has attempted to set guidelines whereby minimum values of surface macrotexture can be specified. This research organization contends that new pavements and surfacings should have sufficient texture to give the same skidding resistance at high speeds as at low. From values measuring the average depth of textural voids between the upper and lower asperities, the variable referred to as texture depth (T_D) can be found. TRRL recommends that maintenance intervention should take place when texture is reduced to the point at which a 20 percent drop in skid resistance occurs when vehicle speed is changed from 30 to 80 mph. The effect of macrotexture on the change in skidding resistance with speed in shown in Table II. The

Table II

The Effect of Macrotexture on the

Change in Skidding Resistance with Speed (24)

Drop in skidding resistance with speed change from 50 to 130 km/h (30 to 80 mph) %	Texture Depth (mm)	
	Flexible	Concrete*
0	2.0	0.8
10	1.5	0.7
20	1.0	0.5
30	0.5	0.4

* when textured predominantly transversely

TRRL conditions require a 2.0 mm (0.08 in.) texture depth for new bituminous surfacings, with maintenance being required as the texture depth approaches 1.0 mm (0.04 in.). Rose and Galloway (25) also support a minimum macrotexture value of about 1.0 mm (0.04 in.).

Measurement of Surface Texture. Several methods have been devised in an attempt to evaluate pavement surface macrotexture characteristics. Currently in use are the profilograph, the texturemeter, the laser, photogrammetric tests, the tire-noise test, regular and modified sand patch methods, the putty impression method, and the outflow meter test.

The profilograph, developed by the Pennsylvania Transportation Institute (26), is designed to scribe a magnified profile of the surface texture as a probe is drawn across a surface. The vertical movement of the probe is magnified through a linkage system and a magnified duplication of the texture profile is scribed on a chart recorder.

The texturemeter is a hand-operated device used to measure the coarseness of the texture of pavements. Developed by F. H. Scrivner of the Texas Transportation Institute (27), the texturemeter consists of a series of evenly spaced, parallel vertical rods mounted in a hollow frame. The metal rods can move individually in a vertical plane against a light spring pressure. A wire is connected at one end of the frame, passes through holes in the upper end of the rods, and is attached to the spring-loaded stem of a 0.001-in. dial gage mounted on the opposite end of the frame. As the frame is pushed down against the surface, any irregularities in the surface cause the wire to form a zig-zag line which will prevent the dial gage from resuming its straight-wire zero reading. The coarser the pavement texture, the more deformities will take place in the wire and the higher will be the dial reading.

The laser method utilizes a light beam and a receiver. As explained by Gee et al. (28):

> Light is emitted from the laser and is incident on the pavement surface. The light reflected from the surface is generally scattered in all directions. The... linearly polarized light...will experience "depolarization", where reflected light is no longer linearly polarized, but is elliptically polarized... . The degree of ellipticity is a function of pavement surface characteristics and can be determined from the two perpendicular...axes of the polarization ellipse.

The photogrammetric method of measuring texture is similar to aerial photography surveying in that coinciding photographs are used together in a stereophotographic technique. The pavement stereophotographs have horizontal and vertical scales located in the picture, and are viewed under a mirror-stereoscope or a microstereoscope. The pavement is then classified by a comparison chart and assigned a textural code number according to its surface roughness (29).

The tire noise concept was first investigated by the Pennsylvania Transportation Institute in 1972 when field testing suggested that texture-related factors influence tire noise. Macrotexture can be analyzed by mounting a microphone with a windscreen near a standard, full-scale, treadless tire and carefully measuring the noise output (30).

The regular sand patch test is widely used in the measurement of texture depth because of the simplicity and availability of the necessary equipment. The test itself consists of pouring a known volume of sand on the clean surface of pavement. Next, with a 6½ cm diameter hard rubber disk mounted on a flat wooden disk of the same size, the sand is spread over the surface by working the disk with a flat circular motion. The sand is spread in a circular patch until the surface depressions are filled to the level of the peaks as shown in Figure 4. By measuring the diameter of the patch and knowing the volume of sand needed to form the circle, one can obtain the average texture depth (T_D) by the equation:

$$T_D = \frac{V}{R^2}$$

where:

T_D = average texture depth

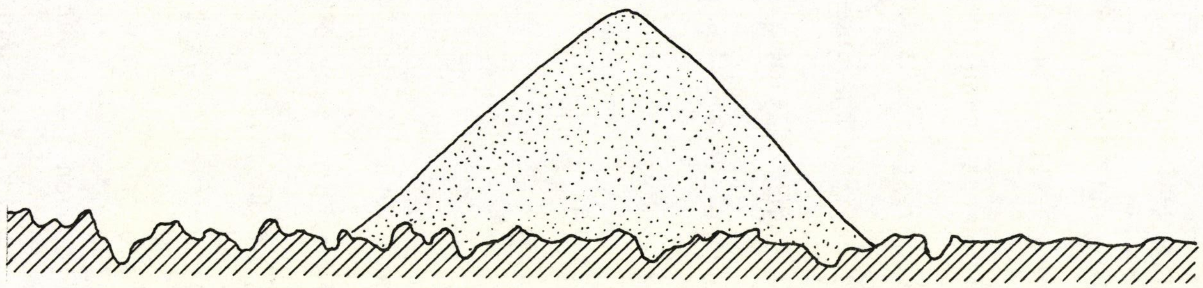
V = volume of sand

R = radius of the sand patch

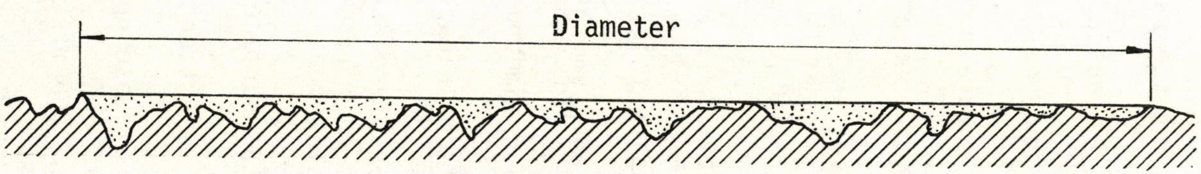
The modified sand patch test incorporates the same basic idea as the regular sand patch test, i.e., Surface Area X Average Depth = Volume, but requires more equipment. To more accurately define the surface area of sand, a rectangular metal plate with a rectangular hole of a specified size, or a rectangular hard rubber plate with a circular hole of a specified size, is used. The plate is set on the surface of the pavement and the volume of sand required to fill the cavity is measured. By calibrating the device on a flat surface with no texture, the volume of sand required to fill the textural voids of the specified area can be computed. The deeper the median texture depth, the more sand will be needed to fill the cavity (31).

The putty impression test is also similar to the regular sand patch test. A ball of silicone putty is placed on the surface and a metal plate with a specified flat, circular, 4-in.-wide indention is used to flatten the putty. The indention is such that when the putty is flattened on a textureless surface, it will form a flat, circular shape 4 in. in diameter. The more irregular the surface macrotexture,

(i) Known volume of fine sand of uniform particle size poured on road



(ii) Sand spread to form circular patch with 'valleys' filled to level of 'peaks'



(iii) Texture depth = $\frac{\text{Volume of sand}}{\text{Area of patch}}$

Figure 4. Regular Sand-patch Method of Measuring Texture Depth

the smaller the resulting putty diameter will be because more material will be required to fill surface voids.

The outflow meter test is performed with a round cylinder device with two reference marks approximately 4 in. apart on the cylinder wall. A neoprene gasket is attached to one end of the cylinder, separating the cylinder and the base from the pavement surface. The surface to be tested is pre-wetted with water for one minute, after which the device is placed on the wet surface and filled with water. The water is permitted to drain out of the cylinder through the voids of the pavement surface beneath the neoprene gasket. The time required for the water level in the cylinder to drop from the upper reference line to the lower line is recorded. The length of time required for the outflow is inversely proportional to the average texture depth of the voids. In using this approach, one assumes that none of the water penetrates the pavement and flows laterally within it as might occur with an open graded surface course (31).

Asphalt Mixture Characteristics

Surfaces of roadways are designed to provide stability, durability, skid resistance, and rider comfort, plus many other related items. Primarily, the surface requirements of durability and skid resistance control the selection of the asphalt mixture to be used. These two requirements impose opposite demands on the mixture characteristics. Durability of the mixture is achieved when the asphalt content is high and the void content is low, whereas skid resistance is achieved when the smallest practical amount of asphalt cement is used and a high void content is provided.

Asphalt Concrete. Asphalt concrete mixtures can be classified as a dense graded or an open graded mix. Typical aggregate gradations from several state highway department agencies are shown in Table III (32, 33, 34, 35). The dense graded mixtures are similar except for the North Carolina Type I-2, which is finer graded. The coarse aggregate fraction (plus No. 10 material) of the Arkansas, Pennsylvania and Louisiana mixes constitutes, respectively, 57, 58, and 56 percent of the total aggregate in the mix. The optimum asphalt content of these asphalt mixtures ranges from 5 to 7 percent, and desirable air void content ranges from 3 to 6 percent. They are considered to provide a stable, durable, and smooth riding surface. Their skid resistant properties are variable, depending on the many factors previously discussed.

Plant Mix Seals. Open graded asphalt friction courses, also called plant mix seals (PMS), consist of high-void asphalt mixtures placed on existing pavement surface in thin layers. The advantages attributed to these mixtures during wet weather include: improved skid resistance at high speeds, minimization of hydroplaning effects, minimization of splash and spray, and improved night visibility (36). Other benefits predicted from use of these friction courses are: improved road smoothness, minimization of wheel path rutting, and lower highway noise levels. However, early research reports indicate very little difference in measured skid resistance between dense graded asphalt mixtures and open graded asphalt friction courses when similar materials are used. Adam and Shah (35) reported on a gravel asphalt mixture in Louisiana, after four years of heavy traffic, where the skid resistance of the PMS was 43 and the conventional dense graded mixture skid resistance was 41. Kandhal et al. (34) reported

Table III

Surface Course Mix Gradation from Different States

Gradation % Passing	Arkansas Dense (Type 2)	Pennsylvania Dense (Id-2)	Pennsylvania PMS*	North Carolina Dense (I-2)	Louisiana Dense	Louisiana PMS*
3/4 in.	100	100	100	100	100	100
1/2 in.	88	100	100	100	98	100
3/8 in.	80	91	100	95	86	98
No. 4	60	66	31	80	59	46
No. 10	43	42	14	55	44	13
No. 40	23	18	8	27	30	4
No. 80	11	8	5	14	13	-
No. 200	5	4	2	4	9	1
Reference	32	28	29	7	30	30

* Plant Mix Seal - (open graded friction course)

on a trial PMS in Pennsylvania which had a friction value of 64 after 18 months, whereas the standard dense graded mixture had a skid resistance of 62. They also noted that the PMS had considerably greater surface texture depth at 0.09 in. when opened to traffic, but at the end of 18 months the texture depth had decreased to 0.07 in., which was the same as that of the dense graded mix.

Surface Treatments. Surface treatments have long been one of the cheapest and most effective means of restoring a high skid resistance to an existing road. The reduced expense in comparison with the cost of a surface course overlay, and the speed and ease with which a surface treatment can be placed, make it a very acceptable solution. In fact, surface treatments provide a much better skid resistant surface texture than conventional asphalt concrete and, because of the orientation of the chippings, give a better texture than hot rolled asphalt (37). Several difficulties do arise from the use of surface treatments that limit their application to light traffic volume roads. The newly laid chips are vulnerable to damage by heavy traffic, especially at high-stress points such as bends and intersections. Also, loose surplus chips that are scattered over the roadway during the early life of the treatment may result in broken windshields or similar problems.

Blending Aggregate. One method used to increase the skid resistance of existing aggregate is to mix the existing mineral aggregate with a blend of a more favorable aggregate. This process, not yet perfected, has had little practical use and therefore little information has been complied on its success.

Several researchers, such as Mullen et al. (14) and Galloway and

Hargett (38), have compiled data on the possibility of blending the larger aggregate of a wear resistant rock into a mix containing soft, polish prone aggregate. Mullen's work established a direct linear relationship between the coefficient of friction values of a blend resulting from varying percentages of each aggregate in the mix. Figure 5 illustrates Mullen's work with a blend of limestone (polish prone) and crushed silica gravel (polish resistant) aggregates used in an open graded mix.

Galloway and Hargett (38) concentrated on blending artificial lightweight aggregate with natural aggregate. In most instances, artificial lightweight aggregate does not satisfy durability requirements for use as the coarse aggregate in asphalt pavements (39). When it is combined with other, more sound aggregates, a better overall performance is possible. For example, in combination with a high polish prone yet durable rock, lightweight artificial aggregate has economic advantages that would make it more favorable for blending than a natural rock with wear-resistant qualities.

Effects of Traffic

The effect of traffic on the skid resistance of pavement has not been finalized by research because of the many variables that must be evaluated for each different road. Efforts to evaluate pavement conditions and effects of mixed traffic first started in California in 1942 (40). Since that time, most work has focused on fatigue and stress relations, and not on problems of skid resistance.

Field testing has shown that identical surfacing materials compared at different sites have skid friction constants that are inversely related to the volume of traffic (15, 24). The conclusion

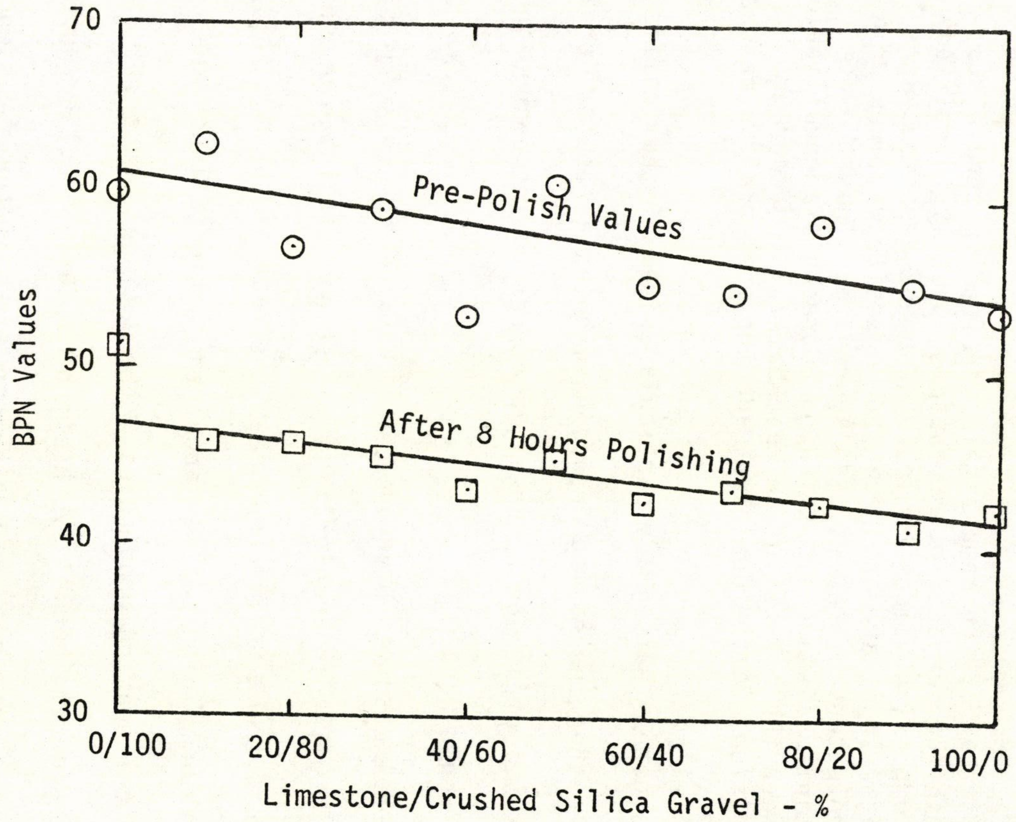


Figure 5. Polishing Properties of Blended Aggregates in an Open-Graded Mix (14)

drawn from these observations is that the effect of traffic on skid resistance is not cumulative from year to year, but rather is relatable to rate of loading (average daily traffic). Therefore, assumptions and concepts used in fatigue studies do not apply to skidding applications.

Salt (24), in an attempt to explain these observations, states that the reason most generally accepted:

> ...is that at the same time as traffic is tending to polish the surface, other factors, usually identified as complex, physico-chemical phenomena described as 'weathering', are acting in the opposite way, restoring microtexture of the exposed aggregate. Thus the resultant resistance to skidding represents an equilibrium between the effects of certain naturally occurring conditions on the one hand, and those of traffic on the other.

Salt also notes that on any particular site, if the traffic flow is changed, either increasing or decreasing (as a result, for instance, of road development in the area), a corresponding change in skidding friction would result.

Several attempts have also been made to estimate the effects of truck traffic in accelerating polishing. It is an uncontested belief that truck traffic enhances the wear of surface friction and texture, but opinions differ about the ratio of truck wear equivalency to car wearing properties. The National Cooperative Highway Research Program (NCHRP) Synthesis of Highway Practice No. 14 (16) reports that one truck is equivalent to about three passenger cars, but equivalencies up to 11 to 1 have been used. In 1976, Dahir and Meyer (15) reported that on the average, a truck was equivalent to about 18 passenger cars in reducing pavement skid resistance.

These differences illustrate the need for more complete and controlled traffic studies in relation to change in pavement skid resistance. Traffic volume is influenced by many factors ranging from cyclic variations such as season of the year, day of the week, and hour of the day to more general and uncontrollable factors such as the national economy and international relations. Also, the type of truck load would be of concern. Is the road carrying average loads, or is the road a route for delivery of steel and masonry products? It is obvious that the most accurate of studies involving truck traffic undoubtedly are localized to the specific conditions acting on a pavement.

Measurement of Skid Resistance

To evaluate the results of improved skid resistant pavements and the older slick pavements, several methods of measuring skid resistance have been developed (16): the braking force method, including the stopping distance method and the method of deceleration; locked-wheel trailers; the side-force method; and the portable and laboratory instruments, including the widely used British Portable Tester.

Other methods for characterizing pavement skid resistance are technically just as valid, but the approaches listed are the basic methods that have been used in the United States and England. Each method gives different numerical results and responds differently to such variables as speed or pavement texture and the tire or rubber elastic and viscoelastic properties. In the measurement of pavement skid resistance, it is best not to speak of "coefficient of friction", because the term implies a fairly simple basic interaction between the two bodies sliding in relation to each other. The term "friction factor" conveys a more general meaning and is used when one does not wish to or cannot define the nature of the fricitonal interaction.

However, it is incorrect to say that a pavement has a certain friction factor (or coefficient of friction) because friction always involves two bodies. By definition, then, skid resistance of a pavement is the force developed when a tire that is prevented from rotating slides along the pavement surface (16).

The Standard ASTM Method E 274 (41) prescribes all variables that influence pavement friction factor. Measurements made in accord with this method are reported as skid numbers (SN), such that:

$$SN = 100f = 100\ F/W$$

where:

SN is subscripted with the speed of the test tire

f = coefficient of friction at that speed

F = tractive force applied to a sliding (locked) standardized tire at a constant speed (usually 40 mph) along an artificially wetted pavement

W = dynamic vertical load on test wheel

Field Tests

One of the more detailed reports on experiments with equipment was given by Dillard and Mahone (42) in 1964. Machines used included towed skid testers, stopping-distance vehicles, and portable testers. The machines were tested on five test surfaces located on a flight strip at Tappahannock, Virginia. Each test surface was constructed to provide a different skid number. The researchers concluded from their study that the spread in skid test data from the various machines was statistically significant and that more work was required to improve the accuracy and precision of the devices.

Salt (24) describes the development of an apparatus to measure skid resistance reliably by the British TRRL. The sideway-force machine has been adopted in Britain as the basic measure of pavement skid resistance. The present sideway-force coefficient routine investigation machine (SCRIM) was turned over to commercial production in 1968 and 15 are now in use throughout the world.

The side force is generated by a test wheel running at an angle of 20 degrees to the direction of travel. The vehicle is mounted on a production commercial vehicle chassis and contains its own water tank holding 713 gallons (2700 liters) to wet the road in front of the test wheel. This amount of water is satisfactory for 31 miles (50 km) of testing, about half of one day's work. The test wheel has a smooth tire and its own deadweight, spring, and shock absorber to give a known static reaction between tire and road. In the hub is an electrical load cell to provide the sideway-force input to the recording system.

The side force output, on magnetic or punched paper tape, consists of a record of sideway-force coefficients (SFC) at intervals of 16 ft. (5 m), 33 ft. (10 m), or 66 ft. (20 m). A speed input is provided from the vehicle transmission system to the record, and data are provided to indicate location of the test site.

<u>Locked-Wheel Trailer</u>. Use of a tire representative of those commonly used on vehicles was thought to provide results of skid resistance tests that could be applied directly to a vehicle's performance in traffic. The difference in performance, however, of the

variety of tires currently available for use on private vehicles makes such a generalization questionable. This difficulty cannot be overcome, but the use of a standard tire on the locked-wheel trailer allows comparison of the skid resistance of different pavements. The American Society for Testing and Materials (ASTM) has defined a standardized, full-size test tire in Standard ASTM Specification E 249 (41). It is a bias-ply 7.50 x 14 tire with five circumferential grooves. Its use eliminates the tire type and design variables in the skid resistance measurement of the pavement.

The most widely accepted method of locked-wheel testing is use of the two-wheel trailer in accordance with Standard ASTM Method E 274 (41). This trailer is towed at a specified speed over the dry pavement by a towing vehicle that carries a water supply, instrumentation, and other associated equipment. At the time of the skid measurement, water is applied in front of the test wheel. The test wheel is then completely locked by a suitable braking method and after sliding has occurred for a predetermined period of time (to permit the temperature in the contact patch to stabilize), the frictional force between the pavement and the test wheel is measured by transducers. The electronic signal is relayed to a recorder for tabulation and visual observation. The frictional coefficient measured by this test is reported as the skid number, at the speed of the towing vehicle.

The Federal Highway Administration (FHWA), Department of Transportation, has established Field Test and Evaluation Centers (FTC) in key locations around the United States. The objective of these centers is to reduce interstate variations in locked-wheel skid measurements of pavement surfaces. To accomplish this objective, the FTC provides technical advisory services, static and dynamic calibrations, and standardizstion services for skid resistance measurement systems.

The AHD skid trailer No. 1 was calibrated at the Texas Transportation Institute FTC, Texas A&M University, in April 1973. The report (43) summarizes the calibration, correlations, and evaluations (as arrived condition and after calibration completed) by the FTC on the Arkansas No. 1 skid measurement system. On the basis of the results of this calibration work, the skid test measurements were related to a standard which at that time was the Central Interim Reference System.

British Portable Tester. The British Portable Skid Resistance Tester, better known as the British Portable Tester (BPT), was first developed by Sigler of the U.S. National Bureau of Standards (4). Later, it was perfected by the Road Research Laboratory of England as a routine method of checking the resistance of wet road surfaces to skidding. The apparatus measures the frictional resistance between a rubber slider, which is mounted on the end of a pendulum arm, and the road surface. The BPT is designed for laboratory use as well as field testing in accordance with Standard ASTM Method E 303-69 (41). It has become one of the most widely used methods of measuring skid resistance in the laboratory and in the field. The BPT was designed to simulate sliding between a vehicle tire moving at 30 mph and the road surface. Special attention also was given to overcoming all types of weather, rough and uneven road surfaces, and any considerable gradients that might be encountered in testing. The British Pendulum Number (BPN) units of skid resistance measured with the portable tester represent approximately 100 times the coefficient of friction of the particular surface being tested.

Giles et al. (18) pointed out that several factors can affect the test results of the BPT. The main factors usually fit into two groups, those connected with the design of the machine and its method of use and those connected with the performance of the road surfaces under test. These factors include operator variation, physical properties of and the effects of water on the rubber slider, variations in the road surface texture, and seasonal changes.

Giles et al. (18) showed that the difference between operators is not significant, and that the choice of the standard procedure for testing a length of road gives a value representative of the performance of the surface that is independent of any influence of the operator using the machine. Likewise, Underwood et al. (13) indicated that the greatest BPN range, with different operators, was 2; however, there was also a range of 2 for one operator (with repeat tests) where the operator was required to set the instrument up between tests. Most of the variance, it is believed, can be attributed to adjusting the slider length.

To obtain the desired standard of consistency with the tester, it is essential to have a close control on the physical properties of the rubber used as the slider. The most important property is the resilience of the rubber (or its hysteresis losses). Even with the strict limits set by Standard ASTM Method E 303-69 (41), the effect of temperature on rubber resilience gives rise to changes in "skid resistance" results. As the rubber temperature increases, the rubber becomes more resilient (the hysteresis losses become smaller) and the "skid resistance" values tend to decrease. As a result of several tests on road surfaces, an average temperature correction has been derived to allow for the changes in the resilience of the rubber slider (18). However, Giles et al. (18) also pointed out that the effect of temperature is important only for tests made at temperatures below 10 C when the correction becomes of the same order of magnitude as the variations recorded along a typical surface.

Another physical property of the slider rubber that influences the friction is its hardness; "skid resistance" increases with increasing hardness of the slider rubber. The hardness effect is not as great as the resilience effect, however, and the hardness has no appreciable temperature variation. Also, though the friction between a slider and the surface is substantially reduced by the presence of even a trace of water, the lowest results are obtained by wetting the surfaces thoroughly, using a water layer of at least 0.01 in. (18).

Rounding of the slider edge is another factor that affects the BPT results. A change in the slider contact length of approximately 1/20 to 3/8 in. causes about an 8 percent increase in skid resistance; therefore a rubber slider is good for about 500 strikes of the BPT (18).

The BPT can carry out a wide variety of measurements. The calibration of the machine is based on the effective weight of the pendulum arm, the distance over which the slider is in contact with the surface being tested, and the nominal load on the slider. Giles et al. (18) tested about 150 machines in use and showed that, with this kind of absolute calibration, all the machines agreed within an accuracy of $\pm$ 3 percent.

There are some discrepancies among researchers in the terminology used in reporting test values obtained with the BPT. For field tests, the British unit of measure is termed British Pendulum Number (BPN).

The BPN value is obtained with a standard stike path of 5 in. and a 3-in. wide by 1-in. long rubber slider. For British laboratory tests, an auxiliary scale is attached to the regular BPT scale and the test values are reported as "polished stone values" (PSV). PSV are numerically equal to BPNs because of the auxiliary scale; however, the strike path is reduced to 3-in. long and a narrower rubber slider (1½ in. wide by 1-in. long) is used.

Several agencies in the United States use the BPT for laboratory testing and report their test values as "polish values" (PV). In this case the auxiliary scale has not been used, and the values reported were obtained from reading the regular BPT scale. When the sample tested is of the same configuration as the British, then the PV reported is equal to 3/5 PSV. For example, a PV reading of 35 is equal to a PSV value of 58 (44). The smaller test area of the laboratory sample (3 sq. in.) undoubtedly will give a different value than the larger area (15 sq. in.) used in field tests. Therefore, the relationship between BPN and PSV of PV is not exact.

Many attempts have been made to correlate the BPT test results with skid numbers from the locked-wheel devices. Giles et al. (18) reported the results of correlation studies between the BPT, the Virginia Test Car, and the British SFC. Dillard and Mahone (42) presented extensive test data on skid numbers from various devices and the BPN values for five different test surfaces. Not all researchers agree on a specific relationship between the BPT and locked-wheel skid numbers. Sabey, in 1964, discussed the report by Dillard and Mahone (42) and presented a summary of test data correlations between the BPT results and all locked-wheel skid results

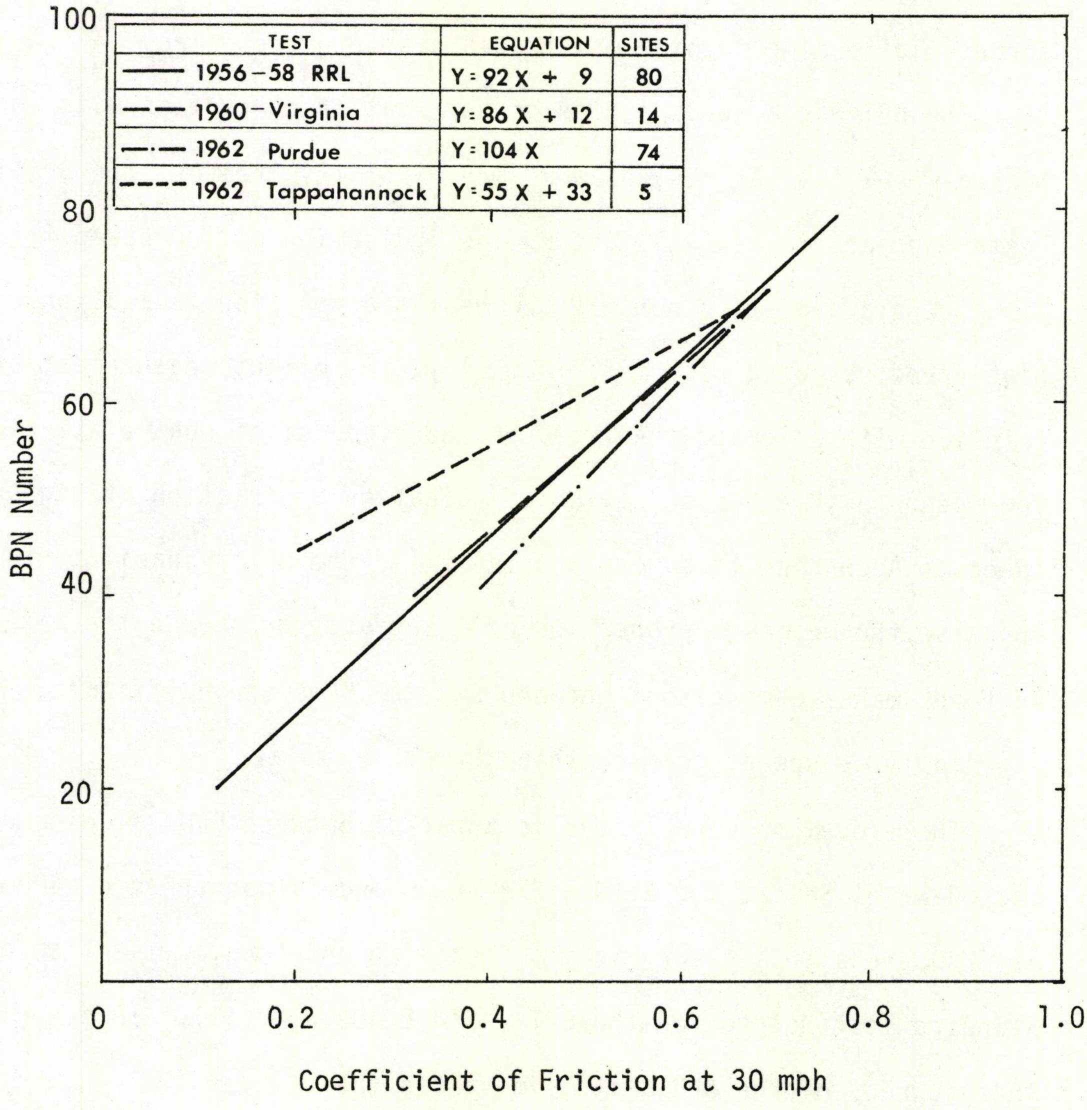


Figure 6. Relationship Between BPN Numbers and Coefficient of Friction from Locked-Wheel Devices (42).

available at that time. The results of these correlations are shown in Figure 6. Test results for a vehicle speed of 30 mph were used in these correlations. All test data correlations are from "patterned tires" except the plot for the Road Research Laboratory's sideway-force skid-testing machine which uses a smooth test tire.

The BPT has a good regression correlation comparison with the locked-wheel trailer tested at 30 mph. For this reason, the British Tester appears to be satisfactory for indication of low-speed frictional characteristics but may be unsatisfactory for indication of high-speed skidding risk (45). Equally, the direct measurement of friction with a locked-wheel tester, carried out at one relatively low speed only, gives no reliable indication of friction at higher speeds. According to Kummer and Meyer (4) the BPT's inability to appraise the drainage properties of a surface, and hence the macro-texture, makes comparisons between SNs and BPNs very difficult, especially at speeds greater than 35 mph.

Therefore, no correlation is apparent between BPN values and locked-wheel SNs at the ASTM E 274 recommended test speed of 40 mph. ASTM supports this theory in its Precision and Accuracy section of the Standard Test Method for the BPT (ASTM E 303-69) (41). In describing the accuracy of the BPN units, it states that:

> 7.1) ...As there is no marked correlation between standard deviation and arithmetic mean of sets of test values, it appears that standard deviations are pertinent to this test regardless of the average skid resistance being tested.
>
> 7.2) The relationship, if any exists, of observed BPN units to some "true" value of skid resistance has not and probably cannot be studied. As a result, precision and accuracy of this test in relation to a true skid resistance measure cannot be evaluated, and only repeatability is given for the method.

Minimum Skid Values. The minimum skid value permissible in pavement design is difficult to predict because of the many possible combinations of speed, weather, tires, drivers, and roadway geometry. The AASHTO Blue Book (46) for geometric design indicates that the coefficients of friction used for design criteria should represent not only wet pavements in good condition, but also surfaces throughout their useful life. It suggests a minimum coefficient of friction of 0.33 for providing minimum stopping sight distance at 40 mph.

Smith (47) discusses the skid values recommended by several state highway departments, and research agencies. Louisiana specifies minimum skid values based on SN_{40} values, whereas a Texas Transportation Institute study suggests a minimum skid value based on "polished stone value". These Texas polished stone values are numerically equal to 3/5 PSV obtained by the British. Recommended minimum skid or polish numbers are shown in Table IV.

Giles et al. (18) suggest that minimum BPN values for critical sections should be kept above 65, but categorize their values to type of site rather than traffic load. They describe the most "difficult" sites as: 1) roundabouts, 2) bends with radius less than 500 feet on unrestricted roads, 3) gradients, 1 in 20 or steeper, of length greater than 100 yd., and 4) approach to traffic lights on unrestricted roads. For all sites they recommend a minimum BPN value above 45.

Salt (24) reports the TRRL recommendations for minimum skid resistance in terms of side-force friction coefficient, polished stone value, and commercial vehicles per lane per day. These recommended values of skid resistance are shown in Table V. The British use the PSV term to indicate values obtained in laboratory tests; BPN

Table IV

Recommended Minimum Skid or Polish Values

A. Louisiana Department of Highways (47)

ADT per Lane	Minimum SN_{40}
Less than 200	40
200 - 999	43
1000 - 5000	45
More than 5000	47

B. Texas Transportation Institute (47)

ADT per Lane	Median PSV*	Range
100 - 400	28	26.5 - 29.5
400 - 1600	32	30.5 - 33.5
1600 - 6800	27	35 - 39
6800 - 27,000	42	40 - 44
More than 27,000	47	47 - 49+

*PSV = Polished Stone Value

C. NCHRP Report 37 (4)

Mean Traffic Speed, MPH	SN_{40} (Minimum)	BPN (Minimum)
30	31	50
40	33	55
50	37	60
60	41	65
70	46	-
80	51	-

D. Texas Highway Department (44)

Present ADT Grouping	Minimum Polish Value
0 - 749	NONE
750 - 1999	30
2000 - 4999	33
5000 - over	35
Interstate Highways	35

Table V

British (TRRL) Criteria for Aggregate PSV Related to Sideway-Force Coefficient and Commercial Traffic

Required mean summer SFC at 50 km/h	PSV of aggregate necessary — Traffic (in commercial vehicle per lane per day)					
	250 or under	1000	1750	2500	3250	4000
0.30	30	35	40	45	50	55
0.35	35	40	45	50	55	60
0.40	40	45	50	55	60	65
0.45	45	50	55	60	65	70
0.50	50	55	60	65	70	75
0.55	55	60	65	70	75	
0.60	60	65	70	75		
0.65	65	70	75			
0.70	70	75				
0.75	75					
AAV — Chipped surfacings	not greater than 14	not greater than 12		not greater than 10		
AAV — Macadams	not greater than 16		not greater than 14		not greater than 12	

numbers are reserved for values obtained in field pavement tests.

Laboratory Tests

Several methods have been used for pre-evaluating skid resistance of aggregates and asphalt mixtures prior to their use in a roadway surface. The laboratory investigation of these pavement surface materials permits the materials engineer to estimate how a pavement will wear and which materials will provide satisfactory service, and thus allows savings in accident costs, maintenance and reconstruction.

According to Goodwin (48), several agencies have made significant contributions to laboratory evaluation of the skidding characteristics of pavement materials: Purdue University, Kentucky Department of Highways, the National Crushed Stone Association (NCSA), the Portland Cement Association (PCA), Pennsylvania State University, and the Tennessee Highway Research Program. In addition, the Texas Highway Department (13), the Oklahoma Department of Highways (49), the Highway Research Program at North Carolina State University (7), and the British TRRL (24) have reported pertinent laboratory test data of skid resistance studies.

The known methods for pre-evaluating skid resistance of materials can be grouped on the basis of relative sample size. For example, the Purdue and Kentucky apparatus use samples between 4 and 6 in. in diameter, whereas the PCA and Tennessee apparatus use samples of sufficient size for testing with a standard size automobile tire. The NCSA, Purdue, and North Carolina utilize a horizontal circular track; Texas, Oklahoma, Pennsylvania, and the TRRL use a vertical circular track such as the British Wheel.

In general, all test methods require a conditioning of the specimen surface, either before or during testing. This conditioning is usually referred to as polishing wear, and is brought about by a rubber annulus such as a tire.

Two procedures are mainly used to accelerate surface wear from the tire beyond the rate obtained by simply rolling the tire over the test surface. One is to fasten the specimen and the tire into place and rotate the tire so that it skids over the surface of the sample as it turns. The other method is to fasten the specimen into place and rotate the specimen in relation to the tire so that each moves individually over the other. A fine abrasive sand spread between the tire and the sample will enhance wear; also, "toeing" either the specimen or the tire slightly away from the normal direction of rotation causes a scrubbing action over the specimen surface which accelerates the polishing effect.

<u>Horizontal Circular Track</u>. An example of this type of accelerated polishing device is the NCSA test track. The track is 14 feet in diameter, 1.5 feet wide, and can accomodate as many as 20 test specimens at a time. Severe traffic action is simulated by the continuous passage of a bus wheel loaded to 2,000 lbs. on a single pneumatic tire. The mixtures being tested are first subjected to wear to remove surface asphalt and expose the aggregate particles. Next the track is thoroughly cleaned and the initial slipperiness test is run. The polishing action of the tire is then begun. Slipperiness is usually measured after 2.5, 5, 10, 15, 25, 40, and 50 thousand wheel passes with a BPT in accordance with Standard ASTM Method E 303 (11).

The NCSA researchers have also measured the slipperiness with a bicycle wheel. This wheel, placed directly on the pavement section to be tested, is ground off to expose the tire fabric over half of its circumference; the other half of the tire retains its full thickness of tread. Counter-balance weights are attached to the rim of the uppermost portion, and the height of the wheel is adjusted so that the wheel turns freely except when the thick portion of the tire is down on the surface. The wheel is released, allowing the weights to rotate it so that the thick portion of the tire strikes the pavement surface, thereby raising the wheel slightly in the slotted supports which hold the axle. The wheel is then supported only by the surface, and it continues to turn until brought to rest by the friction between the tire and the road. The more slippery the pavement, the greater the angel of turn required to bring the wheel to rest. An average of eight readings is taken to indicate the pavement slipperiness (48).

The Purdue University method of laboratory skid testing was developed by their Joint Highway Research Project (50). The apparatus consists of a vertically mounted mandrel with a chuck to carry a pavement sample about 6 in. in diameter, and a power unit to rotate the mandrel at a speed of about 2,500 rpm. A rubber test shoe is mounted on a ballbearing shaft that is in line with the specimen held by the mandrel. The shaft is restrained from turning by a cantilever bar on which are located SR-4 strain gages. The restraint to rotation is recorded as the frictional resistance. In the test, as the specimen is rotated the rubber test shoe is forced against the specimen surface through a mechanical arrangement for applying a constant normal force of 28 psi. The test shoe is held against the spinning specimen for 2 seconds and then removed. After a pause of approximately 2 seconds, the test is repeated and the resistance is reported as a relative value. Water is supplied to the specimen surface during testing.

The Purdue University researchers attempted to correlate the laboratory results with field test results obtained from a stopping-distance automobile. They concluded that surfaces of medium texture had good correlation between laboratory and field tests. For open surfaces, the laboratory method indicated poorer anti-skid characteristics than did the stopping-distance method. For an extremely dense surface, the laboratory method showed higher values than those obtained by the stopping-distance measurements (48).

The North Carolina State University Wear and Polishing Machine development was reported by Mullen and Dahir (51). This small diameter circular track machine is capable of accelerated wearing and polishing of 6-in. diameter pavement specimens. The machine consists of four wheels that travel over a segmented circular track, a 36-in. diameter track which contains 12 specimens, and an electric motor that drives the central shaft. Opposite tires are toed in and the other two tires are toed out about 1.6 degrees to produce a scrubbing action in addition to the rolling action. The tires used on this machine are 4.10 X 3.50 X 5.00 Go-Cart Slicks having a 5½ in. tread width. The tire inflation pressure was maintained at 20 psi which gives a tire-pavement contact pressure of 13 psi. Three replicate test specimens were prepared and tested on the machine to represent each test sample. The specimens were tested for initial skid resistance with a BPT, and then subjected to polish on the machines, with the wheels rotating around the track at the rate of 30 rpm. The standard length polishing action to achieve maximum polish was taken as 16 hours, and the BPN values were taken at 0, 1, 2, 4, 6, 8, 10, 12, 14, and 16 hours.

Test results obtained with this device by researchers at North Carolina State University (7, 14, 51) are shown in Figures 2 and 5. In addition, Mullen (7) reported the results of comparison tests on three differently graded laboratory mixtures molded with a stock aggregate used for control specimens. The results of these comparison tests are shown in Figure 7. The gradation for the mixtures of Figure 7 (open-graded, a No. 4 mix, and the dense-graded I-2) is shown in Table VI. The gradation of the North Carolina University dense-graded I-2 mix is shown in Table III for comparison.

The curves of Figure 7 indicate that a dense-graded I-2 mixture has a higher BPN value than either the open-graded or No. 4 mixture. The No. 4 mixture is a larger aggregate dense-graded mixture than the I-2 mixture. The stock aggregate selected for control was a quartz-disc muscovite gneiss (51).

Mullen (7) also correlated field BPN values with the North Carolina State Highway Commission skid trailer SN values. The results of this correlation work for an I-2 mixture and SN_{40} test data are shown in Figure 8. By regression analysis the best fitted line for this correlation was:

$$SN_{40} = -19.465 + 1.118 \text{ BPN}$$

with a correlation coefficient of 0.909. The open-graded mixture was likewise evaluated and the best fitted line was:

$$SN_{40} = 28.690 + 0.451 \text{ BPN}$$

with a correlation coefficient of 0.774.

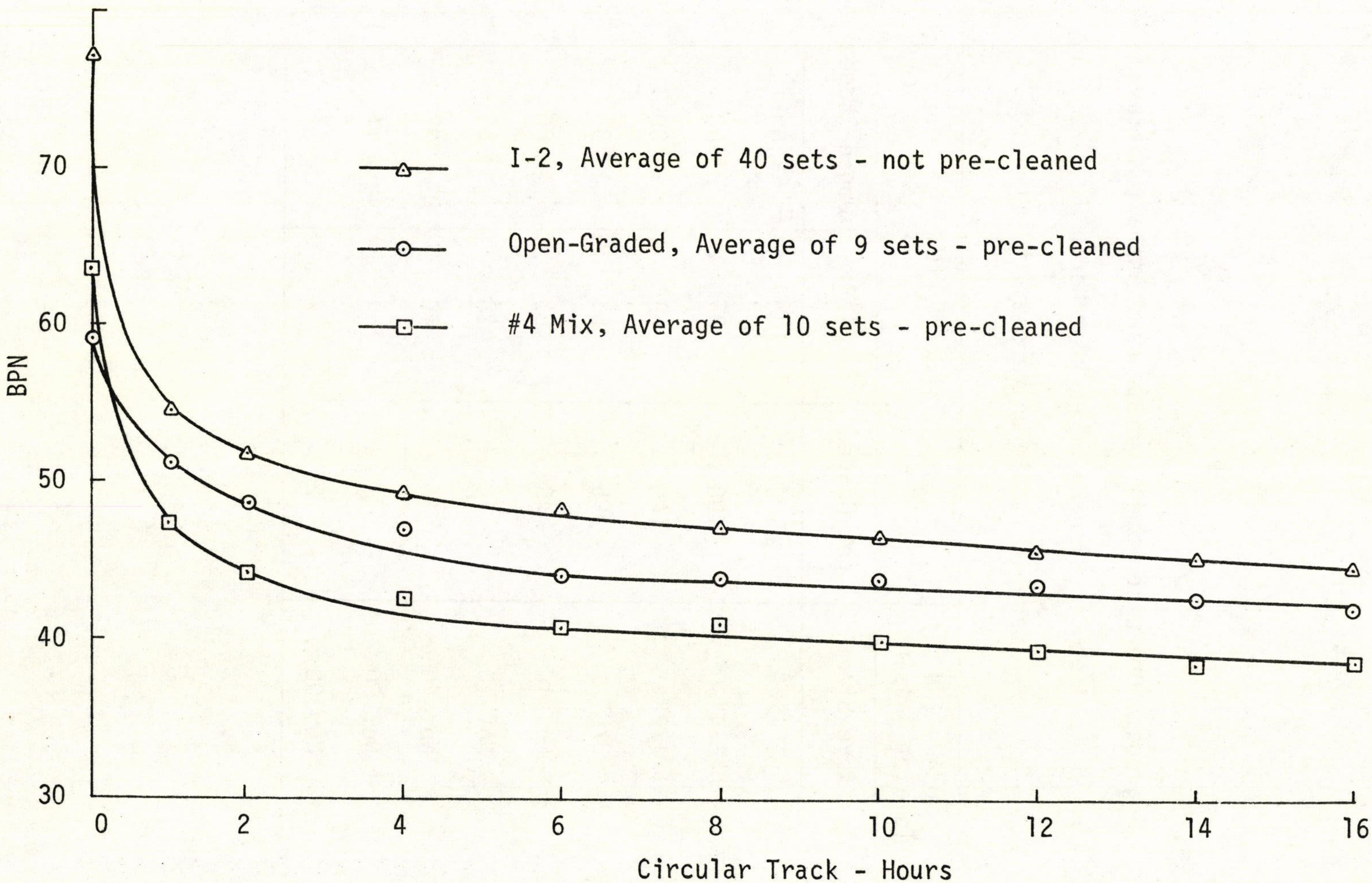


Figure 7. Relationship Between BPN Values and Circular Track-Hours for the Standard Aggregate (7).

Table VI

Aggregate Gradation used for Standard Asphalt Mixtures on Circular Wear Track at North Carolina State University (7)

GRADATION Total % Passing	TYPE OF MIX		
Sieve Size	Open-Graded	No. 4 Mix	Dense-Graded I-2
1"	100	100	100
3/4"	100	93	100
1/2"	100	68	100
3/8"	95	53	95
No. 4	43	33	80
No. 10	10	28	55
No. 40	4	15	27
No. 80	3	4	14
No.200	2	2	4

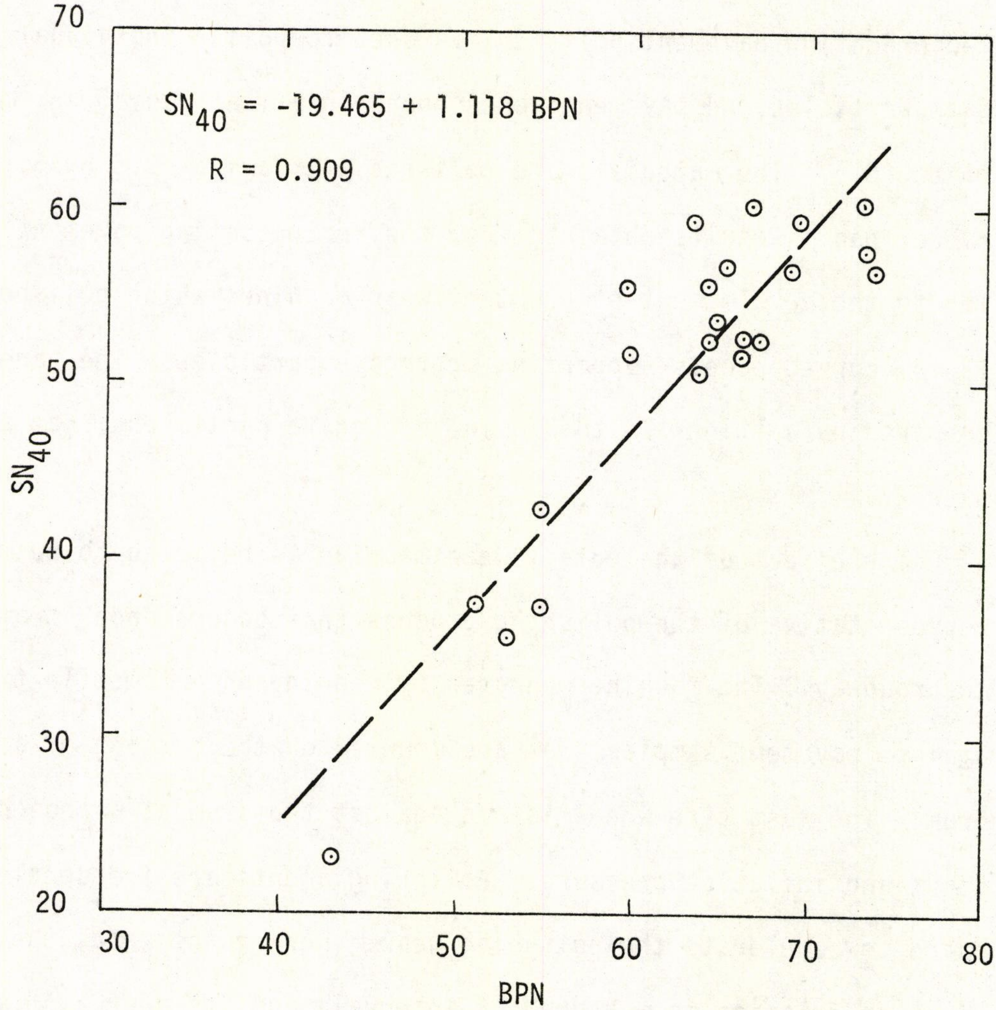


Figure 8. Skid Number at 40 mph vs. BPN,
North Carolina I-2 Mix (7)

Vertical Circular Track. The Pennsylvania State University has three polishing devices: a circular track, a reciprocating pavement polisher, and a rotary drum polishing machine (48). Their modified reciprocating pavement polisher was used to polish individual aggregate particles and pavement cores bound on a flat 3 by 5 in. metal plate (52). The materials are polished with a flat 3.5 by 5.5 in. rubber pad. Results obtained from the reciprocating pavement polisher led to the development of a rotary wear machine, which polishes 4 by 12 in. curved panels supporting aggregate particles. The panels are prepared by setting 1/4 to 3/8 in. aggregate particles in an epoxy matrix.

The effect of the rotary wear machine is believed to be more representative of the polishing process that occurs under traffic on the roadway. The machine operates by running an automobile tire against pavement samples that are mounted on the outside of a rotating drum. The test tire wheel is run against the drum at a chosen speed, load, and inflation pressure. Polishing agents are frequently introduced to accelerate the polishing wear. During the test, the coefficient of friction is measured at intervals and its decrease is used as a measure of the progress of the test. When the coefficient approaches a constant value, the polishing process is complete.

A "drag tester" with a rubber shoe is mounted in such a way as to permit measurement of the panel frictional resistance as the drum containing the test sample rotates. The drag tester has a BPN-type test shoe. The BPT is also used to measure the BPN value of the specimens in accordance with Standard ASTM Method E 303-69.

Dahir et al. (52) reported the results of a Pennsylvania investigation which included 52 aggregate samples and 223 pavement cores representing field test sections. The materials were tested in the laboratory by various polishing methods and friction measurement techniques. In regard to laboratory and field data correlation, Dahir et al. concluded:

> Laboratory-field data correlations indicated that the general level of skid resistance characteristics of surface aggregates may be determined in the laboratory and the aggregates may be ranked similarly by both approaches. However, the correlations failed to produce regression equations that could, with confidence, define specific mathematical relationships for predicting specific field skid numbers.

Dahir also correlated some of the results of the Pennsylvania investigation with the North Carolina results reported by Mullen (53). The correlations between SN_{40} and BPN for the Pennsylvania ID-2A dense-graded mixture resembled but differed from Mullen's results for the North Carolina I-2 dense mixtures.

Of particular interest is the relationship between minimum SN_{40} values obtained in the field and BPN values from the laboratory-polished pavement cores reported by Dahir et al. (52). Figure 9 shows the relationship between 3-year minimum average SN_{40} values and BPN values for laboratory-polished cores. The correlation between field data (SN_{40}) and laboratory polishing data on aggregate panels resulted in a similar trend but the correlations were less satisfactory, having coefficients in the range of 0.55 to 0.65.

A report from the Pennsylvania Department of Highways (Penn DOT) by Furbush and Styers (54) gives the correlation of SN_{40} values and field core BPNs. The pavement cores were taken from all sections of 11 test strips that were constructed for skid resistance studies. The cores then were tested in the laboratory with the BPT and the results

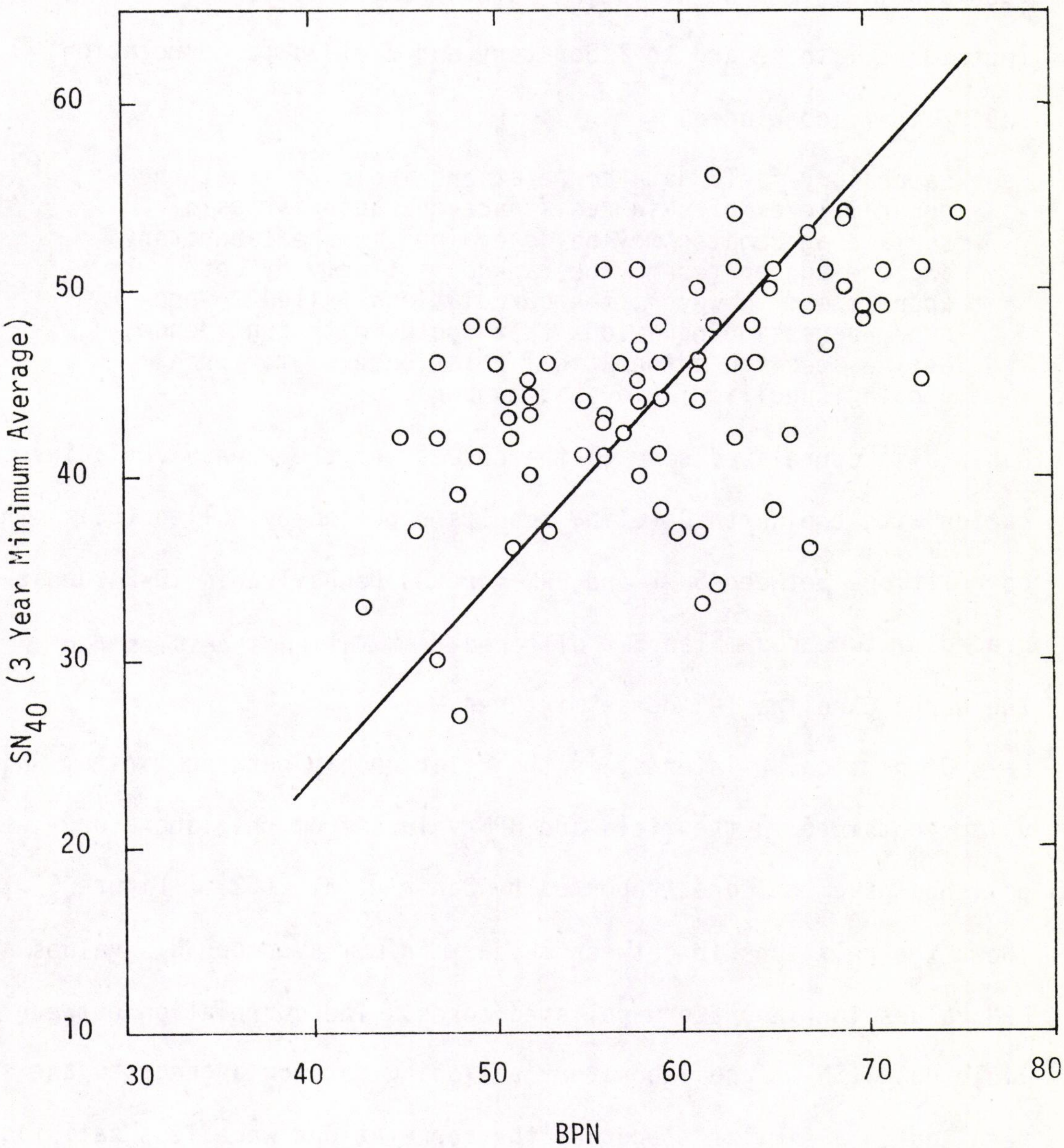


Figure 9. Minimum Average (3 year) SN_{40} v. Polished Core BPN, Pennsylvania State University (52).

(BPN values) were compared with the results of skid-trailer tests that were closest to the date of coring. The best fitted line relating the SN_{40} and a core BPN value was:

$$SN_{40} = 0.0347 + 0.90 \text{ BPN}$$

with a coefficient of correlation (R) of 0.64. One of the conclusions stated in this report was, "At a testing speed of 40 mph the fine aggregate in the mixes has considerable influence on skid resistance."

The Oklahoma Department of Highways (49) and the Texas Highway Department (13) use the British Wheel and follow a modified British Standard 812 Method of Test in evaluation of aggregates for surface mixtures. The British Standard 812 Test for coarse aggregate involves the measurement of the coefficient of friction of aggregate specimens with the BPT after subjecting the specimens to wearing and polishing on an accelerated polish machine (which is termed the British Wheel). The major components of the British Wheel are: 1) road wheel - a steel wheel 16 in. in diameter, 1 and 3/4 in. wide, around which 14 aggregate specimens are clamped; 2) rubber tired wheel - a smooth rubber tired wheel (8 by 2 in.) which rests on the road wheel test specimens with a force of 88 ($\pm$ 1) pounds; 3) aggregate specimen - the aggregate(s) to be tested are molded in a polyester binder with flat faces outward; and 4) grit feed system - accelerated wearing and polishing are induced by the continuous, uniform application of abrasives between the rubber tire and the road tire.

The aggregate specimen size is 3.5 by 1.75 in., having a thickness of a single layer of aggregate; the specimens are molded to the same radius of curvature as the road wheel, around which they are attached.

The aggregate particles to be tested are graded to pass the one-half inch sieve and are retained on the No. 4 sieve. The grit is vibrated from a small hopper to a chute at a specified rate. Water is supplied (also at a specified rate) and washes the grit onto the road wheel near the point of contact of the road wheel and the rubber tire. The grit used is a No. 150 (100 microns) silicone carbide grit. This size grit was selected after research had proved that various sizes of abrasives form stair-step friction values, the finest grit forming the lowest friction value (13).

The Texas Highway Department has standardized their British Wheel test procedure as Test Method Tex-438-A. Briefly, this test is performed on a total of 14 specimens at one time, with a polishing time of 9 hours of wheel operation. A minimum of seven specimens of each aggregate sample are polished. The polishing value of each specimen is determined before and after polish with the BPT in accordance with Standard ASTM Method E 303.

On the basis of the test results from the British Wheel, minimum polish values for coarse aggregate to be used in pavement surfaces have been determined (13). The most recently recommended polish values for the Texas Highway Department are reported by Hankins (44) and are shown in Table IV.

Durability

The durability of asphalt pavement is one of the major requirements for a satisfactory asphalt mixture. The other primary requirements are to provide high stability, adequate skid resistance, an impermeable waterproof layer, and economy.

The factors thought to influence overall pavement performance include structural design of the roadway, asphalt mix design, mineral aggregate properties, asphalt material properties, construction techniques, amount and character of traffic, environment of the road, and maintenance. Distress of the pavement surface leads to reduced performance. All failures, whether caused by base or subgrade material, are reflected in the pavement surface. The causes of pavement failure include poor mix design; poor construction technique; insufficient compaction of pavement, base, and subgrade; differential settlement of the subgrade; and water entering (and reducing the strength of) the subgrade, base, or pavement.

Hveem (55) presents a classification of bituminous road surface failures which describes the types of distress in the surface layer composition and their causes. The three common types of distress and their cause are: 1) disintegration, caused by a lack of asphalt or hardening of the asphalt or by water action; 2) cracking, caused by hardening of the asphalt or low temperatures or lack of asphalt; and 3) instability (plastic deformation), caused by an excess of asphalt, an excess of water, or smooth aggregate particles.

Distress of pavement surfaces is noted as: cracking, stripping, raveling, instability, rutting, distortion, or disintegration. Because durability is a function of these variables, one of the major problems of the highway engineer is to measure pavement durability and associate this measurement with the various types of distress. Evaluation of the foregoing causes of pavement failure reveals that asphalt and water are two of the major factors involved in asphalt pavement durability.

Effectively, then, the durability of asphalt pavement can be investigated by considering the effect of water on the asphalt mixture. The phenomenon of stripping is the result of the effects of water on an asphalt mixture that cause raveling and in extreme cases disintegration of the entire pavement mass. Stripping occurs where there is a loss of adhesion between the aggregate and asphalt cement due primarily to the action of water.

Factors which affect stripping include aggregate properties, such as absorption, surface texture, porosity, and mineral composition; and asphalt cement characteristics, such as surface tension, chemical composition, and viscosity. Thus, adhesivity of asphalt to aggregate is related to the physical and chemical properties of both the asphalt and the aggregate. This adherence is detrimentally affected by the presence of water, i.e., the bond at the solid-liquid interface is or can be disturbed and deteriorated by water action.

In general, siliceous aggregates have been classified as hydrophilic ("water lovers") and tend to strip more readily than limestone aggregates which have been classified as hydrophobic ("water haters"). Hubbard (56) applied the term hydrophobic to an aggregate which will persistently retain an asphalt film in the presence of water. Practical experience reported by Mathews (57) indicates that relatively few aggregates are known which are completely resistant to the action of water under all conditions of practical use. Further, Mathews states:

> For this reason it is not possible to generalize about the behavior of classes of aggregates. In particular, the well-known and often quoted rule that 'acidic' rocks (high silica content) are vulnerable to stripping whereas "basic" rocks (low silica content) are not, is quite wrong.

In many areas the use of siliceous aggregates in paving mixtures has been curtailed because of bad pavement performance experiences. Frequently, the surface of asphalt concrete pavement is made with limestone aggregate which tends to polish under traffic, thus causing a reduction in skid resistance of the pavement surface. The need for improving or retaining skid resistance in surface mixtures has created interest in incorporating the harder, less easily polished aggregates having a high siliceous content into the paving mixtures.

Before constructing pavements with materials to provide adequate skid resistance, the materials engineer must evaluate the durability or stripping tendencies of the skid resistant materials. Investigators have attempted to determine the effects of water on the adhesion between the aggregate and the asphalt cement. Since 1932, tests on compacted bituminous mixtures have included: immersion compression, cold water abrasion, sonic vibration, water susceptibility, and laboratory test tracks. Tests proposed on asphalt cement coated aggregate include: static immersion, dynamic immersion, chemical immersion, and stripping coefficient. Methods investigated to measure in quantitative terms the amount of stripping of asphalt cement coated aggregate include: radio isotope trace, light reflection, mechanical integration, tracer salt, and dye adsorption. Despite the large number of tests devised to study the effect of water on adhesion, an examination of the technical literature reveals a continuing stream of research studies being conducted to evaluate stripping (58).

Much of this research has been directed toward determining the cause or causes of stripping with emphasis on relating laboratory test results to observed pavement performance. Other research emphasis has been placed on improving the stripping resistance of a particular mixture by use of additives and evaluating minimum quantities of additives required to achieve a satisfactory pavement mixture.

Hubbard (56) discusses the important factors that must be considered to ensure pavements of maximum durability against water action. Briefly, these include: thorough wetting of the aggregate with asphalt, which is essential to good adhesion; use of an asphalt having the highest practical viscosity for the specific application, as this is an important deterrent to stripping; time, which is often an important element in developing the maximum adhesion; and avoiding the use of highly hydrophilic aggregates, as they may lose their adhesion to asphalt films in the presence of water.

For stripping to occur, there must be a loss of adhesion between the asphalt binder and the aggregate particles used in a paving mixture. Thus, much research effort has been devoted to the study of the adhesion characteristics between these two materials. Several theories on adhesion and mechanisms of stripping have been developed by various investigators.

Theories of Adhesion

Adhesion is defined as that physical property or molecular force by which one body sticks to another of a different nature (59). Factors which affect the adhesion of asphalt to aggregate include: surface tensions of asphalt and aggregate, chemical composition of these material, viscosity of the asphalt, surface texture of the aggregate, porosity of the aggregate, cleanliness of the aggregate, and dryness and temperature of the aggregate when mixed with asphalt (60). Four major theories of the cause of adhesion are: chemical reaction, mechanical adhesion, surface energy, and molecular orientation (58).

Chemical Reaction. This theory is credited to Riedel (61) and states that the acidic components of the bituminous material react with basic minerals of the aggregate to form water-insoluble compounds at the interface. Though this theory may have some basis in fact, it does not hold true in all cases because good adhesion has been reported between siliceous aggregate and some asphalts by Winterkorn et al. (62). It is generally believed that siliceous aggregates, which are acidic, tend to strip more readily than basic aggregates, such as limestones (4, 63).

Mechanical Adhesion. The aggregate properties considered to affect mechanical adhesion are: surface texture, absorption and porosity, surface coatings and area, and particle size. Lee (64) observed that rough, irregular surfaced aggregate had better retention of asphalt than smooth glassy-surfaced aggregate. Once wetted, a rough-textured surface will produce greater adhesion under service conditions than a smooth surface.

Different aggregates have been shown to absorb asphalt to different degrees (57). Some components of asphalt, primarily the oily constituents, enter the pores or capillaries of an aggregate particle where they are preferentially adsorbed. This action causes the asphalt coating on the particle to become harder. The interlock of the asphalt coating with these pores should make the asphalt adhere more strongly and be less readily stripped by water (64). However, Tyler (65) found no correlation between adsorptive capabity of an aggregate and the amount of stripping resistance.

Nicholson (59) presents test results showing that colloid coatings on sand grains incorporated in a bituminous mixture tend to reduce the mixture's resistance to stripping. Thelen (60) observed that dust on aggregate surfaces had a tendency to trap air when treated with road oils or cutback asphalts and weakened the bond by preventing intimate contact between the aggregate and the asphalt. Thus, stripping is promoted by providing channels at the interface where water can penetrate.

Stross and Anderson (66) report the importance of the particle size to adhesion in bituminous mixtures, particularly material sizes below the No. 200 sieve. Their test results show that aggregate containing appreciable amounts of clay requires much more asphalt for complete coating than is compatible with mechanical stability. Thus, a stable mixture, with excess fines, would tend to strip because of the presence of partially uncoated aggregate particles.

Surface Energy. The ability of asphalt to make intimate contact with the surface of the aggregate is known as its wetting power. The wettability of the solid surface of the aggregate is its ability to be covered by the asphalt (56). The wetting power of an asphalt is largely controlled by its viscosity. Viscosity is that property of a liquid that represents a resistance to flow caused by molecular friction. Viscosity is also related to surface tension, which is the force tending to hold a drop of it in spherical form. Water has a higher surface tension than most liquid asphaltic materials but its lower viscosity makes it a better wetting agent.

When asphalt spreads over and wets the aggregate surface, a change in energy takes place which is called adhesion tension (67).

This adhesion tension is a surface phenomenon and depends on closeness of contact, mutual affinity of the two materials, and time of contact. Numerically, adhesion tension is equal to the sum of the surface tensions of the asphalt and aggregate less the interfacial tension between them.

Results of measurement of the surface tension, interfacial tension, and adhesion tension of the ingredients of some bituminous mixtures are presented by Rice (67), and are shown in Table VII. McLeod (68) notes that an aggregate tends to become coated by the liquid present for which it has the greatest adhesion tension. Results of tests performed by Mack (69) on energy relations at the interface between asphalt and mineral aggregate indicate that the interfacial tensions vary not only with the type of aggregate but also with the type of asphalt. On the basis of the data presented in Table VII, Rice observed that in all cases, except for the quartz-asphalt interface, the adhesion tension was higher for water to aggregate than for asphalt to aggregate. Thus, according to the surface energy theory, water will tend to displace asphalt at an interface where contact is made between water-asphalt-aggregate.

Molecular Orientation. According to Hubbard (56), when molecules of asphalt come in contact with the aggregate surface they orient themselves so as to satisfy all energy demends of the aggregate to the best of their ability. This orientation in viscous liquids and semisolids proceeds rather slowly, and considerable time may elapse before the maximum adhesion between an asphalt film and an aggregate surface is developed. Water molecules are all dipoles, whereas asphalt molecules predominantly appear to have nonpolar characteristics; thus water

Table VII

VALUES FOR SURFACE, INTERFACIAL AND ADHESION TENSION (67)

Property	Value ($ergs/cm^2$)
Surface Tension	
Water	72
Asphalt	26 to 39
Various marbles of limestone	28 to 50
Diabase	42 to 50
Various granites	52 to 73
Interfacial Tension	
Asphalt-water	25 to 35
Quartz-water[1]	0
Quartz-asphalt	14 to 20
Glass-tar	18
Adhesion Tension	
Limestone-asphalt	21 to 26
Slag-asphalt	23 to 26
Sand-asphalt	22 to 30
Sand-tar	40
Quartz-asphalt	75
Limestone-water	58 to 64
Slag-water	63 to 68
Silica sand-water	83

[1] Based on the assumption that the surface consists of a water film.

has an advantage over asphalt in rapidly satisfying energy demands of polar aggregate surfaces. However, asphalt dipoles may have a more powerful energy demand for some aggregates than do water molecules, and may be able to displace water from the surfaces of these aggregates. Surface conditions at different spots on the same aggregate particle may vary appreciably in relation to a given film of asphalt, and adhesion of the asphalt may be stronger over some areas than over other areas.

Mechanisms of Stripping

An asphalt mixture is a system composed of asphalt cement, aggregate, and air. Water may be present because of incomplete drying of the aggregate or may be derived from external sources after construction. Stripping in this system occurs when the bond between the aggregate and asphalt is broken by water. Stripping is the reverse of the adhesion process. Water through some mechanism causes the bond between aggregate and asphalt to be diminished. Five mechanisms of stripping have been advanced: detachment, displacement, film rupture, hydraulic scouring, and emulsification (58, 70, 71).

Detachment. Detachment occurs where the asphalt cement is separated from the aggregate surface by a thin film of water but there is no obvious break in the continuity of the asphalt coating. The water may be present in the capillary pores of the aggregate because of improper drying, or because of the diffusion of water through the asphalt layer. In this state, the asphalt film can be peeled cleanly away from the aggregate. Hughes et al. (63) suggest that the crystal lattice of the mineral reacts with water to form a gel-like structure and detachment is partially due to the rupture of this weakened structure.

Displacement. Mathews (57) states that it is unusual for the bond between a binder and an aggregate to fail at the interface for reasons other than the displacement of the binder by water. Lee (64) performed experiments to measure the equilibrium forces which act on the binder-water-aggregate system. He attributed the displacement of binder to the superior wetting properties of water.

The surface energy necessary to strip asphalt from aggregate is reported by Thelen (60). His work is repeated here as a numerical example. Normal stripping is where a discontinuity or break in the asphalt coating occurs and asphalt, free water, and aggregate are all in contact. The free energy change is:

$$\Delta F = \gamma_{12} + \gamma_{24} - \gamma_{14}$$

where:

ΔF = free energy change

γ_{12} = interfacial tension, aggregate-asphalt

γ_{24} = interfacial tension, water-asphalt

γ_{14} = interfacial tension, aggregate-water

from data of Table VII,

$\gamma_{12} = 17 \text{ ergs/cm}^2$

$\gamma_{24} = 30 \text{ ergs/cm}^2$

$\gamma_{14} = 0 \text{ ergs/cm}^2$

then

$\Delta F = 17 + 30 - 0 = 47 \text{ ergs/cm}^2$

which is the energy potential to cause stripping. The rate at which the displacement occurs depends on the magnitude of the free energy evolved, and the viscosity of the asphalt. High viscosity binders show higher resistance to displacement; thus road surfaces that are subjected to rain before they attain their desired viscosity may show a higher degree of stripping (57). Displacement fails to explain how stripping is initiated when the aggregate is completely coated with asphalt.

Film Rupture. This method of stripping may occur when adhesion of the asphalt cement is not uniform over the entire surface of the aggregate (56). Mathews (57) suggests that the asphalt film is thinnest at sharp corners and edges of the aggregate and the effect of traffic may cause the film to fracture, thus initiating stripping. Once ruptured, and in the presence of water, the asphalt film would tend to take up the form of lowest potential energy by retracting to spherical globules (63).

Hydraulic Scouring. This theory of the mechanism of stripping is presented by Stevens (70). He states that voids in a bituminous mix or seal-coat consist of a largely interconnected pore system which is partially filled with air and water. In a saturated pavement, on impact of a wheel, water is pressed into the pavement in front of the tire and is sucked out as the tire leaves the spot; rapid decompression occurs within the pore system, and a small upward thrust is developed. This movement of water due to the action of traffic would tend to strip hydrophilic aggregate spontaneously, and dust may become mixed with rainwater and assist in abrading the asphalt-aggregate contacts. This theory may explain failure of pavements in the field due to action of water and traffic.

Emulsification. This mechanism has been observed to occur in the laboratory by Fromm (71). The water causes spontaneous emulsion formation as droplets migrate through the asphalt coating to the aggregate. The emulsion is a greyish material which forms on the asphalt surface, and is a water-in-oil emulsion. The rate at which the emulsion forms depends on the chemical nature of the asphalt cement. Fromm reports the use of three commercial anti-stripping additives which were tested; they actually increased the emulsion formation.

Summary. No one theory of adhesion or mechanics of stripping can explain the phenomenon of stripping. The resulting physico-chemical forces acting at the interface between asphalt cement, aggregate, water, and air may promote stripping. The displacement and detachment mechanisms can be considered the primary causes of stripping, and film rupture and hydraulic scouring the secondary causes. Adhesion of asphalt cement to the aggregate seems to be controlled by the characteristics of the aggregate, the asphalt cement characteristics being secondary.

Tests for Stripping

Asphalt-aggregate systems are divided into two types, two dimensional and three dimensional, by Majidzadeh and Brovold (58). The two-dimension binder-aggregate system consists of mixtures in which the asphalt and uniform sized aggregate form a layered and laminated structure. These can be thought of as surface treatments or seal coats. In the second type of system large numbers of well-graded aggregate particles are held together in a coherent three-dimensional honeycomb structure by the asphalt cement. Examples of this asphalt-aggregate system are hot-mix asphalt concrete or road mix type mixtures. Stripping tests are discussed on the basis of whether the bituminous mixture is considered two-dimensional (layered system) or three-dimensional (compacted bituminous mixture).

Tests devised for the layered system of construction have a common procedure. The aggregate to be evaluated is usually of one gradation, commonly sized to pass a 3/8 in. sieve and be retained on a 1/4 in. or No. 4 sieve. The aggregate is coated with the asphalt material, subjected to the effects of distilled water, and then evaluated to ascertain the percentage of asphalt coating still adhering to the aggregate. Several different detailed procedures have been proposed. Four types reviewed here are: dynamic immersion stripping test, static immersion stripping test, chemical or boil test, and detachment test. The laboratory test track, which is discussed with the compacted bituminous test, has also been used to evaluate stripping tendencies of the layered system of construction.

Compacted bituminous mixture tests are used to evaluate the water resistance characteristics of the entire compacted bituminous mixture. They include: immersion-compression (I-C), laboratory test tracks, vertical swell, water susceptibility, abrasion weight loss, and sonic vibration. Each test provides some measure of the change in a physical property of the mix due to the effects of water. This change in physical property then is related to stripping effects of water on the bituminous mixtures over stripping tests of coated aggregate particles include: 1) the test results are quantitative values, 2) the compacted test specimens represent the actual bituminous mixture that will be used in highway construction, and 3) the laboratory specimens are subjected to water action which may simulate actual field conditions, where the pavement mass is exposed to effects of rain and groundwater.

These advantages are offset somewhat by the need for more elaborate test equipment and test procedures to achieve satisfactory test results.

Dynamic Immersion Stripping Test. Nicholson (59) devised a dynamic immersion stripping (DIS) test to evaluate the adhesion of asphalt to various types of fine aggregates. His test mixtures represented sheet asphalt and typically were composed of 12 percent limestone dust (passing No. 200 sieve), coarse concrete sand, and 8 percent asphalt. He weighed out 50 g of the cooled friable mix and placed it in a 250 ml Pyrex Erlemeyer flask. About 175 ml of warm (140 F) distilled water was added to the flask before it was placed in a shaking machine. The machine was rotated at 39 rpm submerged in a water bath whose temperature was held at 140 F during the one-hour test period. The sample then was removed from the machine and the contents examined visually to see if the asphalt had washed off the aggregate.

Dow (72) used the same type of test to evaluate his Colprovia Paving Mixture in 1936. His mixtures were shaken at intervals of 1, 3, 5, 10, 15, and 30 minutes. The mixture was evaluated at the end of each interval and any mixture which did not strip at the end of the 30-minute period was passed as satisfactory. Dow ran tests on various component parts of the aggregate used in his mixture to determine which aggregate size contributed to stripping. Winterkorn et al. (62) used a similar test procedure but varied the test temperature and rate of rotation, and extended the time of rotation up to four hours.

Lang and Thomas (73) also used the DIS test devised by Nicholson. Their report gives the results of tests on 6 different types of aggregate with 23 different brands of asphalt cement (85-100 penetration grade). The samples were rotated at 44 rpm for two 15-minute periods at 75 F, and for additional 15-minute periods at 100 F and 120 F. At the end of each 15-minute period, the samples were examined and the visually estimated percentage of surface area of the aggregate which had been stripped was recorded. Two of the same aggregate types also were subjected to stripping after coating with a 50-60 penetration grade asphalt cement. Considerable variation in the stripping tendency of different asphalt cements was noted. Average results of these stripping tests are shown in Table VIII.

Critz and Goode (74) used a DIS test to evaluate the effect of using additives to decrease the stripping of bituminous coated aggregate. They concluded that the conditions of the test were not sufficiently severe to demonstrate differences between the additives or the effect of using different percentages of additive.

A modification of the Nicholson DIS test was used by Tyler (65), to increase the stripping effect on the bituminous coated aggregate. In this method the coated aggregate, after curing, was separated by hand and immersed in a mason fruit jar containing 1000 ml of distilled water. The jar was placed in a Ro-Tap sieve-shaker and agitated for 30 minutes. The samples were removed from the jar and graded according to visual inspection and count. Tyler concluded that aggregate composed of granite, quartz, and some cherts were hydrophilic and aggregate types such as dolomite, traprock, limestone, and basalt were hydrophobic.

Sanderson (75) used the Tyler test in evaluating the stripping resistance of seven different aggregates. These aggregates were also treated with methylchlorosilanes to determine whether their stripping

Table VIII

Dynamic Stripping Test Results (73)

Aggregate	Per Cent Coating: Penetration Asphalt Cement 50-60	Per Cent Coating: Penetration Asphalt Cement 85-100
Granite	--	34
Quartzite	75	53
Feldsite	--	65
Traprock	--	91
Limestone	99	95
Mixed Gravel[1]	--	99

[1]Mixed gravel composed of 32% limestone, 39% granite, 27% traprock, 1% sandstone, and 1% quartzite.

resistance would be increased. Sanderson concluded that the Tyler test did not simulate actual road conditions. Because the particles of aggregate were agitated violently in a jar of water, it seemed to him that most of the stripping which occurred was caused by mutual abrasion of the aggregate particles.

The State of California Highway Department laboratory test manual describes a dynamic immersion method of test to evaluate film stripping (76). The test is applied to the aggregate fraction passing the 3/8 in. sieve and retained on the No. 8 sieve. After curing, the coated sample is immersed in water at 77 F and agitated for 15 minutes. Additional 15-minute periods of agitation successively at 100 F and 120 F are used in special cases.

Static Immersion. In 1938, Hubbard (56) suggested a test to evaluate resistance to film stripping. The procedure was to prepare laboratory samples with the same proportions of asphalt and aggregate as those intended for the mixtures in the field. After mixing was completed, the blended materials were spread in a thin layer and allowed to stand for 24 hours. A suitable amount of the cold mixture then was placed in a glass jar with a screw top lid, and completely covered with distilled water. The jar and its contents remained undisturbed for 24 fours, after which the mixture was examined for evidence of film stripping. The jar then was vigorously hand shaken for three periods of 5 minutes and the mixture was examined at the end of each period. If only a slight amount of stripping was noted at the end of the third period of shaking, little or no trouble was anticipated with stripping under ordinary field conditions. In 1936 Lee (64) had reported results of a jar test similar to the one described by Hubbard.

A water-asphalt preferential test intended to determine the water-resisting properties of mineral fillers used in a bituminous mixture was described by Stanton and Hveem (77). This test consisted of mechanically mixing 50 ml of a heavy fuel oil heated to 140 F with 10 g of filler dust (passing the No. 200 sieve) for 5 minutes. Then 100 ml of distilled water at 140 F was added and the mixture stirred 5 additional minutes. The jar was set aside to allow the sample to settle until the water became clear. The amount of clean filler in the bottom of the jar was estimated and recorded as the percentage of total filler. Higher percentages of a poor quality filler separated from the oil and collected at the bottom of the jar.

An evaluation of stripping test methods and their usefulness was published by Holmes in 1939 (78). He divided the various tests into three categories: partition tests, displacement tests, and abrasion-displacement tests. The partition test of Holmes was similar to the water-asphalt preferential test previously described by Stanton and Hveem, and the abrasion-displacement test was similar to the dynamic immersion test where the coated aggregate was shaken in a horizontal plane. Holmes thought both of these tests were of little value in evaluating aggregate stripping tendency.

The displacement test was classified by Holmes as the one most frequently used at that time and was essentially the same test used by Hubbard and Lee, with slight modifications. The coating ability of the asphalt before immersion as well as the stripping resistance at the end of the test were noted. For the water-displacement test at 140 F, Holmes coated No. 4 to No. 10 sieve size material with 4 percent cutback asphalt. The mixture was cured 2 days at 140 F, then completely covered with distilled water for 18 to 20 hours at 140 F. The specimen was cooled to room temperature and spread out and dried. The percentage of aggregate still coated with asphalt was determined by visual estimation.

The influence of the pH of water on asphalt stripping from the aggregate was reported by Gzemski (79). The static immersion test previously described by Hubbard was used in the study. The coated aggregate was cured 48 hour at 110 F and then immersed at 77 F in a water solution (whose pH was varied from 40 to 10) for 24 hours. The percentage of asphalt coating that remained on the aggregate at the end of the immersion period was determined by visual estimation while it was still under water. Low pH solutions favored the retention of asphalt on the hydrophilic aggregate (rhyolite and granitic gneiss), whereas with dolomite better retention was obtained at pH values of 8 to 10. The pH of natural waters was surveyed to determine the extent of the variation of this factor. In Pennsylvania during 1944-45, the pH of surface waters varied between 2 and 10. Most of these pH values were in the range of 4.5 to 8.5.

Carroll (80) reports that rainwater in equilibrium with the CO_2 of the atmosphere (as in clouds) has a pH of 5.7, and in the United States the pH of rainwater is generally between 6 and 7. According to Carroll, all water that comes in contact with rocks is slightly acid. When mineral grains are crushed and placed in water, the pH obtained is known as the "abrasion pH". Carroll reports abrasion pH values for common minerals as: quartz, pH 6-7; feldspar, pH 8-9; amphiboles, pH 10-11; pryoxene, pH 8-10; mica, pH 7-9; calcite and colomite, pH 8-10; and clay minerals, pH 6-7. The test results of Gzemski perhaps reflect the tendency of a bituminous material to adhere better in the presence of a water whose pH is compatible with the abrasion pH of the aggregate to which it is applied.

A report of ASTM Subcommittee B-26 on the effect of water on bituminous coated aggregate was published in 1952 (81). A survey of current methods was used as a basis for proposing a standard method of performing the static immersion stripping (SIS) test. This standard method of test was adopted by ASTM, after continued review, in 1959 (82).

The ASTM procedure is applicable to cutback, emulsified, and semi-solid asphalts and tars. The method consists of coating 100 g of pre-washed, selected aggregate (passing the 3/8 in. sieve and retained on the 1/4 in. sieve) with 5.5 percent of liquid or semi-solid bitumen or 8 percent of emulsified asphalt. Temperature of mixing is as required to obtain a satisfactory coating and curing conditions are compatible with the type of bituminous material employed. The coated aggregate is immersed for 16 to 24 hours in distilled water of 6.0 to 7.0 pH at room temperature (77 F). By observation through the water, from above, the percentage of the total visible area of the aggregate which remains coated (above or below 95 percent) is estimated. This method should not be used as a field control measure because such correlation has not been established.

Karius and Dalton (83) evaluated the stripping tendencies of aggregate used for seal coats with a detachment test. The aggregate (5/8 in. to 1/4 in. size) was placed on the prepared bituminous film under dry conditions. After curing 16 hours at 68 F, any loose aggregate was removed from the pan, and the weight of the attached aggregate determined. The test pans were immersed in a water bath at 68 F and the detachment of the asphalt was observed daily for a period of 30 days. Percentage detachment was determined on a weight basis.

Boiling of Chemical Immersion. This type of test was originally developed by Riedel and Weber (84). The test involved placing a sample of bituminous coated aggregate in boiling water for 1 minute and noting whether separation of the bituminous material occurred. The aggregate was classified as hydrophobic if there was no separation. The degree of adhesion then was determined by the resistance to stripping of the bituminous films when boiled for 1 minute in sodium carbonate solutions of increasing concentration. A fresh sample of coated material was used with each solution. The numerical value of adhesiveness (on a scale of 1 to 10) was defined by the concentration of sodium carbonate solution at which the stripping occurred.

Winterkorn et al. (62) compared results of the Riedel and Weber test method with those obtained by using a dynamic immersion stripping test. They observed that the two tests gave comparative results, but the dynamic immersion stripping test approached road conditions more closely than did the boiling test. Other researchers have objected to the use of the boiling test because subjection of the sample to such high temperatures and exposure to the action of sodium carbonate solutions bear no relation to normal road conditions (57, 75).

Holmes (78) reported a water-boil test in which the coating of the aggregate was accomplished in the same manner as in his water-displacement test at 140 F. Then the coated aggregate was placed in a beaker and covered with distilled water. The beaker contents were heated to boiling in 6 minutes and boiled for 1 minute. The boiling sample was then cooled under running water and spread on a flat surface. The amount of coated surface was estimated visually.

Quantitative Methods to Determine Film Stripping. A problem common to the previously enumerated stripping test procedures has been the visual estimation of stripped surface area. Visual estimation provides only a qualitative indication of the stripping tendency of an asphalt-aggregate mixture and depends largely on the judgment of the operator, i.e., the individual performing the test. Considerable research effort has been expended to develop a quantitative method to measure the amount of stripping. The procedures include the use of radioactive isotope tracer, lithium tracer-salt, dye adsorption, mechanical integration, and leaching and stripping coefficient for "quantity" measurements.

These tests have been considered either too complex, requiring extensive equipment and expertise, or their accuracy has been questioned because results were based on the operator's visual judgment of the amount of stripping. Ford et al. (85) reported a better procedure to provide quantitative results and eliminate the judgment aspect of the amount of stripping.

In this study, an apparatus and a technique were developed to measure the amount of exposed surface area on asphalt-coated mineral aggregate particles after they had been subjected to the "stripping" effects of water. The test procedure is based on the principle that calcareous or siliciferous minerals will react with a suitable reagent and create a gas as part of the chemical reaction products. Within reasonable time limits in a sealed container, the generated gas creates a certain amount of pressure that can be considered proportional to the mineral surface area exposed to the reagent.

With proper selection of reagents and reagent concentrations, asphalt, being a relatively inert substance, will not enter into the reaction and will not contribute to the created gas pressure. By use of duplicate aggregate samples, one uncoated and the other asphalt coated and partially stripped, the change in gas pressure of the respective samples can be compared to determine the amount of exposed surface area on the partially coated sample.

This procedure was used to measure the amount of stripping shown by 11 different aggregate-asphalt mixtures. The aggregates, obtained from various Oklahoma sources, included several different types of carbonate and siliceous materials. The quantitative results of the surface reaction test were compared with visual evaluations of similar mixtures which were subjected to static and dynamic immersion stripping procedures and are shown in Table IX.

Immersion-Compression Tests. Developmental work on this method was reported in 1943 by Krchma and Loomis (86). They compacted their mixtures by using a vibratory procedure. The 3 in. diameter by 2 in. high specimens were cured 16 hours at 77 F before being tested in unconfined compression to determine their initial or dry strength. Duplicated specimens were allowed to stand in water at 77 F and were tested in compression after different periods of time. Three types of aggregates were evaluated with a 142 penetration Wyoming asphalt cement. Asphalt coated aggregate samples also were subjected to the dynamic stripping, boil, and swell tests.

The unexpected failure in 1941 of an experimental highway built in Colorado was attributed to the stripping effects of water. This failure prompted research on stripping by the Public Roads Administration. Pauls and Rex (87) reported the work undertaken by that agency in

Table IX

RESULTS OF STATIC AND DYNAMIC IMMERSION STRIPPING TESTS AND SURFACE REACTION TEST (85)

Aggregate	Static Immersion Ret. Coating (%)		Dynamic Immersion Ret. Coating (%)			Surface Reaction Ret. Coating	
	77 F	140 F	1 hr.	2 hr.	4 hr.	(%)	Variation (%)[1]
Cooperton	100	85	95	90	85	90	± 0.5
Hartshorne	100	75	95	90	75	85	± 0.5
Stringtown	100	65	95	90	85	93	± 0.7
Cyril	100	60	90	80	75	64	± 0.7
Keota	100	50	95	90	80	56	± 0.0
Onapa	100	50	95	90	85	68	± 0.8
Asher	100	90	95	90	80	74	± 3.4
Broken Bow	100	90	95	90	70	54	± 3.6
Gore	100	40	90	85	65	65	± 2.2
Hugo	100	95	95	90	80	78	± 0.5
Miami	100	70	95	85	75	60	± 3.5

[1]Variation based on maximum and minimum values of duplicate tests.

developing an immersion-compression test. They used the same source of Colorado aggregate as used in the test highway and compared results with a Potomac River sand and gravel of known quality. Initially they performed DIS tests on the aggregates under study. After testing, the Colorado aggregate showed only 40 to 50 percent of its surface area had retained its asphalt coating, whereas 95 to 100 percent of the surface area of the Potomac River gravel remained coated. With similar mixtures, cylindrical specimens 3 in. in diameter and 3 in. in height were molded at room temperature under a molding pressure of 1000 psi. This molding pressure was maintained for a 1-minute period.

Specimens from each mix were tested in compression after molding and after immersion in water for periods of 1 to 7 days at a temperature of 77 F. The Colorado aggregate mixture showed a retained strength from 0 to 5 percent; the Potomac River sand and gravel mixture had a retained strength ranging from 87 to 103 percent. This method of test was considered so promising that its use was extended to evaluation of other types of aggregates, filler, and asphalts.

Further development of the I-C test were reported by Pauls and Goode (88, 89). The sensitivity of the test was to be increased until the differences in hot mixtures in the laboratory agreed with field observations. They evaluated I-C retained strengths of bituminous mixtures by using 1) vacuum process to accelerate the saturation of the compacted mixture, 2) higher water bath temperatures, and 3) different immersion times.

In the vacuum process, the specimen was kept immersed in a water bath under 27 inches of mercury vacuum until air bubbles ceased to come from the surface of the specimen. This step required about 20 minutes; then the vacuum was reduced to atmospheric pressure, which forced water into the empty voids of the specimen. Swell and strength tests indicated that no initial detrimental effect resulted from vacuum saturation. However, when comparative specimens were vacuum saturated and soaked in the regular manner, a difference was noted in their retained strength. The vacuum process was not severe for hot mixtures of the coarse-grained type, but the fine-grained mixtures were affected to a much greater degree. It was concluded that the degree to which vacuum saturation affected values of retained strength was related to the fineness of the aggregate used in the mixture. Because fine-grained mixtures normally have high resistance to the infiltration of moisture and usually show good service behavior, it was concluded that the vacuum process should not be used in the I-C test procedure.

A definite relation between temperature of immersion water and time of test was observed. This study evaluated the effect of immersion temperatures of 77, 100, 120, and 140 F, along with periods of immersion of 5 hours and 1, 4, 14, and 35 days. The best correlation between field service behavior and I-C results was obtained with 4 days of immersion at 120 F. However, results of immersion tests at 140 F for 1 day and 120 F for 4 days showed a very close agreement. Therefore, results obtained with 1 day immersion at 140 F were considered acceptable.

The Standard ASTM Method D 1075, for effect of water on cohesion of compacted bituminous mixtures, was adopted in 1954 (90). In this test the bituminous mixture is evaluated by preparing 6 specimens, 4 in. in diameter and 4 in. high. The specimens are molded by means of a double plunger device, the final pressure of 3000 psi being applied for 2 minutes. The specimens are cured 24 hours at 140 F in air after which their bulk specific gravity is determined. The 6 specimens are then sorted into two groups so that the average specific gravity of each group is about the same.

Dry specimens are then brought to 77 F and tested in axial compression at a uniform rate of vertical deformation of 0.05 in. per minute for each inch of specimen height. Wet specimens are placed in a 120 F water bath for 4 days (alternate method of 140 F water bath for 1 day) then cooled in a 77 F water bath at least 2 hours before determination of their compressive strength. The index of retained strength is calculated by dividing the compressive strength of the wet specimens by the compressive strength of the dry specimens.

A similar test procedure has been published by the U.S. Army Corps of Engineers (91). Eight Marshall specimens are molded for each test. The standard size of specimen is 4 in. in diameter and 2.5 in. in height. Specimens are divided into two groups so that the average specific gravity of each group is essentially the same. Dry specimens are immersed in a 140 F water bath for not less than 20 minutes and their stability is determined by using a Marshall stability procedure. Wet specimens are immersed in a 140 F water bath for 24 hours prior to determination of their Marshall stability. The index of retained strength is obtained by dividing the wet Marshall stability by the dry Marshall stability. Mixes showing an index of retained stability of less than 75 percent are rejected.

Swanberg and Hindermann (92) reported a variation of the ASTM immersion-compression procedure in which they used specimens 4 in. in diameter and 2 in. in height. The standard testing procedures were followed except the strength of the specimens was determined by using the Marshall Stability Testing Head. Mathews et al. (93) also used this procedure in their 1965 work.

Olsen (94) reported a study in which the I-C test was performed with both the ASTM test procedure (90) and the Marshall stability test procedure (91). A higher percentage retained strength was obtained with the Marshall I-C method than with the ASTM I-C method. Eager (95) noted that in comparing indices of retained stability, the method of compacting and testing the specimens must also be considered. Specimens molded by kneading action, such as by the Hveem Kneading Foot Compactor, show significantly higher stability values as well as higher indices of retained stability than specimens compacted by the ASTM standard double plunger direct compression method.

The 1973 Annual Book of ASTM Standards (41) includes a proposed method of test for effect of water on resistance to plastic flow of bituminous mixtures with the Marshall apparatus. For the mixture to be evaluated, a minimum of 8 test specimens 4 in. in diameter and 2.5 in. in height are prepared in accordance with Standard ASTM Method D 1559 (41). The specific gravity of the test specimens is determined by the Standard ASTM Method D 2726 (41) and then they are sorted into two groups, one to be tested "dry" and the other to be tested "wet". Dry specimens are immersed in a 140 F water bath for not less than 30 minutes and their Marshall stability is determined. The wet specimens, if molded with asphalt cement, are immersed in a 140 F water bath for 24 hours and their Marshall stability is determined. The numerical index of the resistance to flow is obtained by dividing the wet Marshall stability by the dry Marshall stability, and expressing the retained strength as a percentage.

Some criticism has been directed at the I-C test method.

Sanderson (75) states, "The immersion-compression test is long, involved, and requires special apparatus not usually found in a highway engineering laboratory. The results of the test are somewhat dubious, even when performed with the best laboratory equipment." Goldbeck (96) notes that there was not a good correlation between the traffic test in the circular track and the standard I-C test.

Andersland and Goetz (97) report that a bituminous mixture containing hydrophilic rhyolite had a higher I-C retained strength, at the end of one day immersion at 140 F, than either a gravel mixture or a limestone mixture. Their test results are shown in Table X. Paul Thompson, in the discussion to this paper, notes his observation that very dense specimens would not be completely saturated during the entire 4 days immersion time at 120 F. He recommends that the time for immersion of the specimens should be calculated from the time the specimens are entirely saturated by water, such saturation having been effected either by a vacuum saturation technique or by a preliminary water immersion.

One phenomenon reported by some investigators using the I-C test method deserves elaboration. In some instances the bituminous mixture being evaluated indicates a retained strength greater than 100 percent. When asked if there were instances of higher compressive strength being obtained after water immersion, Goode (98) replied, "Oh yes, we quite frequently get a retained strength greater than 100 percent, particularly with a mixture containing limestone aggregate". He theorized that the small amount of moisture absorbed by the test specimen actually creates tensile forces between the coated particles of limestone aggregate within the specimen which result in a higher

Table X

STRIPPING RESISTANCE OF BITUMINOUS MIXTURES
INDIANA AH TYPE SURFACE COURSE (93)

Days Immer.	Static Immersion[1] (% Coated--140 F)	I-C (% Ret. Strength)	Sonic Modulus (% Ret. Modulus)
	Lafayette Gravel		
0	100	100	100
1	73	90	96
3	60	88	94
5	58	84	94
7	55	82	93
9	55	77	92
	Greencastle Limestone		
0	100	100	100
1	77	91	94
3	73	84	92
5	70	77	91
7	70	72	90
9	70	64	89
	Massachusetts Rhyolite		
0	100	100	100
1	17	93	96
3	12	81	91
5	12	68	80
7	12	60	74
9	10	28	71

[1] Static immersion sample sized to pass 3/8 inch sieve and be retained on 1/4 inch sieve.

compressive strength than would be obtained with a dry specimen.

Laboratory Test Tracks. Circular test tracks have been developed to evaluate the durability of bituminous mixtures under the action of traffic. In 1936, Goldbeck (99) reported the results of investigations on single and double surface treatments, mixed-in-place construction, premixed cold laid pavements, and asphalt concrete. The mixtures were compared on the basis of depth of rutting. Test track results were also related to field observations of similar mixtures. Temperature and moisture conditions were varied during the evaluation of track specimens. Goldbeck was of the opinion that stripping was likely to take place more rapidly in warm weather than in cold weather because the asphalt is softer and would be stripped more readily in that condition.

In 1949, Goldbeck (96) reported a comparison of immersion-compression test results with laboratory traffic test results. Circular track stability and durability values were obtained by measurement of rut depths. The durability test track pavement was immersed 7 days at room temperature (65 to 75 F), then 20,000 passes of 1000 lb. wheel load were made with the track immersed. Six different asphalt concrete mixtures were tested, with immersion-compression retained strengths ranging from 71 to 105 percent. All of the mixes were judged very durable from the test track loading results.

Holmes (78) also gives an account of the use of a circular test track to evaluate the adhesivity of asphalt in bituminous mixtures. After compaction and curing of the bituminous mixture, the track specimen was submerged in water at 90 F and the asphalt-aggregate mixture broken down by running the test wheels over the pavement surface. The number of revolutions of the machine required to loosen

10 percent of the total mixture was chosen as the breakdown point. It was noted that duplicate test track results agreed within ± 15 percent. Test track results were correlated with static immersion stripping test results and the adhesivity of various bituminous mixtures was determined. Different additives were used in the asphalt to increase adhesivity. Holmes found the maximum improvement obtained with additives corresponded to a relative track life of 3.88 times that of untreated asphalt. He also observed that incorporation of lime into the bituminous mixture did not appear to alter the true adhesivity properties of the asphalt.

This work of Holmes was continued and enlarged by Klinger and Roediger (100). They attempted to establish whether a correlation existed among the SIS test, circular test track, and field performance results. They concluded that relative service life of the pavement was not predicted by the static immersion stripping test results or the circular track test results. Additives increased the stripping resistance of the mixtures tested by SIS test and circular test track methods, but no effect was evident in the pavements exposed to traffic.

An immersion wheel-tracking test developed to evaluate the significance of the traffic stresses in stripping of binder from aggregate was reported by Mathews and Colwill (101). The bituminous sample was immersed in water and subjected to the action of a reciprocating wheel running over its surface. The length of time for which the sample withstood this treatment without collapse was taken as the index of performance. The failure time for the 12 aggregates tested ranged from 1 minute to more than 48 hours. Mathews et al. (93)

later published the results of research work in which bituminous mixtures, using 16 different aggregates, were evaluated by the static, dynamic, boil, immersion-compression strength (by the Marshall Testing Head), and immersion wheel-tracking tests. These test results were compared with the road performance of the aggregates. An index of stripping was used by Mathews (102) to determine road performance, coupled with a 6-year field observation to determine when an aggregate exhibited appreciable stripping. He concluded that the best correlation with road performance of the aggregate in bituminous-macadam was given by the immersion wheel-tracking test.

Other Test Methods. Several other procedures have been used to evaluate stripping resistance of a bituminous mixture. In 1934, Stanton and Hveem (77) reported a swell test made on a compacted specimen of oil mixed with aggregate which represented the grading used in actual construction. The compacted mixture (4 in. in diameter by 2 in. high) was submerged in its mold for 24 hours and the amount of vertical swell was determined. The permissible amount of swell was a function of the surface area of the aggregate in the mixture. They considered the swell test to be the most reliable method of determining the probable effect of moisture on the road surface.

Skog and Zube (103) report development of a water susceptibility test for studying the resistance of a bituminous mixture to moisture. A sample of the design mix was compacted with a kneading compactor, cured 75 hours at 140 F, subjected to moisture vapor passing up into the specimen, and then tested for its Hveem stability and cohesion. The specimen resembled a standard Hveem specimen except for a 0.5 in. hole through its center. A minimum Hveem stability value for the mixture was specified.

An abrasion test to evaluate probable field performance of a bituminous mixture was reported by Swanberg and Hindermann (92). The compacted mixture, 2 in. in diameter and 2 in. high, was cured 24 hours at 140 F for 4 days. Specimens were cooled and their saturated surface dry weight determined, then cooled to 35 F in a water bath. The specimens were placed in a Deval abrasion machine filled with water at 35 F, and the machine was rotated for 33 minutes (about 1000 revolutions). The specimens then were weighed in a surface-dry condition and the percentage weight loss determined. Test results were compared with field performance results of similar bituminous mixtures. The researchers concluded that the abrasion test gave satisfactory correlation with field performance and reasonable agreement with immersion-compression test results.

Another abrasion test developed to subject the surface of a compacted bituminous mixture to dynamic water action along with simulated tire action was reported by Skog and Zube (103). A 4 in. diamter specimen, held in its mold and after pre-conditioning, was clamped in a special shaking unit. Water and 4 solid rubber balls were placed on the mold and subjected to shaking for a 15-minute period. Water temperature was maintained at 100 F during the test. The weight loss of the specimen and visual estimate of the amount of stripping were then determined. Test results were correlated with results for different sources and grades of asphalt and different aggregate sources. The abrasion loss varied with different asphalt sources when the aggregate type and source were held constant.

A non-destructive method of test for evaluating stripping resistance in compacted bituminous mixtures was reported by Andersland and Goetz (97). A sonic test method was employed with a beam specimen 12 in. long by 2.5 in. thick. The test specimens were molded and cured, and their initial sonic modulus of elasticity was determined at 40 F in a saturated dry condition. Specimens then were immersed in tap water at 140 F. At intervals of 1, 3, 5, 7, and 9 days the sonic modulus of elasticity of each specimen was determined. The percentage retained sonic modulus of elasticity then was calculated. Comparable results from the SIS test at 140 F and the I-C test as well as the sonic test results are shown in Table X. The researchers concluded that the sonic test might produce laboratory results that would correlate well with field performance results. However, no field evaluation of the mixtures studied in the laboratory were reported.

Another non-destructive method of test of compacted asphalt mixtures to evaluate stripping resistance was reported by Schmidt and Graf (104). They used the change in resilient modulus of mixes that were subjected to the effects of water to indicate the amount of stripping that occurred. The resilient modulus was determined by a dynamic testing method on specimens molded in the Hveem kneading compactor by the Standard ASTM Method.

A light pulsating load was applied through a load cell across the vertical diameter of the specimen, which caused a corresponding elastic deformation across its horizontal thickness. The dynamic deformation was measured with sensitive transducers requiring only a few grams of activating force. From these measurements the resilient modulus was calculated. It was noted that the same specimen can be used first to determine the resilient modulus and subsequently to determine other properties such as the Marshall or Hveem stability or Hveem cohesion.

The resilient modulus of the asphalt mixtures was evaluated for the effects of: 1) type of aggregate, 2) various water exposure conditions, 3) type of asphalt cement, and 4) influence of additives. The retained resilient modulus decreased for mixtures that had been exposed to the effects of water as the exposure temperature increased. The rate and extent of the retained resilient modulus were proportional to the concentration of water present in the specimens. Likewise, with a higher asphalt content the specimens indicated a higher retained resilient modulus.

CHAPTER III

MATERIALS AND PAVEMENTS INVESTIGATED

The asphalt used in this research is representative of that used in construction of Arkansas asphalt pavements. Standard test procedures used to determine its physical properties include: penetration, ductility, absolute viscosity, thin film oven, and specific gravity.

The aggregates selected for this research were sampled at 18 sources located throughout Arkansas. Of the 18 different aggregates sampled, 7 were limestone, 5 were sandstone, 3 gravel, 1 was novaculite, 1 was syenite, and 1 was synthetic aggregate. The physical properties evaluated from the aggregate include: specific gravity and absorption, Los Angeles abrasion, soundness, and acid insoluble residue. The stripping properties of the aggregate also were tested to evaluate the tendency of the asphalt to adhere to the aggregate in the presence of water.

The pavements selected as test sections for this study contain aggregates from 10 quarries or pits in central and western Arkansas. Of the 10 quarries represented, 2 produce limestone (ls), 3 sandstone (ss), 2 gravel (gvl), 1 novaculite (nov), 1 syenite (ns), and 1 an expanded clay synthetic aggregate (syn).

Asphalt Cement

The sample of asphalt cement used for this research was obtained from Lion Oil Company at El Dorado, Arkansas. The 25 gallons of asphalt used in this work was obtained from one storage tank in 5 gallon buckets. The material was classified as AC-20 viscosity graded paving grade asphalt cement.

The average physical properties of the asphalt cement are shown in Table XI. The material was tested for uniformity as each bucket was used.

Aggregates

The aggregates sampled represent the major sources of material available for asphalt pavement construction in Arkansas. The aggregates were obtained by visiting each quarry or the pit where stockpiles of each material were stored. Approximately 200-300 lbs. of material of each aggregate type was obtained, ranging in gradation from the 3/4 in. to the No. 10 sieve.

The aggregates are identified in Table XII as to location and general classification and Figure 10 shows where each aggregate is located in terms of county and state. The aggregates are identified by the town adjacent to their location. They are divided into six groups: limestone, sandstone, gravel, novaculite, syenite, and synthetic aggregate. The physical properties of the aggregates are shown in Table XIII.

Limestone

Limestone is a bedded sedimentary deposit consisting chiefly of calcium carbonate ($CaCO_3$) which yields lime when burned. Limestone is the most important and widely distributed of the carbonate rocks and is the consolidated equivalent of limy mud, a calcareous sand, or shell fragments (105).

Table XI

Physical Properties of Asphalt Cement

Characteristics	ASTM[1] Method	Test Value
Viscosity, 140 F, poises	D2171	1720
Penetration, 77 F, 100 g, 5 sec	D5	78
Ductility, 77 F, cm	D113	150+
Tests on Residue from Thin-Film Oven Test	D1754	
Viscosity, 140 F, poises	D2171	3210
Penetration, 77 F, 100 g, 5 sec	D5	57
Ductility, 77 F, cm	D113	150+
Average Weight Loss, % original	-	+0.052
Specific Gravity, 77/77 F	D70	1.021

[1] 1973 Annual Book of ASTM Standards, Part II

Table XII

Aggregate Identification and Source

ID Number	Sample	County	Location Sec.	Twp.	Rg.	Geological Unit or Period	General Classification
1	Twin Lakes	Baxter	25	20N	14W	Ordovician Age	Limestone
2	Valley Springs	Boone	33	18N	19W	Boone (F) [1]	Limestone
3	Hampton	Calhoun	27	14S	13W	Quarternary terrace	Gravel
4	Van Buren	Crawford	04	09N	31W	Hartshorne (F)	Sandstone
5	Kentucky	Crittenden	-	-	-	Unknown	Limestone
6	Malvern Novaculite	Hot Springs	09	04S	17W	Paleozoic Age	Novaculite
7	Rocky Point	Independence	34	12N	06W	Hale (F)	Limestone
8	Black Rock	Lawrence	13	17N	02W	Ordovician Age	Limestone
9	England	Lonoke	05	02S	09W	Arklite-Synthetic	Synthetic
10	Cabot	Lonoke	22	04N	10W	Atoka (F)	Sandstone
11	Texarkana	Miller	06	17S	28W	Quarternary terrace	Gravel
12	Murfreesboro	Pike	07	08S	25W	Quarternary Alluvium	Gravel
13	Russellville	Pope	35	08N	20W	Hartshorne (F)	Sandstone
14	Big Rock	Pulaski	08	01S	12N	Nepheline	Syenite

Table XII (concluded)

ID Number	Sample	County	Location Sec.	Twp.	Rg.	Geological Unit or Period	General Classification
15	Jenny Lind	Sebastian	28	07N	31W	Hartshorne (F)	Sandstone
16	Johnson	Washington	21	17N	30W	Boone (F)	Limestone
17	West Fork	Washington	30	15N	30W	Hale (F)	Limestone
18	Bald Knob	White	23	08N	06W	Atoka (F)	Sandstone

[1] (F) - Formation

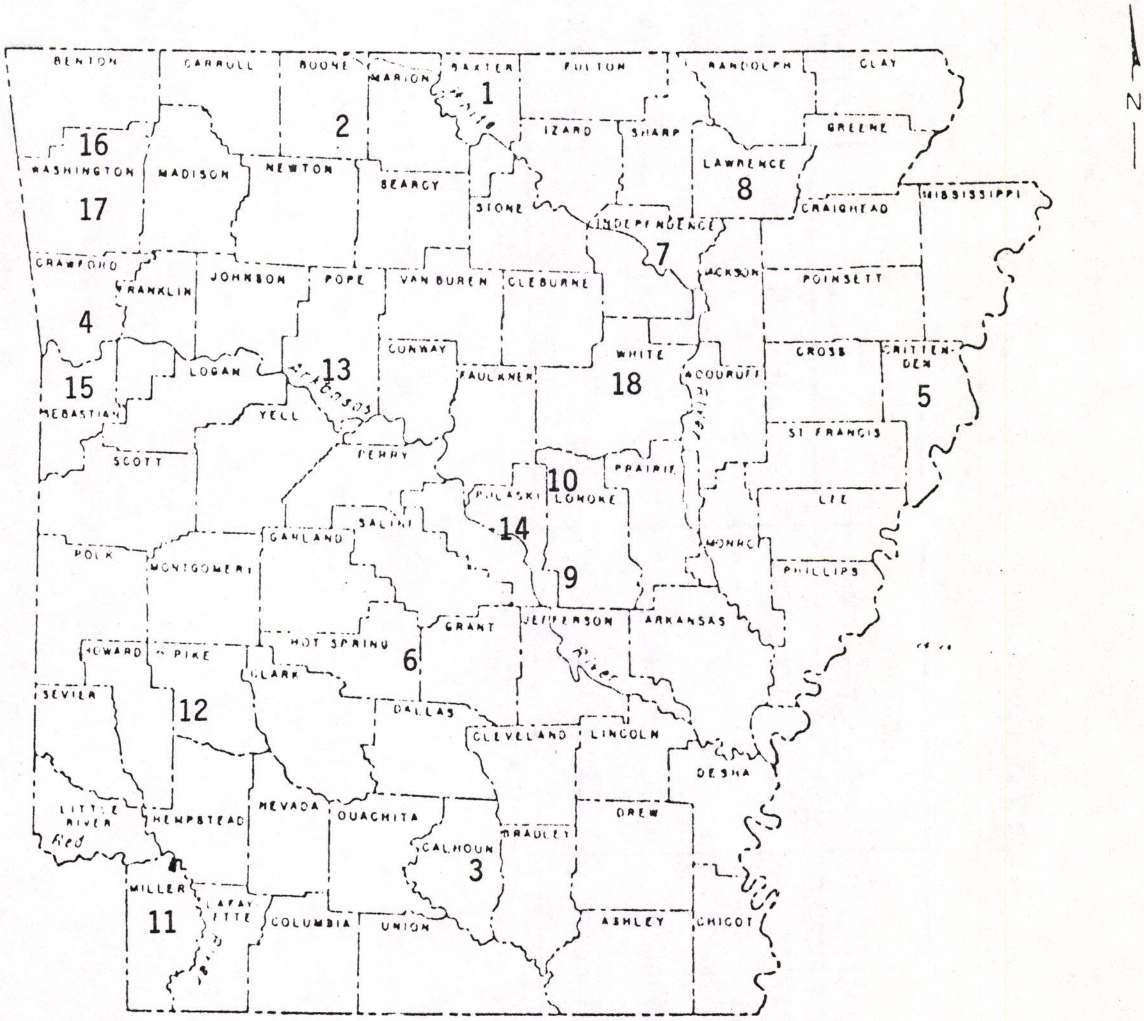


Figure 10. Location of the Study Aggregates

(For Aggregate Number Reference See Table XII)

Table XIII

Aggregate Physical Properties

ID Number	Sample	Bulk[1] Sp. Gr.	Absorption %	L. A. Abrasion[2] Grade C	Soundness[3] Na_2SO_4	% Insoluble Residue[4] + No. 200
1	Twin Lakes	2.606	2.9	24.9	5.8	9.0
2	Valley Springs	2.662	0.8	28.1	1.5	2.5
3	Hampton	2.518	1.6	20.9	1.8	98.8
4	Van Buren	2.533	1.8	26.5	2.0	95.6
5	Kentucky	2.651	1.1	20.6	2.4	2.7
6	Malvern Novaculite	2.518	1.5	22.0	0.8	99.9
7	Rocky Point	2.632	0.9	22.0	0.6	3.8
8	Black Rock	2.691	1.3	20.5	6.3	9.5
9	England	1.223	24.0	43.1	10.6	97.9
10	Cabot	2.588	1.3	19.6	0.7	96.7
11	Texarkana	2.613	0.6	21.2	0.3	99.6
12	Murfreesboro	2.544	1.4	21.3	0.3	99.6
13	Russellville	2.578	1.4	22.1	1.0	97.1
14	Big Rock	2.602	0.5	21.4	1.0	92.8

Table XIII (concluded)

ID Number	Sample	Bulk Sp. Gr.	Absorption %	L. A. Abrasion Grade C	Soundness Na_2SO_4	% Insoluble Residue + No. 200
15	Jenny Lind	2.514	1.9	23.6	0.4	98.2
16	Johnson	2.532	2.4	21.0	0.9	21.6
17	West Fork	2.654	0.9	24.7	0.6	0.7
18	Bald Knob	2.477	1.2	33.1	4.8	99.3

[1]Reference ASTM Designation : C 127, Part 10 - (Specific Gravity of 3/4" to No. 10 Material)

[2]Reference ASTM Designation : C 131, Part 10

[3]Reference ASTM Designation : C 88. Part 10

[4]Reference Oklahoma Test Method OHD-L-25 (Appendix A)

Arkansas has abundant limestone sources across the northern one third of the state. Seven limestones, Twin Lakes, Valley Springs, Kentucky, Rocky Point, Black Rock, Johnson, and West Fork were tested. The Kentucky limestone sample was from a quarry at Gilbertsville, Kentucky. The sample was obtained from the Warren Brothers Hot-Mix plant at West Memphis, Arkansas.

Sandstone

Sandstone is a sedimentary rock composed of noncarbonate grains 0.06 to 2.0 mm in diameter, which are cemented together in some fashion. The cementing material may be quartz, opal, calcite, dolomite, clay, or oxides of iron, either reddish (hematite) or yellowish (limonite). The colors are variable; white to gray, buff to dark yellow, and red to reddish brown are common (106). These colors depend largely on the nature of the cement.

According to McBride (107) many different classification systems for sandstone have been proposed, but none has been devised which adequately treats all of the important sandstone attributes. The sedimentary structure, texture, and composition of the sandstone are the three main characteristics used in his study. Composition is generally the most important feature for evaluation as a highway material.

Five sandstones, Van Buren, Cabot, Russellville, Jenny Lind, and Bald Knob, were studied. These sandstones are from the Hartshorne and Atoka formations which cover a large part of central Arkansas.

Gravel

Gravel is a loose or uncolsolidated coarse granular material, larger than sand grains. When such material is transported by running water, it is sorted according to the strength of the current. In some cases, beds are formed which consist of approximately equal sized particles. The particles which compose gravel are rock fragments and individual minerals.

The form and appearance of these pebbles depend on the conditions of erosion, transportation, and deposition. Those which have undergone considerable transportation are likely to have a very smooth surface with a characteristic faintly dimpled, slightly dented appearance caused by their repeated collisions during movement. If the pebble is composite, it commonly is pitted by weathering and removal of softer or more easily altered minerals (106).

Three gravels, Hampton, Texarkana, and Murfreesboro were tested. All three gravels are from the Quarternary terrace or Alluvium formation which covers a large part of the southern and eastern sections of the state.

Syenite

Syenite is a plutonic ingeous rock consisting principally of alkalic feldspar usually with one or more mafic minerals such as hornblende or biotite. The feldspar may be orthoclase, microcline, or perthite. A small amount of plagioclase may be present. A small amount of quartz is usually present but in some examples nepheline may take its place. Accessory minerals are sphene, apatitie, and opaque oxide (105).

The sample tested was from the Big Rock quarry in Pulaski County. The material was crushed blue nepheline syenite, with 60 percent orthoclase feldspar, 10 percent nepheline, 10 percent hornblende, 5 percent diopside, and 15 percent titanite, apatite, magnetite, or biotite.

Novaculite

Novaculite is a dense, hard, homogeneous, fine-grained sedimentary rock composed of pure silica. Minor impurities may be iron oxide, alumina, calcium carbonate, and clay minerals. It is translucent on the edges and has a marked conchoidal fracture.

Novaculite is very like chert, both in composition and its behavior as a road-making material. Originally, the term "novaculite" was applied to rocks found in the lower Paleozoic section of the Ouachita Mountains of Arkansas (105).

Synthetic

Arklite synthetic aggregate is expanded clay aggregate manufactured by rotary kiln process at 2000 F. It consists of angular fragments, reasonably uniform in density and relatively free from flat or elongate particles or other deleterious substances.

The aggregate is manufactured to have a Los Angeles abrasion of not more than 40, and when subjected to five cycles of the soundness test it shall have a loss not to exceed 30 percent (sodium sulfate). The Arkansas Lightweight Aggregate Corporation near England, Arkansas, in Lonoke County manufactures the aggregate.

Pavements Investigated

The highway pavements selected for field skid testing were chosen to represent each type of aggregate used in Arkansas pavements. A list of road paving projects completed in the last few years with the study aggregates was obtained from the Marshall Mix Design file of the Arkansas Highway Department's job file records. The pavements that were constructed from each source were identified on a county map. The help of district engineers, resident engineers, and materials engineers was used to locate the pavement surfaces that would provide a representative sample of the aggregate under investigation.

The highways chosen for skid testing evaluation included 10 of the study aggregates. Projects of at least two different ages constructed from each of the 10 sources were selected. The pavements tested were generally in the western part of the state, and ranged from the Missouri line on the north to the Louisiana-Texas line on the south.

The England synthetic aggregate skid test work was limited to only one road section, which was a surface treatment job. This synthetic aggregate does not meet the AHD specification (32) for use in the Arkansas Asphalt Concrete Hot-Mix Surface Course (ACHMSC) mixtures.

All of the pavements selected were constructed to meet the Arkansas specification for ACHMSC Type 2 mix. The gradation curve of this mix indicates that it is well-graded, with a top size of 3/4 in. material and with 4 to 10 percent material passing the No. 200 sieve. During the initial stages of field skid testing, a few pavements conforming to the AHD ACHMSC Type 3 mix (top size 1/2 in.) were added to the list of study pavements for comparison.

The pavements tested are all two lane roads with a range in daily traffic of 400 to 11,600 vehicles. The pavements for study were initially selected in 1975, and partially evaluated in June with the AHD skid trailer before it broke down. At that time, most of the pavements appeared to be in good condition and were satisfactory for investigation.

Test Pavement Locations

A total of 34 different pavement jobs were selected for field testing. The test sections are listed in Table XIV. Each section is identified by the name of the aggregate with which it was constructed, the route section and lane tested, and the AHD job number under which it was constructed. The listing also indicates the date of completion of the surfacing, which was taken from the asphalt inspector daily report.

The oldest pavement was constructed in May 1968 and the newest pavement was completed in September 1975. The field study aggregates and the number of pavement test sections, respectively, are: Johnson ls (4), Valley Springs ls (5), Cabot ss (3), Van Buren ss (2), Jenny Lind ss (6), Texarkana gvl (3), Murfreesboro gvl (4), Big Rock ns (4), Malvern nov (2), and England artificial (1).

Also shown in Table XIV are the beginning and ending road log mile between which skid tests were conducted, and the road log mile where BPT tests were performed. Each BPT site was numbered, and a total of 60 pavement sites were tested.

The breakdown of total miles tested with the skid trailer, by type of aggregate, is: limestone, 53.6 miles; sandstone, 63.4 miles; gravel, 42.6 miles; novaculite, 24.1 miles; syenite, 23.9 miles; and

Table XIV

Pavement Test Sections and British Portable Test Sites

BPT Test Site No.	Route - Section/ Lane	Job No.	Aggregate (type)	Log Mile		Skid Length (mile)	BPT* Test Sites (log mile)	Date Completed
				Start	End			
1,2	16-3/East	9-552	Johnson (ls)	4.3	10.4	6.1	6.8, 9.5*	10- 8-68
3	71-16/North	9563	Johnson (ls)	22.5	24.5	2.0	23.1	10-25-71
4	71-17/North	9563	Johnson (ls)	0.0	5.5	5.5	4.5*	10-25-71
5,6	45-5/East	4-769	Johnson (ls)	11.3	15.6	4.3	11.9*, 14.4	11- 7-75
7,8	62-2/East	9-567	Johnson (ls)	8.4	17.3	8.9	10.4*, 14.9	10-16-70
9,10	65-1/North	9-549	Val.Spr.(ls)	6.7	16.3	9.6	10.6, 15.3*	8-27-68
11,12	62-7/East	9-580	Val.Spr.(ls)	0.2	4.4	4.2	1.0*, 4.0*	12-29-70
13	62-7/East	9-617	Val.Spr.(ls)	4.7	9.8	5.1	6.5	11-29-73
14	65-4/South	9-545	Val.Spr.(ls)	5.0	10.6	5.6	5.2	5-31-68
15	65-4/South	9-554	Val.Spr.(ls)	11.2	13.5	2.3	12.4*	9- 5-68
16,17	5-15/South	5-594	Cabot (ss)	0.1	7.4	7.3	0.4, 5.5*	3-27-71
18,19	5-14/South	5-580	Cabot (ss)	5.3	16.7	11.4	8.8, 16.3*	11-10-70
20	5-12/South	6-635	Cabot (ss)	1.0	8.9	8.9	4.9*	3- 4-74

Table XIV (continued)

Pavement Test Sections and British Portable Test Sites

BPT Test Site No.	Route-Section/ Lane	Job No.	Aggregate (type)	Log Mile		Skid Length (mile)	BPT* Test Sites (log mile)	Date Completed
				Start	End			
21	71-15/North	4-651	Van Buren(ss)	3.7	7.2	3.5	5.6*	8- 5-71
22,23	162-1/West	4518	Van Buren(ss)	1.3	5.1	3.3	1.9*, 3.8	6- 4-74
24	71-10/North	4743	Jen.Lind(ss)	25.2	31.0	5.8	29.2*	7-23-75
25,26	71-13/North	4661	Jen.Lind(ss)	0.1	7.1	7.0	0.9*, 4.0	10-19-72
27	71-13/North	4547	Jen.Lind(ss)	7.2	10.3	3.1	8.0*	4-15-71
28	71-14/North	4547	Jen.Lind(ss)	0.0	1.6	1.6	1.0	4-15-71
29,30	71-14/North	4-687	Jen.Lind(ss)	1.8	7.2	5.4	4.4*, 5.3	4-15-71
31	96-3/East	4-681	Jen.Lind(ss)	2.6	5.5	2.9	2.8	10- 9-72
32	96-3/East	4-948	Jen.Lind(ss)	5.8	8.5	2.7	6.4*	3-28-74
33,34	71-1/North	3655	Tex. (gvl)	5.1	9.2	4.1	6.5*, 8.7	9-25-75
35,36	71-1/North	3632	Tex. (gvl)	9.4	14.9	5.5	9.7*, 12.7	12- 8-71
37,38	82-2/West	3-633	Tex. (gvl)	1.0	6.3	5.3	1.0, 3.8*	5- 8- 73
39	71-1/North	3623	Mur. (gvl)	15.2	17.3	2.1	16.3	4-20-72

Table XIV (concluded)

Pavement Test Sections and British Portable Test Sites

BPT Test Site No.	Route-Section/ Lane	Job No.	Aggregate (type)	Log Mile		Skid Length (mile)	BPT* Test Sites (log mile)	Date Completed
				Start	End			
40	71-2/North	3623	Mur. (gvl)	0.1	3.4	3.3	2.3*	4-20-72
41,42	32-3/East	3-644	Mur. (gvl)	2.2	11.0	8.8	5,1*, 7.4	4- 3-75
43,44	27-2/North	3741	Mur. (gvl)	7.0	11.9	4.9	8.3, 10.7*	5-18-73
45,46,47	4-3/West	3-622	Mur. (gvl)	1.0	9.6	8.6	1.2,6.2*,7.5	4-14-72
48,49	15-9/South	6-590	Big Rock(gm)	0.1	3.9	3.8	1.9, 2.9*	5- 6-70
50,51	88-9/East	2-626	Big Rock(gm)	0.3	10.3	10.0	4.5, 8.5*	6- 9-71
52,53	11-5/North	2-639	Big Rock(gm)	0.2	4.3	4.1	0.9*, 3.4	4-26-72
54,55	70-8/East	6-602	Big Rock(gm)	0.0	6.0	6.0	1.9*, 3.0	4- 5-71
56	5-7/South	6-602	Malvern(nov)	0.2	6.1	5.9	2.8*	4- 1-71
57,58	70-9/East	6-604	Malvern(nov)	5.3	17.3	12.0	10.7*, 11.7	7-21-71
59	70-10/East	6-604	Malvern(nov)	0.3	6.5	6.2	4.8	7-21-71
60	294-2/West	6-500	England(syn)	1.5	6.0	4.5	4.7*	-

* Indicates pavement core sample location.

synthetic, 4.5 miles. A total of 212 miles of pavement was skidded during the course of this investigation.

Pavement Mix Design and Traffic

The pavements were all constructed to meet the AHD specification for ACHMSC mixtures. Each mixture was designed in the Materials and Test Laboratory at the Central Headquarters in Little Rock.

One set of core samples were obtained from 32 pavement jobs for laboratory evaluation. The selection of the core site location was derived from BPT test sites which previously had been skidded with the AHD Skid Trailer, Arkansas No. 1, in June 1976.

The study pavements from which cores were obtained are shown in Table XV. The core sites are numbered in accordance with the BPT site number and are identified by road log mile. Study pavements which were not cored (listed respectively by aggregate name, job number, and route section) are: Valley Springs ls, job 9-617, 62-7; Valley Springs ls, job 9-545, 65-4; and Jenny Lind ss, job 4-681, 96-3. A total of 32 sites were selected for coring, but one pavement section (job number 9-580) was cored at both of its BPT test sites.

Table XV also contains the Marshall mix design data for each job cored along with the actual percentage of asphalt cement in the pavement mixture as reported on the "final" daily report of inspection at the asphalt plant. The January 1977 Average Annual Daily Traffic (AADT) for each core site was obtained from the AHD 1976 Traffic Volumes Map and is shown in Table XV. The estimated total number of wheel passes over each core site, which is discussed in another section of the report, is also indicated in Table XV.

Table XV

Pavement Core Sites - Mix Design Data and Traffic

BPT Test Site No.	Aggregate	Route-Section/ Log Mile	AHD Marshall Mix Design Data						Batch Plant[2] % AC	1977 AADT[3]	Total WP[4] ($x10^6$)
			Grade AC	Type[1] Mix	% AC	% Air Voids	Stab. (lb.)	Flow 0.01 in.			
2	Johnson (ls)	16-3/9.5	60- 70	2	5.3	3.8	1830	12	5.2	3800	10.232
4	Johnson (ls)	71-17/4.5	85-100	2	5.7	2.9	1810	12	5.5	11400	18.196
5	Johnson (ls)	45-5/11.9	AC-20	2	5.6	3.7	1720	12	5.4	1000	0.443
7	Johnson (ls)	62-2/10.4	60- 70	2	5.6	5.7	1620	13	5.6	3900	10.014
10	Val.Spr.(ls)	65-1/15.3	60- 70	2	5.3	3.3	1920	12	5.1	2800	7.215
11	Val.Spr.(ls)	62-7/1.0	60- 70	2	5.5	4.2	1840	13	5.6	3500	5.648
12	Val.Spr.(ls)	62-7/4.0	60- 70	2	5.5	4.2	1840	13	5.6	3500	5.648
15	Val.Spr.(ls)	65-4/12.4	60- 70	2	5.2	2.4	2160	13	5.1	2700	6.310
17	Cabot (ss)	5-15/5.5	60- 70	2	5.5	4.9	1840	12	5.3	1450	2.674
19	Cabot (ss)	5-14/16.2	60 -70	2	5.3	4.7	1610	12	5.4	1600	2.482
20	Cabot (ss)	5-12/4.9	60- 70	2	5.3	3.4	1230	11	5.3	2500	2.341
21	Van Buren(ss)	71-15/5.6	85-100	3	5.5	3.4	1650	11	5.3	7700	12.397

Table XV (continued)

Pavement Core Sites - Mix Design Data and Traffic

BPT Test Site No.	Aggregate	Route-Section/ Log Mile	AHD Marshall Mix Design Data						Batch Plant[2] % AC	1977 AADT[3]	Total WP[4] ($x10^6$)
			Grade AC	Type[1] Mix	% AC	% Air Voids	Stab. (lb.)	Flow 0.01 in.			
22	Van Buren(ss)	162-1/1.9	85-100	2	5.6	5.7	1720	12	5.6	4400	3.972
24	Jen.Lind(ss)	71-10/29.2	AC-20	2	5.4	3.3	1340	11	5.4	2850	1.682
25	Jen.Lind(ss)	71-13/0.9	85-100	2	5.4	3.3	1340	11	5.3	4500	6.231
27	Jen.Lind(ss)	71-13/8.0	85-100	3	5.7	3.9	1410	12	5.6	4100	7.609
29	Jen.Lind(ss)	71-14/4.4	85-100	2	5.4	3.3	1340	11	5.4	8700	9.626
32	Jen.Lind(ss)	96-3/6.5	85-100	2	5.4	3.3	1340	11	5.2	1400	1.388
33	Tex. (gvl)	71-1/6.5	AC-20	2	5.3	3.2	1100	11	5.3	2400	1.286
35	Tex. (gvl)	71-1/9.7	60 -70	2	5.3	3.1	1260	12	5.2	2550	4.525
38	Tex. (gvl)	82-2/3.8	60- 70	2	5.3	3.1	1260	12	5.4	3100	4.388
40	Mur. (gvl)	71-2/2.3	60 -70	2	5.5	2.8	1360	12	5.4	3550	6.031
41	Mur. (gvl)	32-3/5.1	60- 70	2	5.6	5.3	1390	12	5.6	1100	0.743
44	Mur. (gvl)	27-2/10.6	60 -70	2	5.7	4.1	1770	13	5.7	3450	4.125

Table XV (concluded)

Pavement Core Sites - Mix Design Data and Traffic

BPT Test Site No.	Aggregate	Route-Section/ Log Mile	AHD Marshall Mix Design Date						Batch Plant[2] % AC	1977 AADT[3]	Total WP[4] ($x10^6$)
			Grade AC	Type[1] Mix	% AC	% Air Voids	Stab. (lb.)	Flow 0.01 in.			
46	Mur. (gvl)	4-3/6.2	60- 70	2	5.6	N/A	-	-	5.7	1400	2.507
49	Big Rock(gm)	15-9/2.9	60- 70	3	5.8	4.1	1390	12	5.8	1800	4.240
51	Big Rock(gm)	88-9/8.5	60- 70	2	6.0	5.2	1260	12	6.1	1000	1.898
52	Big Rock(gm)	11-5/0.9	60- 70	2	5.9	3.7	1480	12	5.9	320	0.559
54	Big Rock(gm)	70-8/1.9	60- 70	2	5.8	5.9	1460	12	5.8	2700	4.204
56	Malvern(nov)	5-7/2.8	60- 70	2	5.6	4.2	1550	13	5.8	750	1.407
57	Malvern(nov)	70-9/10.7	60 -70	3	6.0	7.6	1220	13	5.9	4800	8.596
60	England(syn)	294-2/4.7	-	-	-	-	-	-	-	1200	2.968

1 Refers to AHD ACHMSC type.

2 From last report of inspection at the asphalt plant.

3 Average annual daily traffic.

4 Estimated total wheel passes to April, 1977.

CHAPTER IV

TEST METHODS AND EQUIPMENT

The primary pieces of equipment used for skid resistance evaluation in the field and laboratory testing were a locked-wheel skid trailer supplied by the Arkansas Highway Department, a British Portable Tester and an Accelerated Polishing Device (APD). In addition, a texturemeter and the regular sand patch test were used in the field to evaluate the pavement surface texture. The regular sand patch test with an extremely limited volume of sand was used for the texture depth measurements on the laboratory core specimens.

The durability test equipment employed in this study included the Marshall Testing Apparatus, the Dynamic Immersion Stripping Device, and the Surface Reaction Test Device. The study aggregates were tested both for their film stripping characteristics, i.e., as separate individual particles, and for their behavior when incorporated into compacted asphalt mixtures.

All laboratory and field test procedures were performed in accordance with applicable standard ASTM methods if possible. Otherwise, the test procedures and the equipment utilized are reported herein.

Field Tests

The pavements under investigation were evaluated by skid trailer tests. Two "Arkansas Skid Measurement Systems" (ASMS) or locked-wheel skid trailers were used during the field testing of the research program. During the summers of 1975 and 1976, a Soil Test skid trailer model number ML 350 HX, Serial No. B 27331, was used with a 3/4 ton Chevrolet truck as the towing vehicle. This skid unit is shown in Figure 11. The system named ASMS No. 1 was calibrated by the Texas Transportation Institute (TTI), in April 1973 (43).

In the third series of skid tests performed in May and June 1977 the second AHD unit, identified as ASMS No. 2, was used. This skid trailer is shown in Figure 12. The new system was designed and built by the Mechanical Engineering Department, College of Engineering, at the University of Arkansas. ASMS No. 2 was calibrated by TTI in October 1976 (108). This unit was towed by a 1 ton 1975 Dodge truck. Details of these skid units are given in the Texas Transportation Institute Reports (43, 108).

Skid numbers, for this research work, obtained from these two different skid trailers were corrected by use of the Texas Transportation Institute calibration equations:

For Field Tests Before April 1973 for ARMS No. 1

$$SN_{40}\ (\text{ARSMS}) = -4.88 + 0.94\ SN_{40}\ (\text{ARMS No. 1})$$

For Field Tests After April 1973 for ARMS No. 1

$$SN_{40}\ (\text{ARSMS}) = 1.85 + 0.90\ SN_{40}\ (\text{ARMS No. 1})$$

For Field Tests Before October 1976 for ARMS No. 2

$$SN_{40}\ (\text{ARSMS}) = -1.69 + 0.85\ SN_{40}\ (\text{ARMS No. 2})$$

For Field Tests After October 1976 for ARMS No. 2

$$SN_{40}\ (\text{ARSMS}) = -0.10 + 0.82\ SN_{40}\ (\text{ARMS No. 2})$$

where:

SN_{40} ARSMS = Skid Number Based on Area Reference Skid Measurement System

Figure 11. Arkansas Highway Department ASMS Number One

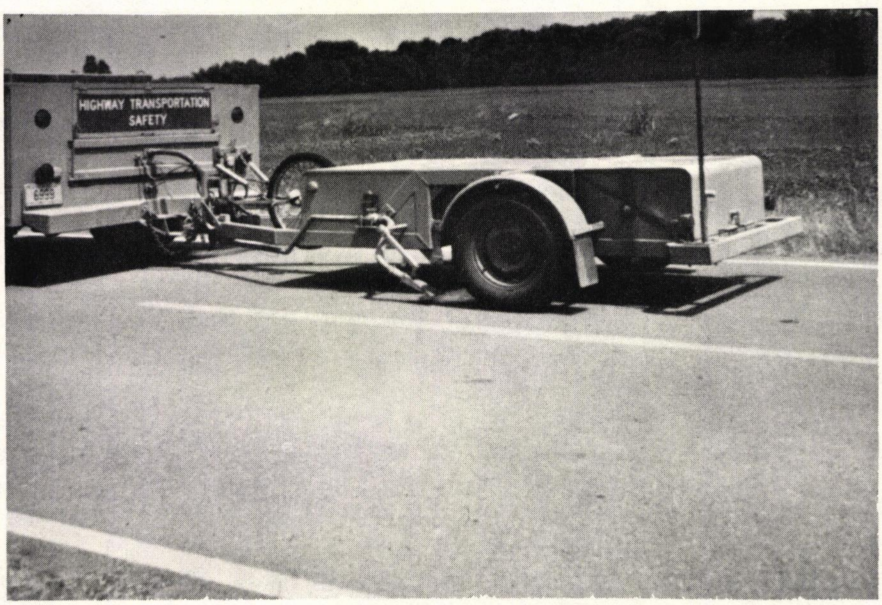


Figure 12. Arkansas Highway Department ASMS Number Two

SN_{40} ARMS (___) = Skid Number Measured with Arkansas Trailer

All field skid test readings were adjusted by means of the equations to relate them to the standard system.

Pavement Skid Testing

To find the range in skid numbers, the selected pavement first was tested about every 0.2 to 0.3 mile by the Arkansas Skid Trailer at 40 mph and the skid number (SN_{40}) and location by road log mile (L.M.) were recorded. The skid sites tested were in general on straight, level sections of the open highway. Briefly the test procedure is as follows: 1) the driver attains the test speed of 40 mph and holds this speed during the entire test sequence, 2) the operator presses the test button each time to initiate a test sequence, 3) the skid cycle of watering, locking wheel, and unlocking wheel is automatic and requires about 6 seconds, 4) the skid data are printed on tape.

In the initial test series, the wetted skid sites were marked with a numbered bag containing steel shot placed from a chase car. After completion of skidding along the test pavement, two BPT test sites were selected at points of high or low SN_{40} values. In all skid tests the inner wheel path (IWP) (left wheel path) was skidded.

Test Site Evaluation

The site chosen for testing with the BPT had been marked previously with a bag to help locate the skid tire mark. The sites were selected where ample sight distance was available because of the danger involved in stopping traffic, especially when testing the inner wheel path with the BPT.

Two flagmen were used to control traffic flow. The road log mile was recorded and "squares" were painted on the road surface at each BPT test point. The pavement was tested in the skid trailer tire mark at about 20 foot intervals with the BPT. The pavement was also tested in the outer wheel path (OWP) and between the wheel paths (BWP) adjacent to the IWP test sites.

A typical test site is shown in Figure 13. The operator has tested in the OWP and BWP and is in the process of testing in the IWP with the BPT. A close-up view of the skid trailer tire mark and the BPT tester is shown in Figure 14.

The high and low BPT site locations were selected during the summers of 1975 and 1976. The skid trailer was used over the entire section of pavement each year. The BPT test sites were the same from year to year for comparisons to show whether the sites retained the high and low SN.

British Portable Tester. The BPT used in this work was obtained from Soiltest, Inc., as their model No. HT-120. The tester was made by Wessex Engineering and Metalcraft Co. Ltd., Frome, Somerset, England, with pendulum Serial No. 7398. The BPT calibration was checked at the TRRL in December 1973. The rubber sliders used in the field work (1 X 3 in. size) were delivered with the tester. The laboratory rubber sliders (1 X 1¼ in. size) were obtained from Troxler, Inc.

The field tests with the BPT were carried out in accordance with Standard ASTM Method E 303 (41). The BPT was also calibrated before field tests were initiated. The BPN value of a "standard" brick was

Figure 13. View of Typical BPT Test Site

Figure 14. View of Skid Trailer Tire Mark and BPT

measured before and after the pavement's BPN value was obtained, to ensure that the BPT was functioning properly. The standard bricks BPN value had been determined at 77 F in the laboratory and was considered to be a reference surface that changed very little in the field. All BPN values were adjusted as required, based on the brick BPN value before and after pavement tests at the site.

The temperature of the pavement and the air was recorded at each test site. Care was taken to get a layer of water at least 0.02 in. on the pavement by flushing it thoroughly with a squirt bottle before each strike of the BPT. The test site was struck with the BPT 3 in. rubber slider a minimum of five times or until a constant BPN value was obtained.

Nine different setups of the BPT were performed at each BPT test site. The surface of the pavement was spray painted (Figure 13) at each of the test points. These painted squares were used to locate the exact pavement site for subsequent testing in 1976 and 1977. The IWP point was "flagged" to ensure that the skid trailer operator skidded the identical pavement area during the repeat tests.

Texture Depth Measurements. Two methods were used to determine the average texture of the pavement: the texturemeter developed by Scrivner (27) and the regular sand patch test. The regular sand patch method was run at six locations for each site, halfway between each BPT test location, i.e., two tests for each wheel path and two between the wheel paths. The sand patch tests are shown in Figure 13.

The volume of sand for the sand patch method was measured by the 1.550 in.^{3} (25.4 cm^3) field cup shown in Figure 15. The sand was

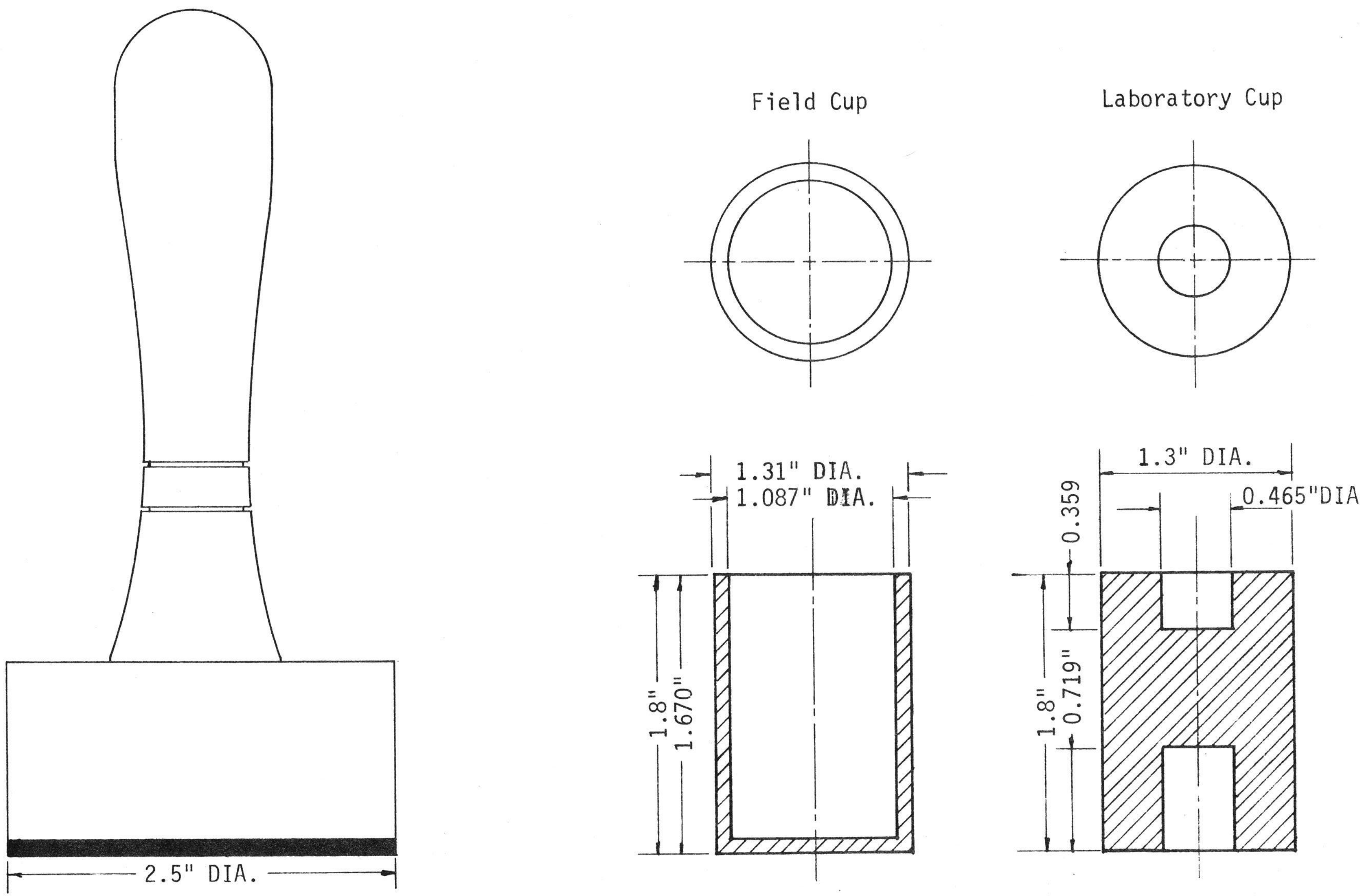


Figure 15. Scale Drawing of Regular Sand Patch Texture Depth Apparatus.

sized so that it would pass a No. 50 sieve and be retained on a No. 100 sieve. A straight knife-edge was used to strike-off the sand flush with the top of the cup, and a hard rubber disk attached to a wooden handle (Figure 15) was used to spread the sand over the pavement surface. Diameter measurements were read to the nearest quarter inch and average texture depth was computed in millimeters. The texture evaluations were performed only during the 1977 testing period.

The texturemeter then was used at each BPT test site. Twenty-seven readings were taken at the test section locations. For each of the nine BPT test sites, the texturemeter was used three times: once immediately before the site, once just next to the site, and once immediately past the site.

Laboratory Tests

Laboratory tests were conducted on asphalt cement, mineral aggregate, and asphalt mixtures. In general, the common laboratory test procedures and specimen preparation techniques used by the Arkansas Highway Department (109) were employed in this research.

The physical properties of the asphalt cement were determined by using standard ASTM methods. These values are shown in Table XI with the relevant ASTM method of test.

The physical properties of the aggregates are shown in Table XIII. All of these test values are based on results from duplicate test samples. The applicable standard ASTM methods were followed except for the insoluble residue (IR) test. Details of the IR test and the particle shape test are reported in another section of the report. Each aggregate sample also was evaluated for its film stripping characteristics by the static immersion stripping test (SIS), the dynamic immersion stripping test (DIS), and by the quantitative surface reaction test (SRT).

The polishing characteristics of each aggregate sample were evaluated by the British Wheel Test (by the Texas Highway Department) and the accelerated Polishing Device that was constructed for this investigation. The aggregates also were incorporated in asphalt mixtures and polished on the APD for their friction values. All friction measurements were made with the British Portable Tester previously described and their Arkansas Polish Value or their Polish Value was determined.

The relative stripping resistance of each aggregate sample was evaluated in asphalt mixtures by the immersion-compression (I-C) test. For this test the Marshall Apparatus and a vacuum saturation technique were used. Immersion-compression specimens were tested in a 50 blow compaction mix and also were remolded at 35 blow compactions and retested.

All asphalt mixtures were molded at optimum asphalt content as determined by the Marshall Mix Design Method. The theoretical maximum specific gravity of each sample mixture was measured. Approximately 550 asphalt specimens were molded, compacted, and tested for this investigation.

Aggregate Tests

Each sample of aggregate was sieved into four different sizes on a Gilson Testing Screen. These U.S. Sieve Sizes were: 3/4 to 1/2 in., 1/2 to 3/8 in., 3/8 in. to No. 4 sieve, and No. 4 to No. 10 sieve. Specific test samples then were obtained from these gradations by quartering.

The amount of hydrochloric acid insoluble residue for each aggregate sample was determined in accordance with the Oklahoma Highway Department Method OHD-L-25. This test procedure is reproduced in Appendix A.

The aggregate particle shape was determined by the British Standard 812 Method. The flakiness index of an aggregate particle is found by separating the flaky particles, i.e., particles that have a thickness (smallest dimension) of less than 0.6 their nominal size, and expressing their mass as a percentage of the mass of the sample tested. The elongation index of an aggregate sample is found by separating the elongate particles, i.e., particles that have a length (greatest dimension) of more than 1.8 their nominal size. The test procedure used was as detailed in the Asphalt Institute publication MS-13, Asphalt Surface Treatments (110).

Asphalt Mix Design

The asphalt mixture was designed to conform to the specifications for a AHD ACHMSC Type 2 mix. The specification mix limits are shown in Figure 16. The laboratory grading is shown in Table III. The mixture gradation was controlled by sieving the aggregates into eight different sizes on a Gilson sieve and then recombining to obtain the desired gradation. In all the mixtures the fine aggregate (minus No. 10 sieve size) was kept constant and only the coarse aggregate

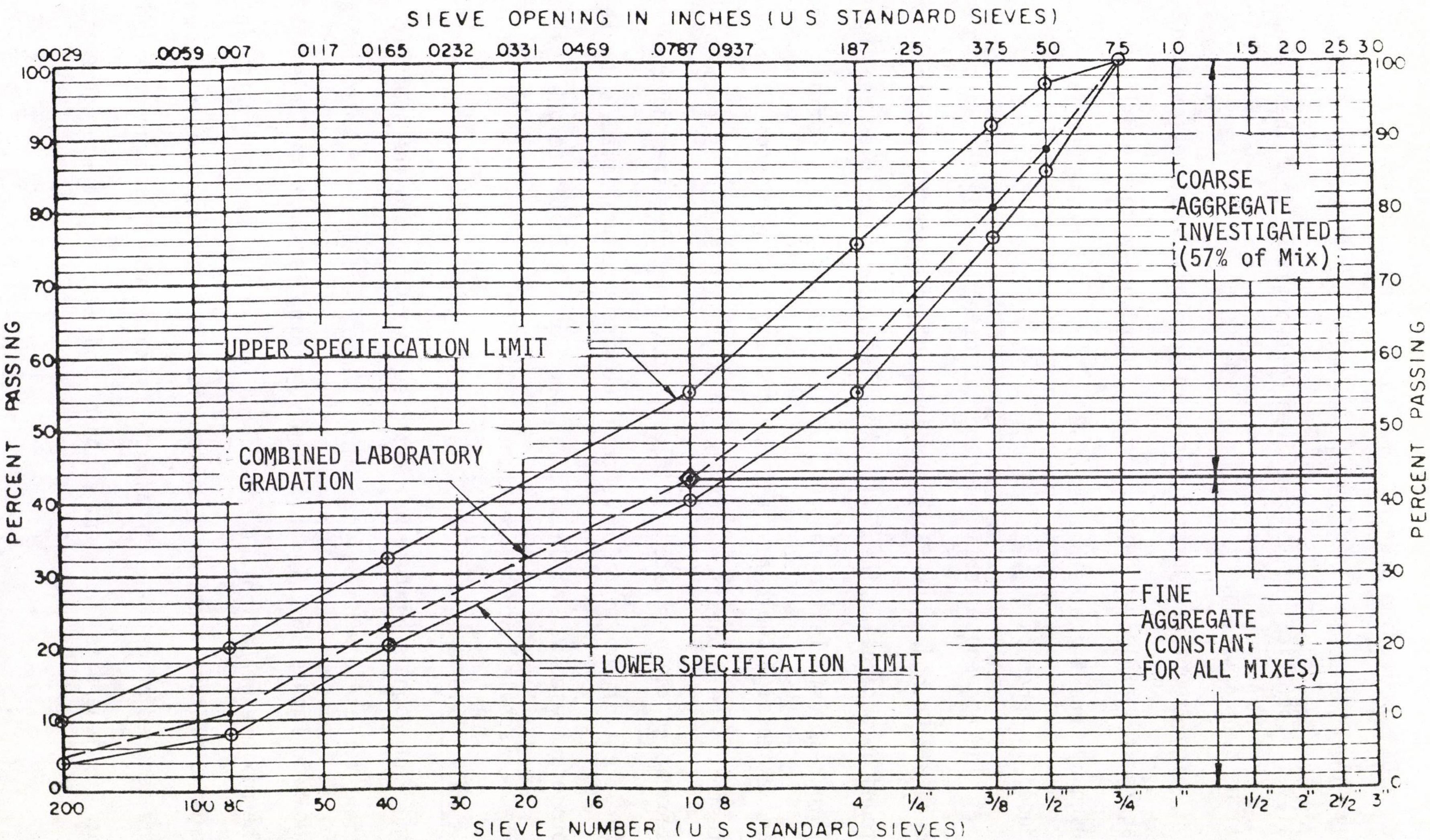


Figure 16. Arkansas Highway Department ACHMSC Type 2 Aggregate Grading Limits and Combined Laboratory Gradation

under study (size 3/4 in. to the No. 10 sieve) was changed. The fine aggregate was all West Fork limestone except for the sieve size No. 80 to 200 which was sand obtained from the Arkhola Hot-Mix Plant at Fort Gibson, Oklahoma. Each mixture was identified by the name of the coarse aggregate used in the mix. The mixture was coarser than the midpoint gradation because the preliminary trial mixtures gave air void contents below the specification limit when the midpoint gradation was used.

Molding and Testing Asphalt Samples. A decision was made to use an asphalt cement content by weight of the mix between 4 and 6 percent and to determine the optimum asphalt content by the Marshall Method. For testing, the blended aggregate and asphalt cement were heated to 275 F and mixed in a Hobart mixer until thoroughly coated, and finally placed in a holding oven at 255 F until molding. Four specimens were molded at each asphalt content.

The specimens were molded at 50 blows per side in accordance with Standard ASTM Method D 1559 (41) and allowed to cool. Next, their heights were measured to the nearest 0.001 in.; then their bulk specific gravity was determined , in conformance with the Tentative Standard ASTM Method D 2726 (41), by use of a Mettler P3N Balance. Each group of specimens was tested in a Marshall testing apparatus, after the specimens had been soaked in a 140 F water bath for 30-40 minutes, to determine their stability and flow.

Remolding Marshall Specimens. The bursted Marshall specimens were reheated to 255 F, loosened by hand with a trowel, and remolded at 35 blows per side for the immersion-compression test. This lower compactive effort was used to obtain greater air voids. Again, after remolding, the specimen heights, bulk specific gravity, and air voids were determined.

Voids Analysis. The theoretical maximum specific gravity (Gr) of each asphalt mixture was determined by testing specimens, of two different asphalt contents mixed, as described heretofore by the Standard ASTM Method D 2041 (41). From these data the effective aggregate specific gravity was calculated:

$$\text{Gage} = \frac{\text{Pag}}{\frac{100}{\text{Gr}} - \frac{\text{Pac}}{\text{Gac}}}$$

where:

Gage = effective specific gravity of aggregate

Gr = maximum specific gravity of asphalt mixture

Gac = specific gravity of asphalt

Pag = aggregate, percent by total weight of mixture

Pac = asphalt, percent by total weight of mixture

Once the effective specific gravity of the blended aggregate was determined, the maximum theoretical specific gravity (Gmt) for each specimen asphalt content was calculated.

Gmt values obtained by this method were also compared with theoretical maximum specific gravities calculated by using the aggregates, bulk specific gravity (Gba) and apparent specific gravity (Gag). This procedure was used to determine whether discrepancies occurred in any of the specific gravity tests. It is also noted that the asphalt absorbed by the aggregates and the effective asphalt content of the asphalt mixture can be estimated from the foregoing procedures.

For each specimen molded, the percent air voids (AV), voids in the mineral aggregate (VMA), and percent voids filled with asphalt (VF) were determined. The optimum asphalt cement content was selected in accordance with the procedure given by Wallace and Martin (111).

Accelerated Polish Asphalt Mixtures. The asphalt mix specimens prepared for the APD were all molded at 50 blows per side at their optimum asphalt contents. A total of 12 specimens were molded to represent each aggregate sample, 6 specimens were used for the polishing test, 3 specimens were tested "dry" for their Marshall stability and flow in the normal manner described previously, and 3 specimens were tested "wet" after vacuum saturation for their Marshall stability and flow. These 6 Marshall stability specimens were also remolded at 35 blows per side and again subjected to the I-C test. The asphalt mixtures prepared with two study aggregates were called "blends". They were batched, mixed, molded, and tested as described heretofore.

The blended asphalt mixtures, optimum asphalt content was estimated from the previously performed mix designs. However, only 7 specimens were prepared for each blend; 3 specimens were polished and 4 were tested for their Marshall stability and flow values. The blended asphalt mixtures' coarse aggregate consisted of equal parts, by weight, of each study aggregate; the fine aggregate fraction remained the same for all mixtures. However, when the England synthetic aggregate was blended, the proportioning of coarse aggregate was by volume, on the basis of their aggregate bulk specific gravities.

The combined study aggregate bulk specific gravity and water absorption, i.e., for the 3/4 in. to No. 10 material, is shown in Table XIII.

Accelerated Polishing Equipment

The two primary pieces of equipment used for the laboratory investigation were the British Portable Tester and an Accelerated Polishing Device. The BPT used in this work has been described.

The initial project proposal for this investigation indicated that an accelerated polishing device would be designed and built to evaluate the polishing characteristics of aggregates and asphalt mixtures. However, this plan of study was not approved by the Federal Highway Administration (FHWA) and the proposal was rewritten to accomplish the polishing tests by use of the British Wheel. The investigators planned to modify the asphalt mixture specimens by using a curved molding device to accommodate testing on the British Wheel. However, after 12 months of unsuccessful effort to procure a British Wheel, approval was obtained to design and construct the Accelerated Polishing Device as initially proposed.

British Wheel Tests. As a condition of the approval to construct the APD, the polishing values with the Arkansas APD were required to be correlated with the results from the British Wheel test. For this correlation work, duplicate aggregate samples from all 18 sources were prepared for testing. Coarse aggregate used for accelerated polish testing was 3/8 in. to the 1/4 in. size which had been carefully selected to be representative of each quarry.

The aggregate samples were quartered, half being retained for APD evaluations and the duplicate sample of each aggregate shipped to the Materials and Test Division, Texas Highway Department (THD), Austin, Texas. Personnel of the THD prepared and polished the 18 samples on a British Polishing Wheel in accordance with the Texas test method (112). The samples were tested before and after polishing with a BPT and the aggregates, "polish values" were recorded. Seven duplicate samples of each aggregate were prepared and tested. Details of the British Wheel test equipment are reported in Chapter II. Test values used in this work from the Texas British Wheel data are reported as Texas Polish Value (TPV).

When the polished aggregate samples were returned from the Texas Highway Department, they were tested with the Arkansas BPT in accordance with the Standard ASTM Method E 303 (41), and in general the final "polish values" obtained with the respective BPTs were the same. The Texas Polish Values are compared with the Arkansas Polish Values (APV) in another section of the report. The Texas polishing method utilizes curved 1.75 by 3 in. specimens and the Arkansas method requires flat, circular, 4 in. diameter specimens. Correlations between the two different methods of accelerated polishing and BPT test procedures are reported in Chapter V.

<u>Arkansas Accelerated Polishing Device</u>. The Accelerated Polishing Device is a small-scale circular test track modeled after the test device reported by Mullen (7). The apparatus was designed by the Principal Investigator and was constructed in the Engineering

Experiment Station Machine Shop for this investigation. The device was designed to induce an accelerated wearing of the pavement surface aggregates due to simulated traffic so that ultimate polish could be reached in a few hours. Figure 17 is a photograph of the APD device.

The machine itself consists of a circular track over which two two tires can rotate. The wheels on which these tires are mounted are at the opposite ends of a weighted axle, the revolution of which is controlled by the vertical shaft of an electric motor. There are 12 equally spaced 4 in. diameter specimen holders around the 28 in. diameter (centerline of specimen to centerline) track. The polishing tires were nylon 11 X 6.00 - 5, Go-Cart "super slicks", 2-ply rating, obtained from Sears Roebuck and Company.

Each tire was inflated to 20 psi gage, for a 15 psi average contact pressure with a wheel loading of 100 lbs. One wheel was adjusted forward at approximately 1 degree from the radial axle center to toe-out, and the other wheel was adjusted back from the radial center to toe-in, to produce an accelerated polishing action. The axle is driven by a 3/4 horsepower electrical motor by the vertical shaft from under the track. Attached to the motor is a "Zero-Max" variable speed drive by which the speed can be varied from 0 to 400 rpm. The operating speed used in the study was set at 30 rpm or 3600 wheel passes per hour, which totals to 86,400 wheel passes per 24 hours. This speed was chosen because lower speeds were found to induce more wear of the specimens than did higher speeds. The polishing table is equipped with a resettable automatic counter that records the total wheel revolutions.

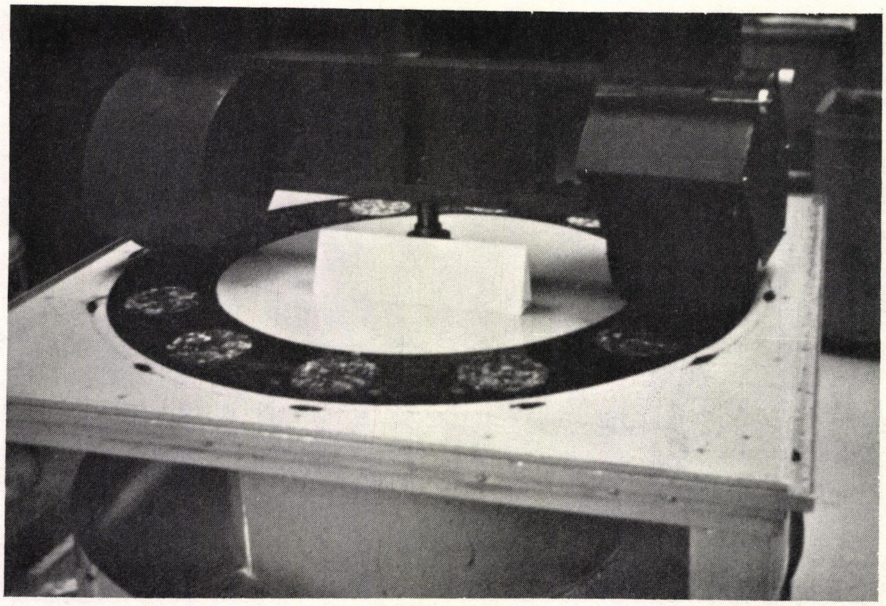


Figure 17. The Arkansas Accelerated Polishing Device

The specimen holders are steel cylinders attached to the track table. The holders have three leveling screws in the base to allow the specimen's height and leveling to be adjusted before and at intervals during testing. Before testing, the APD specimens are bonded to steel base plates so that they can be fastened securely and leveled or raised in height from beneath the table. The details of the apparatus are shown in Appendix B. The APD geometry and major parts are shown in Figure B1, the APD plan view is shown in Figure B2, and the APD specimen holder plan and elevation views are shown in Figure B3.

Arkansas Polishing Methods

The polishing tests performed in the laboratory for this investigation included both aggregate specimens (duplicate samples of the Texas British Wheel test specimens) and asphalt mixture specimens. The accelerated polishing test specimens were all 4 in. in diameter but of different height or thickness.

The asphalt mixture specimens were prepared as previously detailed. The aggregate sample specimens were cast in polyester resin in a mold, then bonded to a base plate that was secured to the specimen holder of the APD.

The British Portable Tester was used to measure the friction value of the 4 in. diameter specimen and this value is reported as the Arkansas Polish Value. A 1 by 1¼ in. rubber slider was used with a 3 in. strike path for all laboratory tests on the flat specimen. However, for the standard brick calibration test of the BPT, a 5 in. strike path was used.

The BPT back support leg was modified for use in testing the specimens in place on the APD. This configuration of the APD was used for all laboratory tests including the correlation work on the British Wheel specimens.

Aggregate Specimen Molding. For the laboratory investigation of the polishing characteristics of aggregate, a minimum of six specimens, composed of the 3/8 to 1/4 in. size aggregate, were made by casting the aggregate in polyester resin. The specimens, after proper curing, were cleaned and inspected, and the polyester bottom was ground flat. The flat samples were then ready to be tested.

A method of aggregate sample molding with a polyester compound was developed by Gunn (113). The specimens made by the Arkansas method were flat, circular, and 4 in. in diameter to adapt to the APD. The method developed for casting aggregates into polyester specimens is outlined in detail in Appendix C. A view of the APD molded aggregates on the storage shelf, pending the polish test, is shown in Figure 18.

British Portable Tester Methodology. The method of test with the BPT in the laboratory conformed in general to the Standard ASTM Method E 303. One of the first steps undertaken, prior to the use of the Arkansas BPT for laboratory investigation, was the calibration of the apparatus. The ASTM method of calibration was attempted; however, difficulties were encountered because of the lack of certain required pieces of equipment such as a pendulum holder. The pendulum holder restrains the pendulum from moving and facilitates the proper testing of the spring loading. Without it, the pendulum tends to

Figure 18. Arkansas Aggregates Molded for the Accelerated Polish Test

slip when loaded and proper calibration is hindered.

To accommodate for the lack of this piece of equipment, the spring loading was calibrated on a Mettler P3N Automatic Balance. The Mettler balance is a direct load read-out system and is accurate to the nearest 0.1 gram. The spring load of the Arkansas BPT measured with the Mettler was 2413 grams. This value was within the 2500 ± 100 grams limit set by ASTM.

All British Portable Testers, when used for laboratory testing, are equipped with a base plate designed to support the test specimen securely. This plate has a mount especially designed for holding curved British Wheel specimens. A modification of this base plate was made by attaching a circular holder which would support the Arkansas specimens securely when being tested for initial APVs. This modification of the base plate along with the BPT and Arkansas test specimens are shown in Figure 19. The apparatus was designed also to facilitate the testing of asphalt concrete specimens.

The BPT apparatus was modified by the use of an alternate back leg assembly. The back leg assembly that was made for the BPT was too long to permit its use for testing specimens on the Accelerated Polishing Device. Therefore, another leg which was shorter but functioned approximately the same as the original was made and used. This alternate back leg is shown attached to the BPT in Figure 20. Also illustrated in this photograph are the comparative size and shape of the curved British Wheel specimen on the left and the flat Arkansas aggregate specimen mounted in the base plate holder.

The BPT can be equipped with two types of rubber sliders. One

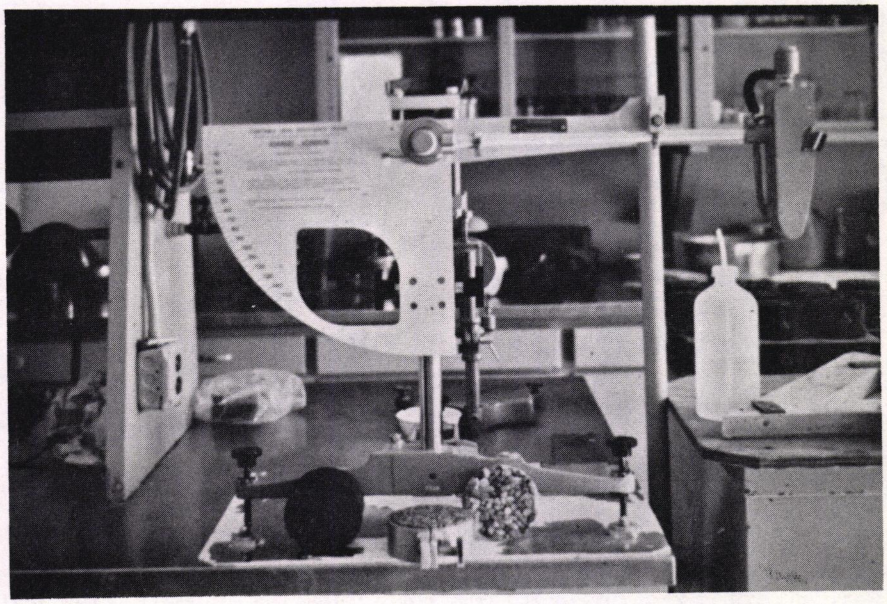


Figure 19. Arkansas BPT Base Plate Circular Specimen Holder and Arkansas Test Specimen

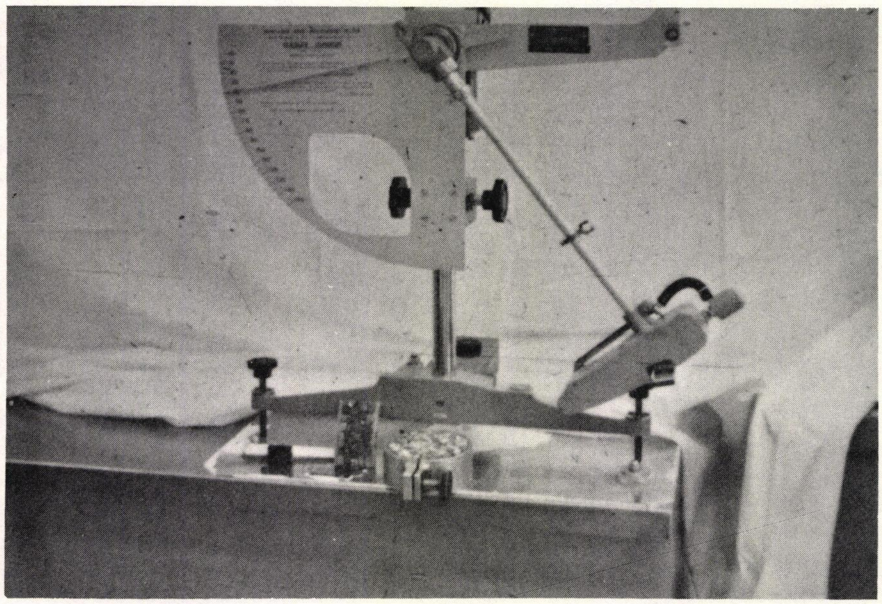


Figure 20. Arkansas Modified Back Leg Assembly on BPT with Arkansas Test Specimen Compared to Texas Curved Specimen

is a 3 by 1 in. slider for field testing and the other is a 1 by $1\frac{1}{4}$ in. slider for laboratory testing. To allow correlation of laboratory results with field results, the difference in slider sizes and strike paths, with respect to skid numbers had to be determined. The 3 in. wide and $1\frac{1}{4}$ in. wide sliders were used to test several different specimens. The tests indicated that the different sliders yielded the same results when the slider lengths were the same; therefore, the slider width had no effect on the test results as long as the surfaces tested were uniform across the slide path.

The different slider strike paths of 3 and 5 in. were also of concern. Several flat surfaces were tested at both the 3 and 5 in. strike paths. The resulting values were direct proportion to the slider strike paths because the spring loading was constant. Values obtained with the 3 in. strike path could be multiplied by 5/3, and the value obtained over a 5 in. strike path on the same surface could be reporduced. The reverse also proved to be true. Therefore, laboratory values obtained with a 3 in. strike path theoretically could be adjusted by a factor of 5/3 and the field values could be predicted. However, testing conditions must be identical in order to maintain a level of adequate accuracy.

The Standard ASTM Method E 303 is specifically designed for the laboratory testing of curved British Wheel specimens over a 3 in. strike path. The method of testing developed for Arkansas does not, however, involve curved specimens. The Arkansas specimens are flat, circular, and 4 in. in diameter. The only similarity is the length of strike path which is also 3 in. The ASTM test procedure thus was modified in this area to accommodate the testing of the Arkansas specimens. Test results on flat, circular, 4 in. diameter samples with the 1¼ in. rubber slider on a 3 in. strike path are reported as Arkansas Polish Values.

One foreseen problem with the use of the British Portable Tester was how to determine when the apparatus was yielding correct friction values. Even if the BPT was calibrated correctly, various factors such as temperature, slider wear, type of rubber, and improper adjustments could influence the results. A brick was chosen for convenience to be the controlling standard. The "standard brick" was struck several times over a 5 in. strike path and an average polish value of 40 was set as the standard value at 77 F. Any variation from this number would involve an adjustment to the BPT test values.

Before and after each set of specimens was tested, the BPT operation was checked by hitting the standard brick. The values obtained from the brick indicated whether or not the BPT was operating correctly. Because the laboratory temperature during testing was held relatively constant, the brick values reflected such influencing factors as rubber slider wear and aging. All the 1¼ in. rubber sliders supplied with the BPT apparatus were tested on the brick and different polish values were obtained. Some of the sliders, which had identical initial brick values, had different values after they had been used for various testing periods. The brick, therefore, proved that as a rubber slider wore down, the measured polish value was reduced, and thus incorrect values would be obtained.

Another factor of considerable interest was that when new, unworn rubber sliders were tested on the brick, their polish values were not the same. This finding indicated that some of the sliders had aged and become harder and thus yielded different values. A review of the literature indicated that the rubber properties should be monitored carefully to obtain correct results. All the sliders were tested with a durometer, in accordance with Standard ASTM Method D 2240-68. A durometer is a penetration type of device which measures resilience and hardness in various types of rubber. However, no significant differences between the rubber sliders could be found. The brick was the only indicator of "false" values which would be obtained if all the rubber sliders were assumed to be the same.

During the course of the investigation, the standard brick was observed to "polish" as a result of repeated check tests with the BPT sliders. By comparative testing of polished aggregate specimens of known polish value, the change in the standard brick value was determined and the laboratory and field test values were adjusted accordingly. The initial, or pre-polish, Arkansas Polish Value of each specimen was measured with the BPT on the bench test base plate. Once this value was obtained, a reference mark was placed on the specimen so that it would be properly oriented for future tests when placed on the Accelerated Polishing Device.

To allow the BPT to be placed on the APD to test the specimens in place, a portable aluminum platform was designed and built. This apparatus, shown in Figure 21, easily attaches to the axle of the polishing wheel and is supported on ball rollers so that it can rotate into position for the testing of each individual specimen. Once into position for testing, the platform is pinned into place (Appendix B, Figure B2) to ensure that it remains stationary during testing and also that the BPT is always in the same location in relation to the specimen. The BPT is then mounted on the plate, as shown in Figure 22, and the specimens are tested in place on the Accelerated Polishing Device. The initial APV was determined for each specimen after it was mounted into the Accelerated Polishing Device.

Core Description and Preparation. Cores were secured from the 32 BPT pavement test sites at locations shown in Table XV. Only jobs 9-617, 9-545, and 4-681 were not cored. Before drilling of the core, a wax pencil was used to mark the direction of traffic on the pavement surface in the area that would be included on the core surface. The cores were approximately 4 in. in diameter and were drilled with round, diamond-studded saw blades attached to a vertical-shaft, water-cooled coring machine. The core samples were taken by personnel of the Materials and Test Division of the Arkansas Highway Department in February and March 1977.

Six specimens were removed from the area of the BPT test points: two from the outer wheel path, two from the inner wheel path, and two from between the wheel paths. Each core was labeled as to site location, core number, and date removed, and immediately sealed in a plastic bag for protection. After the cores were received in the

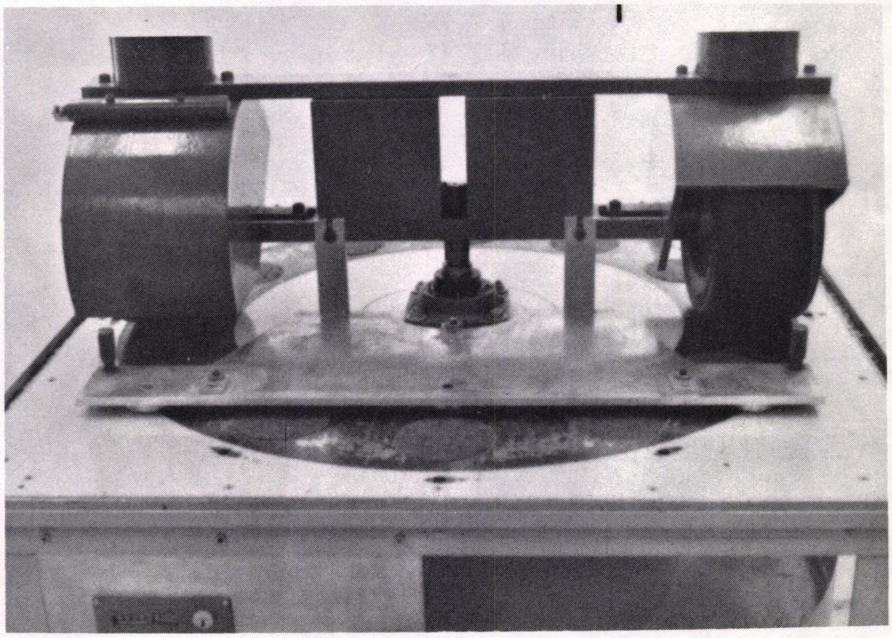


Figure 21. Table Platform for British Portable Tester

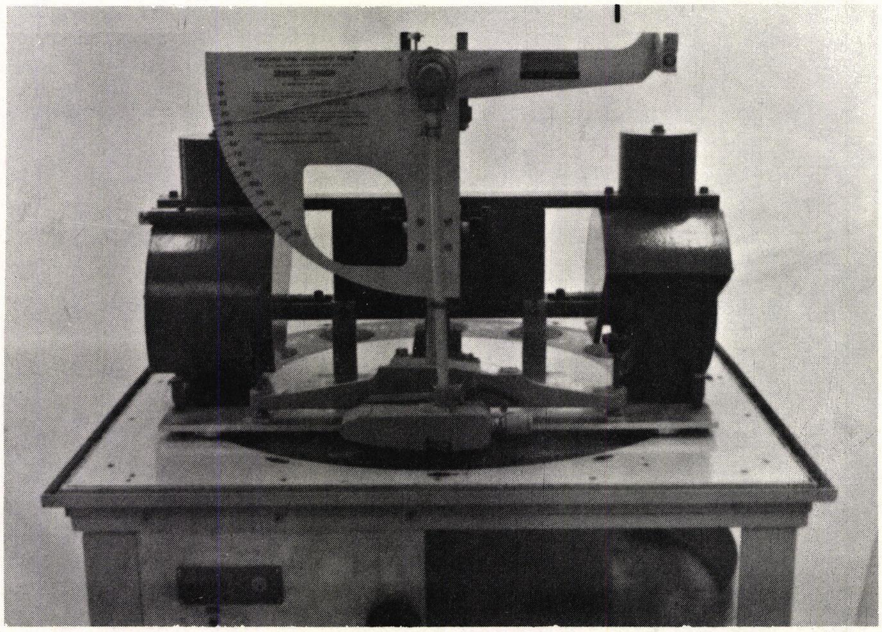


Figure 22. Position of BPT when Testing Samples on the Arkansas Accelerated Polishing Table

laboratory, it was noticed that the plastic bags had prevented the cores from drying after the wetting during sawing. Therefore, the cores were removed from the bags and placed with the surface side down on a flat table top for storage. This method of storage was believed to protect the surface of the sample and allow it to maintain its individual properties.

After each core's initial approximate height was recorded, a Target Masonry Saw with a 14 in. diamond-studded blade was used to remove any extra length of core material. Just enough material was removed to furnish a "squared-up" surface on the bottom of the specimen parallel with the pavement surface. The desired height of the core after trimming was $2\frac{1}{2}$ in.

The bulk density of each core was measured, and the average thickness of the surface course layer was estimated. The AHD Marshall Mix Design information was used to estimate the Gmt of the mixture; the actual asphalt cement and aggregate gradation from the final daily report of inspection at the asphalt plant were used in this calculation. A voids analysis calculation was made on each core to determine the AV, VMA, and VF. Of course, this was just an estimation of these values as the bottom layer of some of the cores was a "binder" type of mix, and some were old surface treatments. However, these voids analyses may be of interest in comparing other test data from the IWP, BWP, and OWP test sites, i.e., texture depth measurements.

The diameter of most cores slightly exceeded the 4.05 in. allowable dimension for mounting in the Accelerated Polishing Device, so that the sides of the specimens were sanded with a Rockwell Disk Sander and 40 to 60 grit, aluminum-oxide, 12 in. diameter sandpaper. Again, care was taken not to affect the surface properties of the cores in any way.

The initial texture depth of each core surface was found by the regular sand patch method but with a much smaller volume of sand than is used in field testing. As Figure 15 illustrates, the laboratory cups used to measure the volume of sand were of two sizes. The majority of textures allowed the larger 0.787 in.^{3} (2.00 cm^3) cup end to be used, but in some instances the diameter of the sand patch exceeded the 4 in. core and the 0.394 in.^{3} (1.00 cm^3) cup was employed.

The outer dimensions of the laboratory cup are approximately equal to the outer dimensions of the field cup, as this size is convenient in handling. The inside dimensions of the larger (2.00 cm^3) laboratory cup are in the same length-to-diameter ratio as those of the field cup, but the length dimension in the smaller (1.00 cm^3) laboratory cup is simply half of the larger laboratory cup's length. A straight knife-edge was used in striking off the excess sand, and the identical spreader was used as in the field tests. Patch diameters were measured in tenths of inches and the average texture depths were recorded in millimeters.

Preparation of Specimens for Polishing. The aggregate specimens to be polished were bonded onto a steel base plate with contact cement. The detailed method of bonding and removing specimens from the base is reported by Gunn (113). The prepolish APV was then determined.

The asphalt mixture specimens were also tested on the bench test plate for their initial APV. A reference mark was placed on each specimen to ensure proper orientation when mounted on the APD. The procedure for the bonding of these specimens was similar to that detailed in Appendix D for aggregate specimens except that "2-Ton" epoxy super glue was used rather than contact cement. A curing time of 12 hours was used to ensure a good bond between the asphalt specimen and the base plate.

The asphalt pavement core samples used for the investigation consisted of two specimens from each wheel path and two from between the wheel paths. Visual inspection was used to select the test specimens for accelerated polishing that maintained the best general condition both of the surface and the overall specimen. One specimen was selected from each wheel path and one from between the wheel paths for laboratory polishing. The remaining three cores were held in reserve for possible extraction and gradation analysis.

All six core specimens, from each study pavement, were tested with the BPT on the bench test plate to determine the initial APV. Each specimen was positioned in such a manner that the BPT measurement was made in the direction of traffic, as in the pavement. The cores then were mounted on steel base plates by applying epoxy glue to the core bottom and the plate and joining as previously described for asphalt mixtures.

Polishing and Testing Specimens. Twelve specimens were tested during each polishing test cycle. The specimen weight and height (with base plate) were measured before polish, and after polish the final weight and height were recorded to permit calculation of wearing rates. The specimens were secured on the table, with the plane of the reference mark at 90 degrees to the plane of the wheel path, and adjusted to a height of 1/16 in. above the plane of the track surface to ensure adequate exposure of each specimen to the polishing action. The height was checked during polishing and readjusted if necessary to ensure adequate exposure.

The operational cycle time with the Accelerated Polish Device to achieve "ultimate" aggregate polish (a minimum of friction resistance) was estimated to be 20 to 30 hours. A review of the literature (13, 49) indicated that the initial polishing rate of aggregate specimens is fairly rapid, then decreases and approaches a constant level with continued polish time. The polishing curve for a circular track, shown in Figure 7, resembles a parabolic or exponential curve with the BPN values becoming asymptotic with increasing polish time. Most of the polishing action on asphalt mixtures (Figure 7) was achieved within 3 to 6 hours by Mullen (7).

Trial operations of the APD indicated that a test speed of 30 rpm of the wheels gave a faster polish rate than a speed of 60 rpm. These preliminary test cycles were for 25 hours total polish time, the APV being determined at 1-hour intervals up to 10 hours, then at 12, 15, 20, and 25 hours. To determine the initial polishing trends of the specimens, the first three hours of polishing and testing were divided into 30 minute periods. This initial work indicated that most of the polish had occurred by the end of 20 hours of operation, and this maximum time was used for all routine polish testing in the investigation.

The operation of the BPT in the laboratory followed the same technique as previously described for field pavement testing. The BPT strike path was adjusted to 3 in. of slide distance with the 1 by 1¼ in. rubber slider. The specimen was then wetted (and re-wetted between strikes) and struck with the rubber slider a minimum of five times or until a constant reading on the BPT scale was obtained. This value was recorded as the Arkansas Polish Value.

The Accelerated Polishing Device was operated at 30 rpm and the specimen's polish value was measured at intervals of 30 minutes, 1, 1.5, 2, 2.5, 3, 4, 5, 10, 15, and 20 hours. At the end of the 20-hour polish cycle, the polished specimens were removed from the table and a new set of specimens were prepared for testing. The polish and test cycles were then repeated.

A polishing cycle thus consists of: preparation of specimens; determination of pre-polish APV on the bench test stand; mounting specimens on the Accelerated Polishing Device and determining initial APV in place; polishing and testing the specimens at various time intervals until they are considered to be at their minimum polish value; removal of the samples, measurement of final heights and weights, and separation of the samples from the base plates.

In testing and polishing pavement core specimens, a time equals zero polish value was taken on each specimen after 50 revolutions of the polishing wheels to test the specimens after dust and road oil had been removed. The regular sand patch method then was used to evaluate the final texture depth of the specimen surface.

Film Stripping Tests

The static immersion stripping test procedure conformed to the ASTM standard procedure, with a water immersion temperature of 77 F. A modified SIS test also was used in which the immersion water temperature was increased to 140 F. The same sample preparation and coating technique was employed in both the static and the dynamic immersion stripping tests. The DIS test procedure and evaluation method were designed to obtain a relative stripping factor for each aggregate tested.

The surface reaction test was used to give a quantitative measure of stripping. The stripped specimen to be evaluated was obtained from the SIS and the DIS tests. A duplicate, uncoated specimen was used to estimate the original total surface area of the aggregate.

Sample Preparation. The material from each of the respective sources was sieved to obtain approximately 2000 g of aggregate passing the 3/8 in. sieve and retained on the 1/4 in. sieve. The aggregate then was washed, oven-dried, and quartered to obtain representative samples of approximately 100 g each. The dry aggregate was weighed (100.0 ± 0.2 g) and placed in sealed plastic bags for storage until required in the testing work. Ten samples of each of the various aggregates were prepared in this manner. Six of these samples, i.e., duplicate samples, were used in performing the static immersion stripping test, the dynamic immersion stripping test, and the surface reaction test. The remaining samples were used for specific gravity and absorption tests and for checks on results of the other stripping tests.

Samples for the static immersion and dynamic immersion tests were coated in the following manner. The aggregate and asphalt cement were heated to 275 F prior to the coating operation. To each of the 100 g samples of aggregate, 6 g of asphalt was added. The mixture was stirred and manipulated with a spatula until each rock was coated with asphalt. A hot plate was used to heat that mixture, as required to achieve 100 percent coating. About 3 minutes of hand mixing time was ordinarily required. The individual particles of asphalt-coated rock were placed in a pan of cold distilled water after mixing. Cold water was necessary to prevent the coated rocks from sticking together.

Static Immersion Stripping Test. The sample preparation and coating procedure descirbed follows the standard method of test for coating and stirpping of bitumen-aggregate mixtures, Standard ASTM Method D 1664 (41). After cooling in the chilled water, the coated sample was placed in a glass jar and covered with 600 ml of distilled water. The jar was capped and placed, partially submerged, in a 77 C water bath, and left undisturbed for 18 hours.

The amount of stripping then was visually estimated by using the ASTM standard procedure. To facilitate this evaluation, a comparison chart was prepared by tracing the outline of typical aggregate particles inside a circle the same diameter as the glass jar in which the samples were immersed. A series of these tracings were made and the cross-sectional areas of the aggregate particles in each were darkened to represent different amounts of coated surface. The SIS jars with specimens and the visual comparison charts are shown in Figure 23.

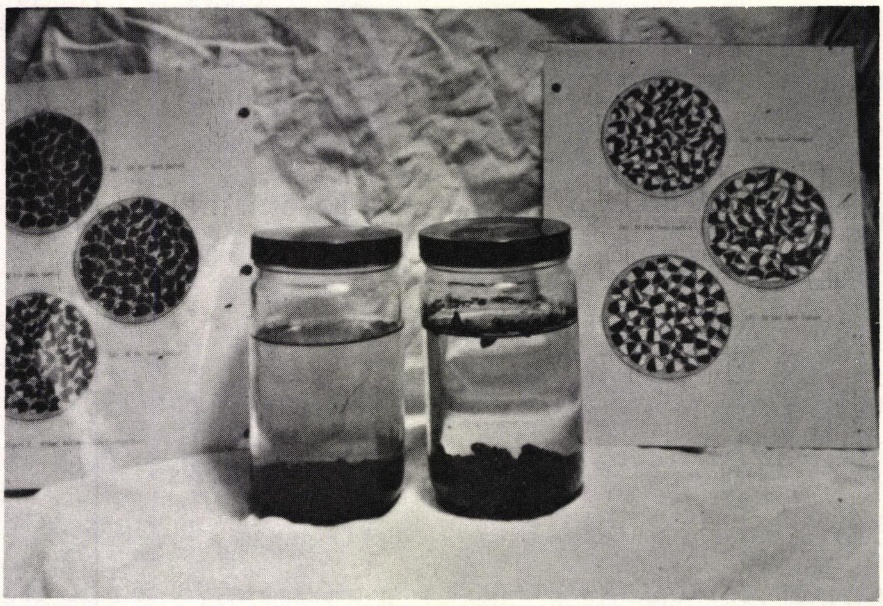


Figure 23. Static Immersion Stripping Test Jars
and Visual Comparison Charts

No stripping of any of the various aggregates was observed when they were coated with asphalt cement and subjected to the static immersion stripping test at 77 F. With a longer period of immersion or higher immersion temperatures, some stripping of the aggregates was anticipated to occur. Therefore, the SIS (77 F) samples were placed in a 140 F water bath and left undisturbed for 18 hours. The amount of stripping (which was considerable) then was visually estimated by using the comparison chart as before. This test method was designated the static immersion stripping test at 140 F.

Dynamic Immersion Stripping Test. To accelerate the stripping action of water on coated aggregate, a dynamic stripping device was purchased. The literature review showed many investigators had used a dynamic immersion stripping test to evaluate the effects of water on asphalt coated aggregate. The method originally used by Nicholson (59) was followed in this study. A DIS apparatus was purchased form Soiltest, Inc. which holds four glass jars of approximately 8 ounce capacity. The device was rotated about a horizontal axis at about 40 rpm. This action caused the coated aggregate sample to fall from one end of the jar through the water to the other end during each revolution. The DIS apparatus is shown in Figure 24.

Preliminary tests with the DIS device revealed that the non-stripping aggregate (Valley Springs limestone) would paritally strip when the sample was tumbled continuously for 4 hours. The siliceous aggregate particles usually retained more than 50 percent of their coating at the end of 4 hours of tumbling. Therefore, a 4-hour DIS test period was chosen, with the temperature maintained at about 77 F,

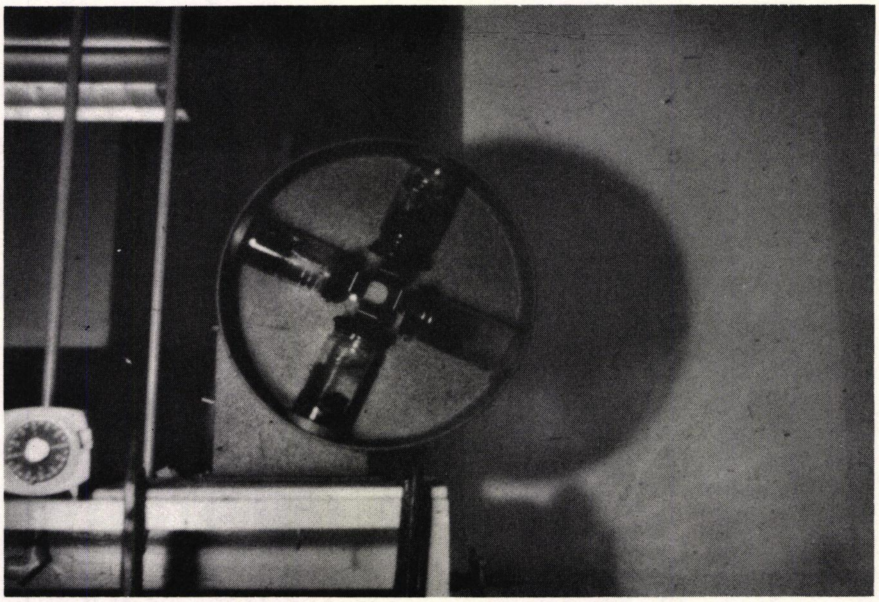


Figure 24. Dynamic Immersion Stripping Apparatus

which was the normal laboratory temperature.

The dynamic immersion stripping test procedure involved coating the aggregate with asphalt cement as was done for the static immersion test, then subjecting the coated aggregate particles to 4 hours of water agitation. A visual estimate of the amount of stripping was made at the end of 4 hours. The evaluation method was similar to that used in the static immersion test, i.e., the comparison chart.

The intended purpose of the DIS test was to induce stripping by subjecting each aggregate sample to the same amount of agitation in a water medium, and then to compare the visually estimated retained coating with a measured amount as determined by the surface reaction test.

Surface Reaction Test. This method of testing requires the measurement of the gas pressure generated during a chemical reaction. Because the temperature of the reaction affects the volume of the gas, it was necessary to measure and record simultaneously the pressure and temperature generated during the reaction. The apparatus developed to contain the reaction and measure the reaction products was essentially a modified 4 quart stainless steel pressure cooker, equipped with suitable instrumentation to monitor and record the desired quantities evolved.

A Sargent dual-arm recorder was used to record both temperature and pressure in the pressure container. A Stratham Model PA 707 TC-5-350 pressure transducer with a range of 0 to 5 psig and a thermistor (linked to a YSI Model 47 Scanning Tele-Thermometer) were mounted on the lid of the container and connected to the recorder. With this equipment, pressure in the container could be determined to the nearest 0.025 psig and temperature to the nearest 1.0 F. An overall view of the SRT equipment is shown in Figure 25.

Two types of acid solutions were used in this investigation. For limestone aggregate, which is predominantly calcium carbonate ($CaCO_3$), about 1.0 normal hydrochloric acid (HCl) was used. A 100 g sample of aggregate, when reacted with 200 ml of 1.0 normal HCl acid solution, would create between 1 and 5 psi of gas pressure in 3 minutes. Carbon dioxide (CO_2) is the gas generated in this reaction. For aggregate composed mainly of silicon dioxide (SiO_2), the reagent required to obtain a measurable gas pressure was concentrated hydroflouric acid (HF). The reaction creates noxious silicatetraflouride gas (SiF_4). This acid and gas are highly toxic to humans and must be handled very carefully. All work with hydroflouric acid was carried out in a well-ventilated fume hood, with appropriate safety equipment. Although the SiF_4 pressure was small, it was of sufficient magnitude to be measured. All work with the acid solutions containing HF was carried out in polyethylene or polystyrene containers.

The detailed test procedure for the surface reaction test and method of calculating the percentage retained coating of asphalt are given in Appendix D. This test method and apparatus were patented as United States Patent No. 3,915,636 of October 28, 1975, by the Principal Investigator, and the patent rights were assigned to Oklahoma State University.

Immersion-Compression Test

The immersion-compression test procedure adopted was patterned

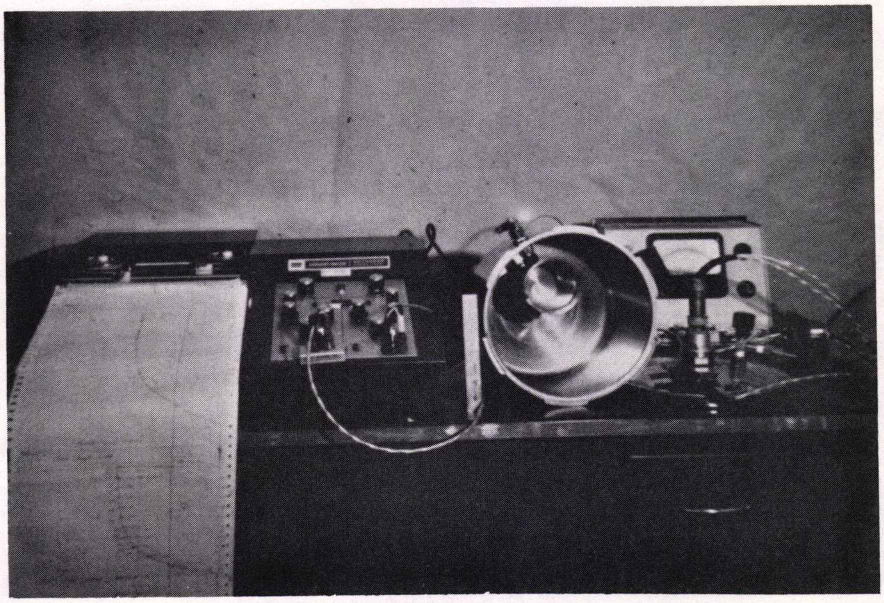


Figure 25. Surface Reaction Test Equipment

after the Standard ASTM Method D 1075 (41). The specific test method followed was the proposed method of test for "Effect of Water on Resistance to Plastic Flow of Bituminous Mixtures using the Marshall Apparatus" reported in the 1973 Annual Book of ASTM Standards (41). This procedure was modified, by vacuum saturation of wet specimens, to take advantage of available molding and testing equipment.

As mentioned in the literature review, the I-C test was developed to be used as an indicator of the stripping effects of water on asphalt mixtures. The problems experienced by other investigators were related to preparing duplicate specimens and then saturating them with water so that there was an opportunity for the water to act inside the compacted asphalt mixture.

As this investigation is concerned with durability of asphalt pavements, it was decided that the "wet" specimens would be vacuum saturated to ensure that water had an opportunity to cause stripping. The "wet" specimen stability was then compared with the "dry" specimen stability to obtain the percentage retained strength.

Preparation of Specimens. The specimens used in this test were prepared as described in the asphalt mix design method. Ordinarily a total of 12 specimens were molded at optimum asphalt content for each aggregate sample. A voids analysis was performed for each specimen. Six of these Marshall specimens were used for the accelerated polish test and the reamining six were subjected to the I-C test. After the compacted bulk density of each specimen was determined, they were divided into two groups, one for APV tests and the other for the I-C test, by a random selection process.

The I-C specimens were then split into a "wet" subgroup and a "dry" subgroup so that the average bulk specific gravity of each subgroup was approximately equal. Because of the limitation of using civil engineering student labor, no time limit control was possible to ensure that all asphalt mixtures had the same time span of testing; i.e., some I-C mixtures were tested within one week from initial mixing and molding, whereas other mixtures required up to three or four weeks for the test to be completed.

Curing and Vacuum Saturation. The wet specimens were vacuum saturated prior to being soaked for 24 hours in a 140 F water bath. The dry specimens remained at room temperature during this time. A vacuum desiccator was used to contain the specimen and the vacuum was obtained with a water aspirator. The apparatus for this test is shown in Figure 26.

The wet specimens were placed in a vacuum desiccator and the air evacuated for 10 minutes under a vacuum of approximately 25 of Hg. The specimens next were flooded with deaired, distilled water and the vacuum process was continued an additional 30 minutes. The vacuum then was released and the system opened to atmospheric pressure which forced water into the void spaces of the specimens.

The wet specimens were placed in a 140 F water bath for 24 hours. At the end of 23 hours in the water bath the specimens were removed, and their saturated surface dry weight was determined. The specimens were kept in an auxiliary 140 F bath, on a laboratory cart, for this operation. The specimens then were returned to the 140 F water bath for subsequent testing at the end of 24 hours of curing.

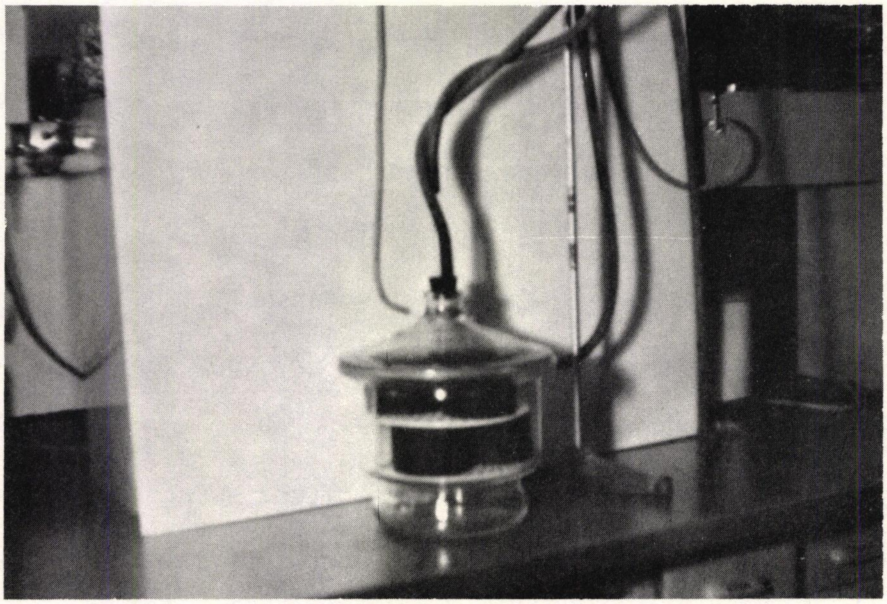


Figure 26. Vacuum Saturation Apparatus for the Immersion-Compression Test

Marshall Stability and Flow. The wet specimens were tested in the Marshall breaking head in accordance with the Standard ASTM Method D 1559 (41). Immediately after completion of the tests on the wet specimens, the dry specimens were placed in the 140 F water bath for testing. After a minimum of 30 minutes, at a bath temperature of 140 F, the dry specimens' Marshall stability and flow were determined.

Retained Strength. The percent retained strength of each group of specimens was determined by dividing the average strength of the "wet" specimens by the average strength of the "dry" specimens and multiplying by 100. The water absorption of each wet specimen also was calculated. In every I-C test, the Marshall flow values were higher for the wet specimen than for the dry specimen.

Economy of Blending Aggregates

The economy of blending two types of coarse aggregate to improve skid resistance of asphalt pavements depends on several factors. The factors to be evaluated include: gradation of the mixture, i.e., amount of coarse aggregate in mix (plus No. 10 sieve material); the cost of the "imported" aggregate; the cost of the "local" aggregate; the cost to haul the imported aggregate to the job site; the percentage of the imported aggregate required to provide the minimum pavement skid resistance for the project; and the benefits derived by reducing the number of accidents caused by slick pavements.

A benefit-cost analysis is needed to justify the expense of hauling a special aggregate for pavement construction. The obvious benefit from imporved pavement skid resistance is the reduction in accident cost to the highway user. Specific highway accident cost information was not available from the Arkansas Highway Department for use in this study. However, because pavement skid resistance is closely allied with accidents, the approach suggested herein is to calculate benefit-cost factors on the basis of the increase in skid resistance obtained by blending aggregates. This increased skid value is related to the effect of increasing the life of a pavement. The increased service life, then, is the benefit obtained from blending aggregates.

Based on the foregoing information, a procedure that can be used in the economic analysis for blending of aggregates to improve skid resistance was developed by Parker (114). The variables can be related by the following equation:

$$Cj\ Li = Aj\ Ca\ (Pa\ Pca + Pfa) + Aj\ (Ca + Cb)(Pb\ Pca) - Cj$$

where:

Cj = original total cost of the aggregate in the job for 100 percent polish susceptible aggregate

Li = increase in life of the unblended pavement to the life of the improved pavement, expressed as a decimal

Pa = percent of parent polish susceptible coarse aggregate expressed as a decimal

Pb = percent imported coarse aggregate expressed as a decimal

Pca = percent coarse aggregate in mix expressed as a decimal

Pfa = percent fine aggregate in mix expressed as a decimal

Aj = total weight of aggregate for the job in tons

Ca = unit cost of parent polish susceptible coarse aggregate in \$/ton

Cb = unit cost of the blending aggregate in $/ton

The various cost factors used in the economic caluclations should be obtained for each specific situation. The current unit prices used in Arkansas pavement construction are available from the AHD Office Engineer's "Weighted Average Unit Prices for the Calendar Year". The current cost of coarse aggregate similar to that to be used in a proposed pavement would be available from local contractors. Also, the cost to haul this material would be available from local contractors for a specific project. The percentage of polish resistant aggregate required to provide a minimum level of skid resistance can be obtained from the results of this investigation. The unknown for which the equation should be solved is the maximum justifiable cost (Ca). It is the amount that can be spent for purchasing and hauling the blended aggregate to the hot-mix plant of the parent polish susceptible aggregate to maintain the minimum level of skid resistant pavement.

Because the unit cost of aggregates fluctuates with supply and demand, no cost data are deemed appropriate for this report. Moreover, as no accident data are available on the specific test sections under investigation, the determination of benefits from improving skid resistance cannot be determined at this time.

CHAPTER V

TEST RESULTS AND DISCUSSION

The purpose of this study was to investigate the durability and skid resistance of Arkansas aggregates and asphalt pavement mixtures. The blending of high quality aggregate with polish prone aggregate to improve skid resistance was studied. The aggregate investigated consisted only of the coarse aggregate portion of an Arkansas Highway Department ACHMSC Type 2 mixture. The fine aggregate portion of the mixtures was a constant and consisted of West Fork screenings and Oklahoma fine sand. The asphalt used was a high quality AC-20 paving grade asphalt cement.

The field tests included friction measurements with the AHD skid trailer and the British Portable Tester, and pavement texture depth measurement by the sand patch method. The laboratory work included: aggregate polishing tests with the Texas British Wheel and Arkansas Accelerated Polishing Device and the British Portable Tester to evaluate friction; polishing tests of Marshall specimens molded with each study aggregate and blends of these aggregates; polishing tests of cores from the pavements investigated; and film stripping and immersion-compression tests on each aggregate and asphalt mixture.

The test procedures described in Chapter IV were followed, except where noted, in the testing of aggregates, asphalt mixtures, and pavements. The results of the test procedures employed are presented in tabular and graphic form.

The test data were analyzed by regression analysis with a standard computer program. Best fitted equations were determined, along with their coefficient of correlation (R), to indicate the relationship between variables.

Field Test Results

The field test results include pavement friction values from the AHD skid trailers, BPN values from a British Portable Tester, and pavement texture depths from the sand patch test. The field tests were performed in the month of June in 1975, 1976, and 1977. The results of the 1977 tests are shown in Table XVI. A total of 60 BPT sites were evaluated from the 34 pavement test sections. Pavement cores were taken from 32 sites along these pavements.

Skid Trailer Test Results

The 1975 and 1976 skid tests were performed with the AHD Arkansas No. 1 trailer. The 1975 tests were terminated after about one third of the pavements had been skidded because the towing truck broke down. Very few useful field data were obtained this first year.

The 1976 skid tests were completed on all roads, although numerous problems arose with the skid trailer water system and computer malfunctions were common. No continuing problems occurred with the use of the BPT in the field. The BPT tests were performed with a different operator each year.

The 1977 skid tests were performed with the new AHD skid trailer, Arkansas No. 2. The AHD skid truck operators were the same for the 1976 and 1977 tests. The Arkansas No. 2 skid trailer operated satisfactorily. Despite considerable difference in the SN_{40} values between the 1976 and and 1977 tests, there was a good correlation between the 1976 and 1977

Table XVI

1977 Field Test Results

BPT Test Site No.	Route-Section/ Aggregate	Skid Trailer		British Pendulum No. (BPN)	Texture Depth
		Avg SN_{40}	Site SN_{40}	IWP/BWP/OWP/Avg	(mm x 10^{-1}) IWP/BWP/OWP/Avg
1	16-3/Johnson ls	33	37	62/57/61/60	9.4/4.2/6.1/6.6
2	16-3/Johnson ls	41	43	70/62/67/66	7.6/3.7/5.5/5.6
3	71-16/Johnson ls	28	30	59/60/57/59	4.5/2.7/4.3/3.8
4	71-17/Johnson ls	32	28	57/57/53/56	4.2/3.4/4.0/3.8
5	45-5/Johnson ls	39	40	64/67/64/65	5.0/4.0/3.6/4.2
6	45-5/Johnson ls	44	43	62/68/65/65	4.4/3.3/4.9/4.2
7	62-2/Johnson ls	27	24	53/45/45/48	5.0/2.6/3.2/3.6
8	62-8/Johnson ls	27	32	54/51/52/52	4.5/2.9/3.9/3.8
9	65-1/Val. Spr. ls	29	28	56/49/56/53	3.2/3.6/3.4/3.4
10	65-1/Val. Spr. ls	29	31	59/56/59/58	4.0/4.5/3.7/4.1
11	62-7/Val. Spr. ls	34	34	59/57/59/58	4.7/3.6/4.0/4.1
12	62-7/Val. Spr. ls	34	38	58/55/57/57	4.3/4.8/5.4/4.8
13	62-7/Val. Spr. ls	39	41	63/58/61/61	4.4/3.0/2.8/3.4
14	65-4/Val. Spr. ls	27	30	51/50/51/51	3.2/3.9/2.9/3.3
15	65-4/Val. Spr. ls	26	28	54/52/50/52	3.9/6.9/3.6/4.8
16	5-15/Cabot ss	55	56	67/69/66/67	7.2/4.8/7.2/6.4
17	5-15/Cabot ss	53	54	65/69/66/67	7.7/5.5/7.5/6.9

Table XVI (continued)

1977 Field Test Results

BPT Test Site No.	Route-Section/ Aggregate	Skid Trailer		British Pendulum No. (BPN)	Texture Depth
		Avg SN_{40}	Site SN_{40}	IWP/BWP/OWP/Avg	(mm x 10^{-1}) IWP/BWP/OWP/Avg
18	5-14/Cabot ss	58	60	68/72/68/70	9.0/6.0/7.6/7.5
19	5-14/Cabot ss	49	47	63/68/65/66	9.3/5.7/7.7/7.6
20	5-12/Cabot ss	50	49	64/64/65/65	5.0/3.6/5.7/4.8
21	71-15/Van Buren ss	56	52	70/70/70/70	4.2/3.6/5.0/4.3
22	162-1/Van Buren ss	38	39	64/67/63/65	1.9/3.5/1.8/2.4
23	162-1/Van Buren ss	38	45	65/68/65/66	2.9/3.4/3.9/3.4
24	71-10/Jen. Lind ss	37	41	68/60/66/65	2.5/2.4/2.6/2.5
25	71-13/Jen. Lind ss	42	42	63/63/63/63	2.9/4.1/2.2/3.1
26	71-13/Jen. Lind ss	49	54	66/63/63/64	3.0/3.5/2.2/2.9
27	71-13/Jen. Lind ss	55	53	66/66/65/66	4.8/3.9/5.0/4.6
28	71-14/Jen. Lind ss	55	58	65/68/65/66	4.7/4.3/4.9/4.6
29	71-14/Jen. Lind ss	47	34	59/60/54/58	1.9/2.2/2.2/2.1
30	71-14/Jen. Lind ss	47	43	64/63/59/62	5.5/2.4/2.0/3.3
31	96-3/Jen. Lind ss	52	50	66/74/65/68	8.3/6.3/7.0/7.2
32	96-3/Jen. Lind ss	54	55	71/73/69/71	4.4/4.6/4.8/4.6
33	71-1/Tex. gvl	47	46	66/65/66/65	5.3/4.7/5.9/5.3
34	71-1/Tex. gvl	47	50	64/64/63/64	7.3/4.1/8.2/6.5

Table XVI (continued)

1977 Field Test Results

BPT Test Site No.	Route-Section/Aggregate	Skid Trailer		British Pendulum No. (BPN)	Texture Depth ($mm \times 10^{-1}$)
		Avg SN_{40}	Site SN_{40}	IWP/BWP/OWP/Avg	IWP/BWP/OWP/Avg
35	71-1/Tex. gvl	46	47	60/62/58/60	5.4/4.4/5.3/5.0
36	71-1/Tex. gvl	46	45	59/61/60/60	3.8/4.0/3.2/3.6
37	82-2/Tex. gvl	35	34	55/55/54/55	7.1/4.4/4.9/5.5
38	82-2/Tex. gvl	35	37	54/55/54/54	5.5/4.1/4.3/4.6
39	71-1/Mur. gvl	47	49	60/62/60/61	7.3/5.0/5.3/5.9
40	71-2/Mur. gvl	43	43	59/59/57/58	5.3/3.8/4.7/4.6
41	32-3/Mur. gvl	53	53	64/75/69/69	5.6/5.0/7.3/6.0
42	32-3/Mur. gvl	53	53	68/69/69/68	9.1/6.0/6.5/7.2
43	27-2/Mur. gvl	45	46	64/64/61/63	7.2/5.6/5.2/6.0
44	27-2/Mur. gvl	45	47	62/63/59/61	7.3/5.6/5.0/6.0
45	4-3/Mur. gvl	42	38	55/53/52/53	6.6/7.1/7.4/7.0
46	4-3/Mur. gvl	47	47	62/74/67/68	7.5/4.3/7.8/6.5
47	4-3/Mur. gvl	47	49	65/73/65/68	8.1/4.9/5.9/6.3
48	15-9/Big Rock ns	33	34	55/56/57/56	5.1/4.2/4.7/4.7
49	15-9/Big Rock ns	33	32	54/58/54/55	5.8/4.6/5.0/5.1
50	88-9/Big Rock ns	35	38	58/62/59/66	4.0/5.1/3.3/4.1
51	88-9/Big Rock ns	34	35	58/61/56/58	4.5/4.6/5.4/4.8

Table XVI (concluded)

1977 Field Test Results

BPT Test Site No.	Route-Section/Aggregate	Skid Trailer		British Pendulum No. (BPN)	Texture Depth
		Avg SN_{40}	Site SN_{40}	IWP/BWP/OWP/Avg	(mm x 10^{-1}) IWP/BWP/OWP/Avg
52	11-5/Big Rock ns	39	37	59/66/60/62	4.7/4.2/4.7/4.5
53	11-5/Big Rock ns	39	38	56/61/56/58	4.8/4.7/4.8/4.8
54	70-8/Big Rock ns	35	35	54/55/55/55	6.8/5.1/5.9/5.9
55	70-8/Big Rock ns	35	38	55/59/55/56	5.8/5.2/6.3/5.8
56	5-7/Malvern nov	43	47	59/63/58/60	8.5/7.4/9.7/8.5
57	70-9/Malvern nov	48	51	58/60/58/59	7.3/6.4/7.5/7.1
58	70-9/Malvern nov	48	47	56/58/59/58	7.4/6.8/7.1/7.1
59	70-10/Malvern nov	47	52	58/59/55/57	7.9/4.8/6.8/6.5
60	294-2/Eng. syn	35	43	71/72/64/65	10.1/13.4/9.8/11.1

skid trailer SN_{40} values.

The SN_{40} data from each year's skid test were plotted for each pavement test section. To illustrate this yearly SN_{40} variation, a graph of each type of aggregate pavement surface is presented.

Johnson Limestone. The SN_{40} test values plotted against road log mile for route 71-16 and 71-17 (Fayetteville Bypass) are shown in Figure 27. The break in the graph is where section 16 ends at the U.S. 62 overpass. The 1977 average SN_{40} for the entire job was 31; the average of the June 2, 1976, tests gave a SN_{40} of 47; reskid tests of June 23, 1976, gave an average SN_{40} of 63. An exceptionally heavy rain occurred on June 22, 1976, over the northwest corner of Arkansas which included all of the study pavements from the Johnson limestone quarry. The June 22, 1976, average skid values increased 33 percent over the June 2, 1976, skid values. This limestone pavement test section had an average daily traffic of between 7,000 and 11,000 vehicles, and the estimated total number of wheel passes was between 9 and 16 million.

Figure 28 shows the same test data plotted as reported above, except they are for route 45-5 between Goshen and the Madison County line. The 1977 job average SN_{40} was 42; the June 2, 1976, average SN_{40} was 60, and the reskid tests of June 23, 1976, had an SN_{40} of 62. The 1976 reskid test values increased only 3 percent. The daily traffic on this relatively new overlay job was between 1,000 and 1,400 vehicles per day. The estimated total number of wheel passes over this job was between 200,000 and 300,000.

Jenny Lind Sandstone. The variations between the 1976 and 1977 skid trailer SN_{40} values on this type aggregate are shown in Figure 29. These test values are for route 71-13 north of Mansfield. The job

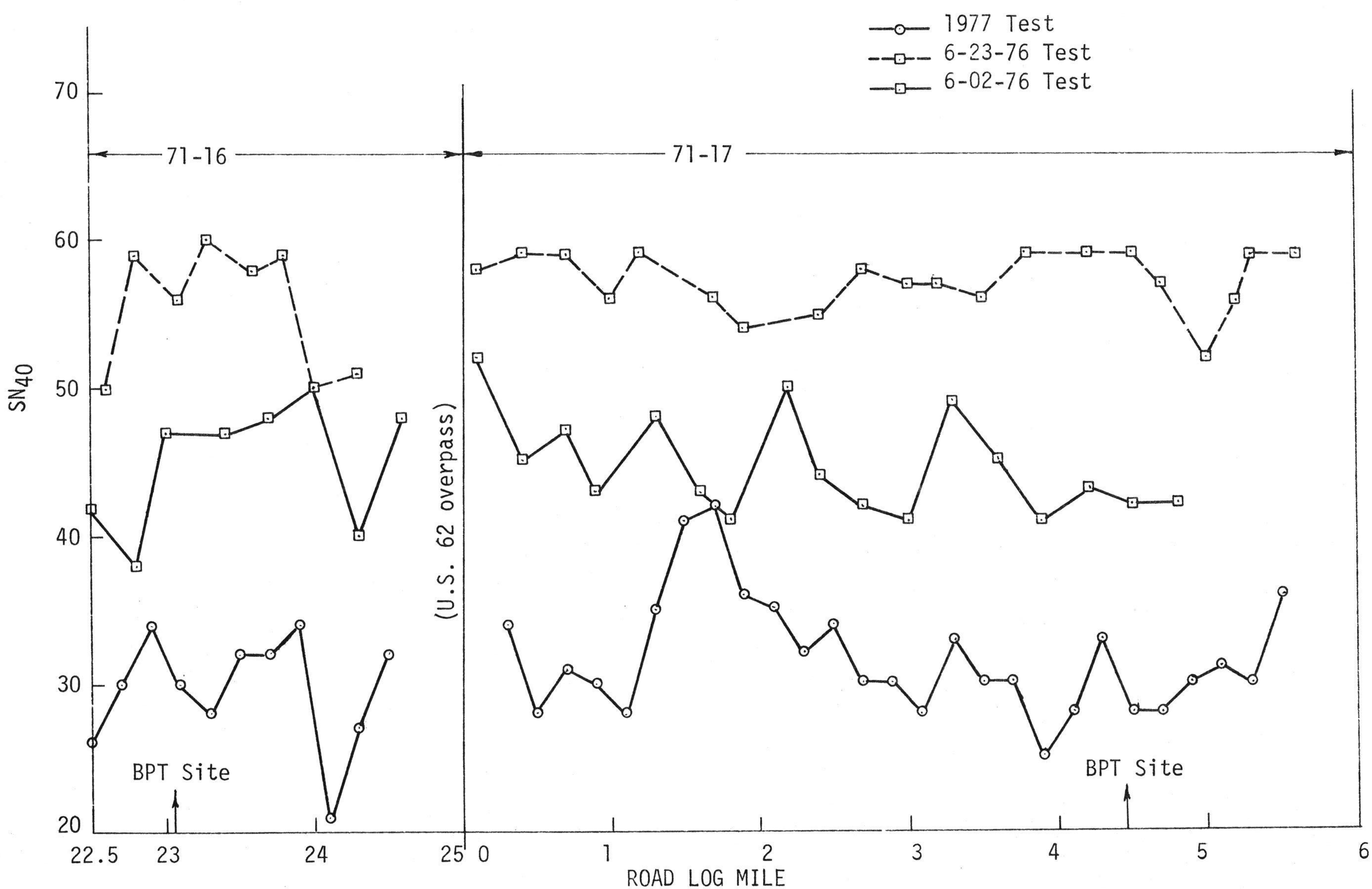


Figure 27. AHD Skid Trailer SN_{40} vs Road Log Mile - Johnson Limestone, 71-16 & 71-17, Fayetteville Bypass

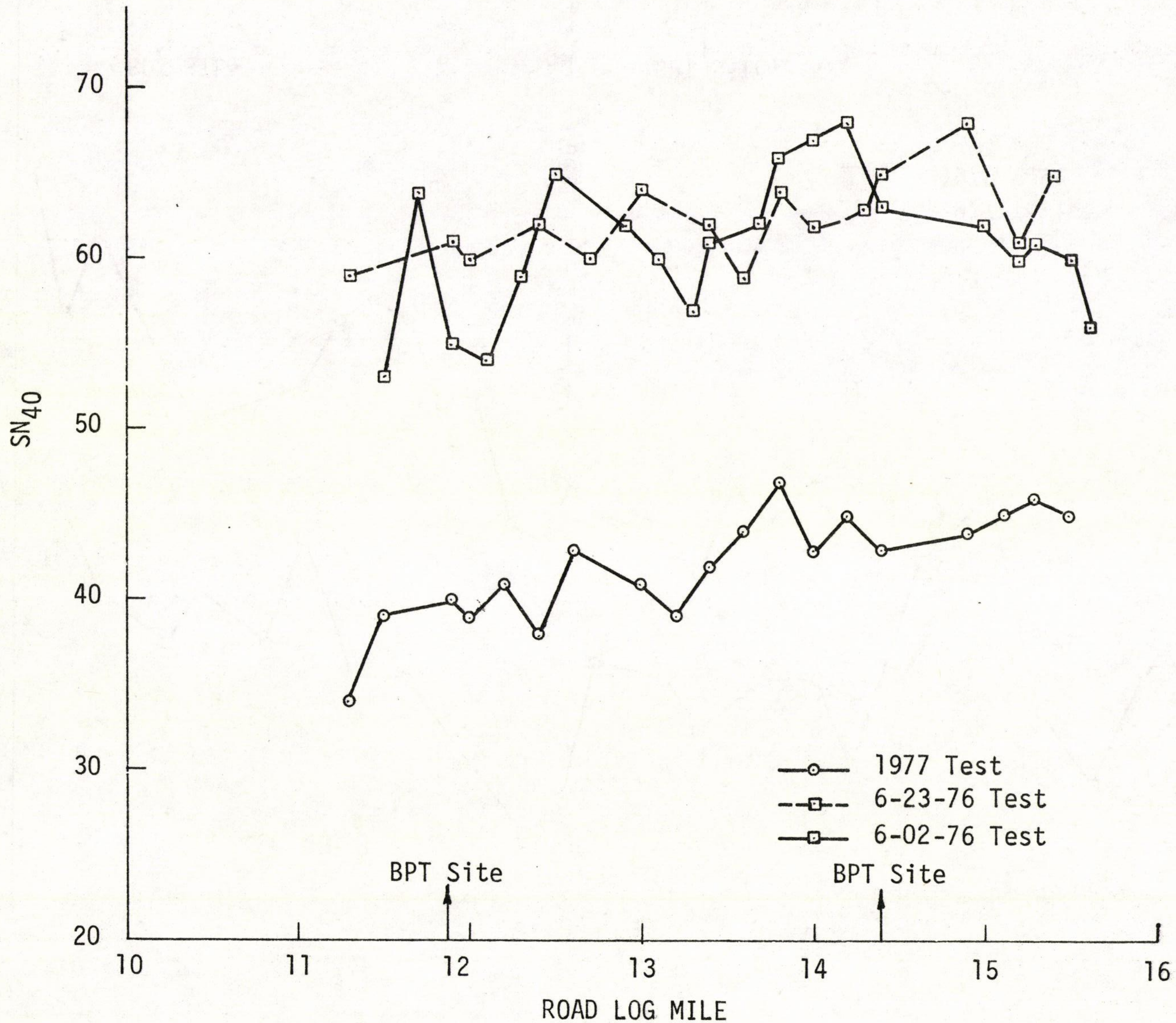


Figure 28. AHD Skid Trailer SN_{40} vs Road Log Mile - Johnson Limestone, 45-5, Goshen East

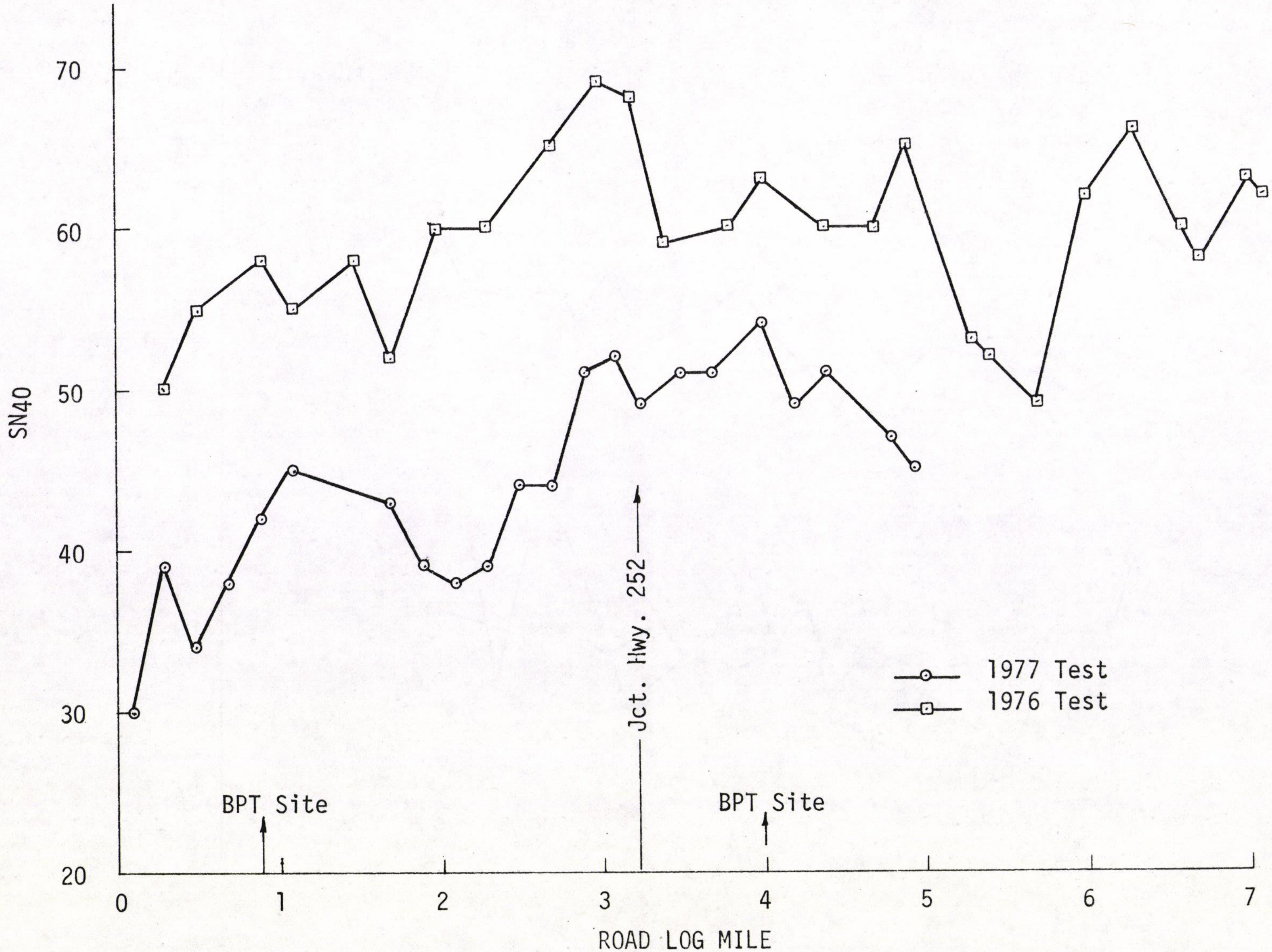


Figure 29. AHD Skid Trailer SN40 vs Road Log Mile - Jenny Lind Sandstone, 71-13, Mansfield North

average 1977 SN_{40} was 44, and the 1976 SN_{40} was 59.

Texarkana Gravel. The skid number variations for gravel mixtures are illustrated in Figure 30. This study pavement is route 71-1 from the Sulfur River north to the junction of highway 134. The job average 1977 SN_{40} was 46, and a 55 SN_{40} value was obtained in 1976. Routine inventory skid test was performed December 1, 1976, by the AHD skid trailer Arkansas No. 2. These SN_{40} values are also plotted in Figure 30 for comparison. No particular correlation of SN_{40} test values is noted, except the June 1977 test results are less than but parallel with the inventory skid test results for most of the road section.

Big Rock Syenite. The skid trailer SN_{40} test values typical of this aggregate are shown in Figure 31. This study pavement was route 88-9 from Altheimer to Cornerstone. The graph contains the results of skid tests for 1975, 1976, and 1977 from this study and also the routine AHD inventory test data of March 17, 1977. The 1977 SN_{40} job average value was 33, and an average of 41 was obtained in 1976.

Malvern Novaculite. The field investigation of this material included only two pavement jobs. The variation in skid trailer results for route 70-10, from the Garland County line to I-30, is shown in Figure 32. The 1977 skid trailer test job average value was 47, and the 1976 skid value average was 66. The 1975 test results along with the AHD inventory test data of October 26, 1976, are also included in Figure 32. The inventory tests were performed with the AHD Arkansas No. 2 rig.

England Synthetic. As discussed in Chapter III, only one pavement section of this material was selected for field testing. The pavement is a surface treatment job on route 294-2, and was skid tested in the

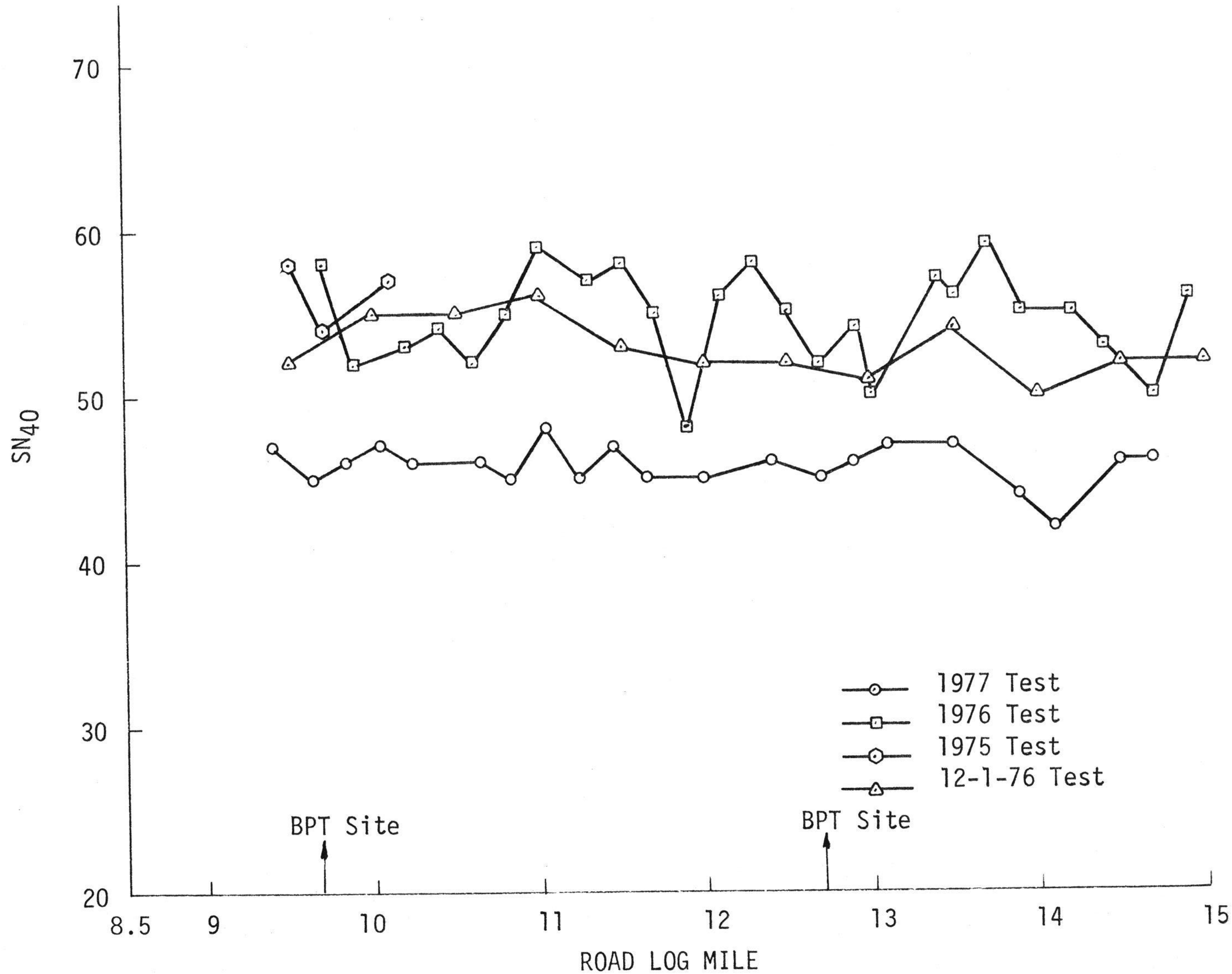


Figure 30. AHD Skid Trailer SN_{40} vs Road Log Mile - Texarkana Gravel, 71-1, Sulfur River North

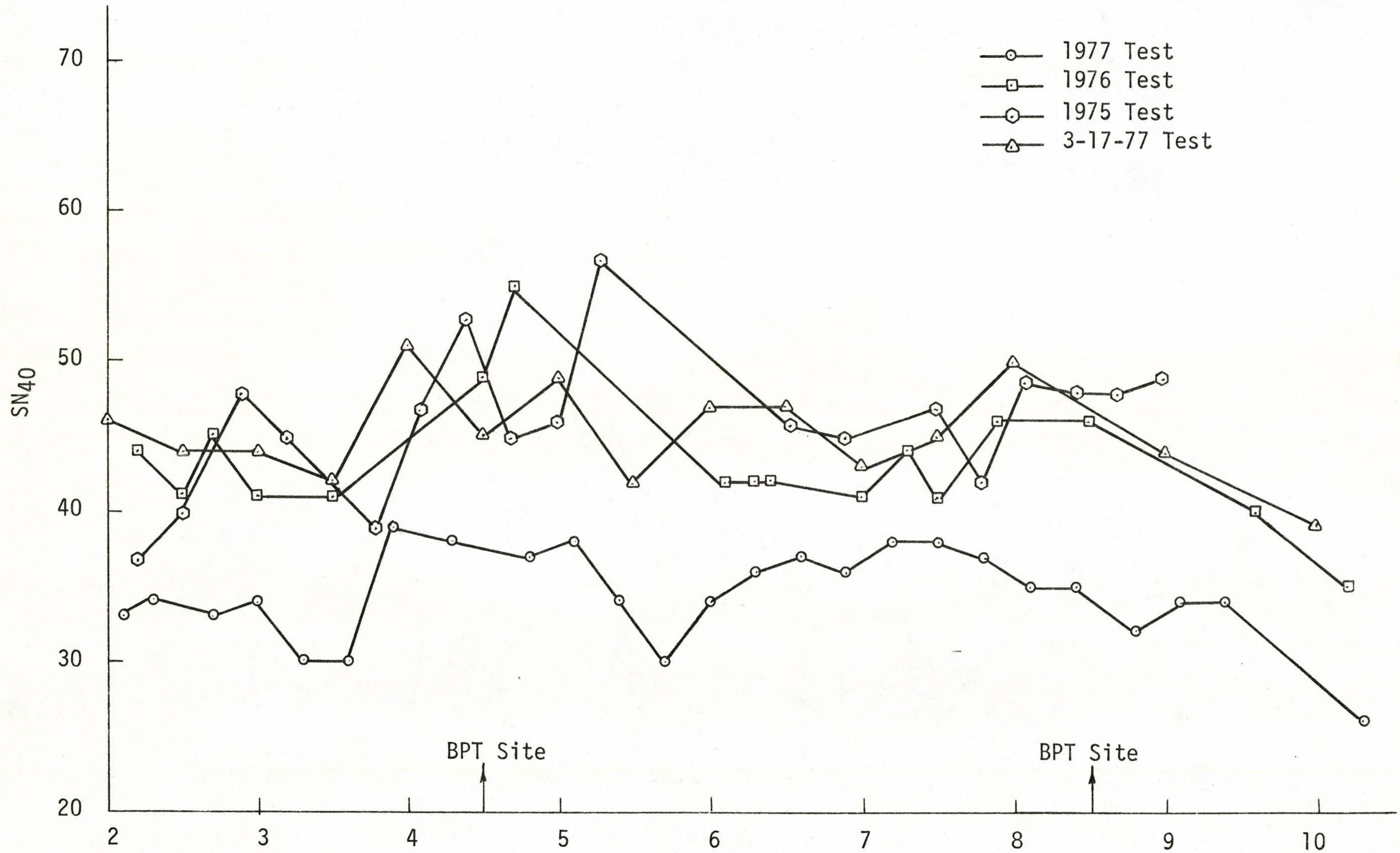


Figure 31. AHD Skid Trailer SN_{40} vs Road Log Mile - Big Rock Syenite, 88-9, Altheimer East

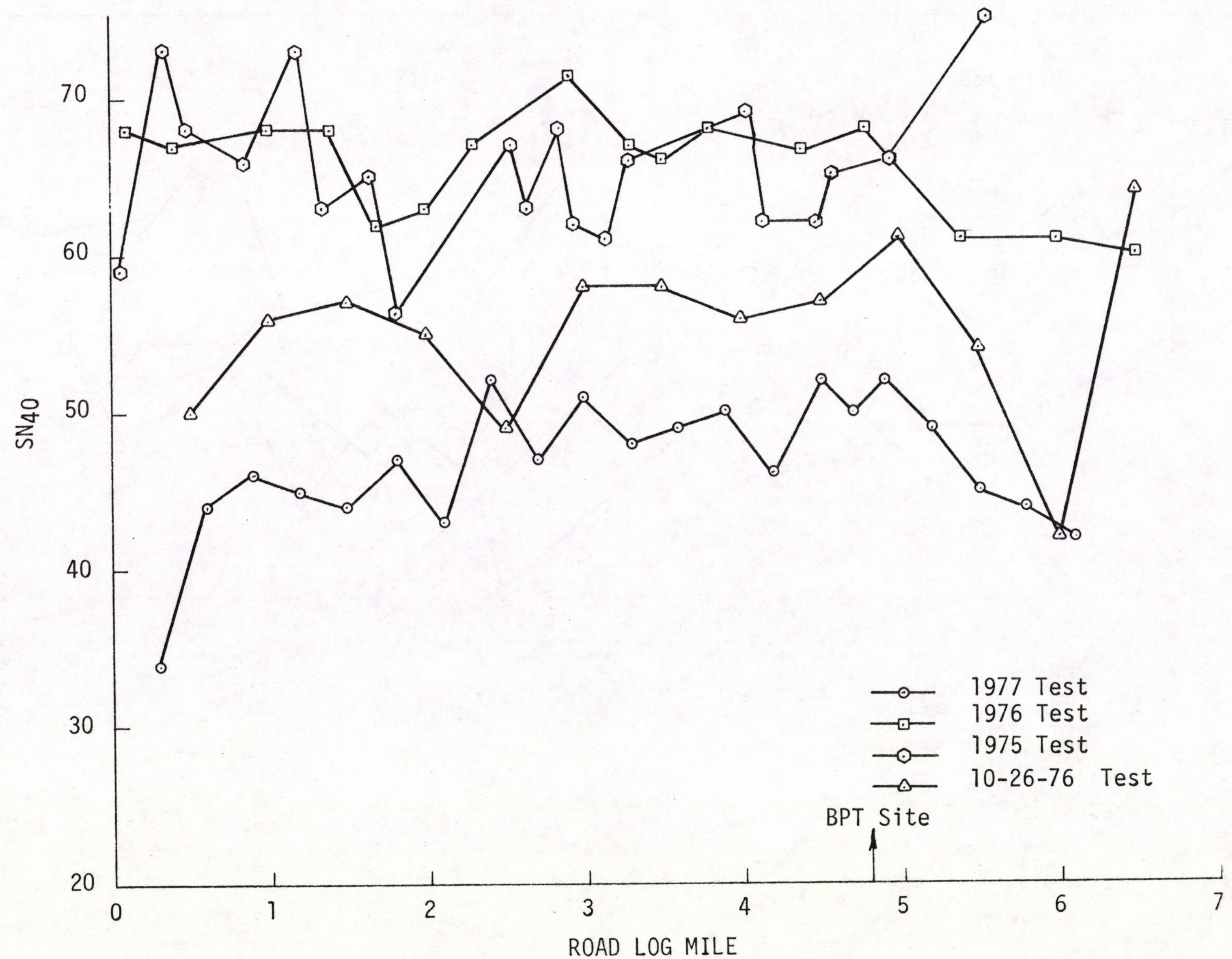


Figure 32. AHD Skid Trailer SN40 vs Road Log Mile - Malvern Novaculite, 70-10, Garland County East

westbound lane from Furlow toward the Pulaski County line. Job average SN_{40} values obtained in 1977 and 1976, respectively, were 35 and 60. The variations in skid numbers for these years are shown in Figure 33. However, these average SN_{40} values are misleading because the 1977 test included the pavement between road log miles 1.5 and 6, whereas the 1976 tests were only performed between road log miles 4.5 and 6. The routine AHD inventory skid tests shown in Figure 33 were performed on December 1, 1976, and covered the total length of the test section. These December SN_{40} test values are in close agreement with the results of the 1977 tests performed as a part of this investigation.

Summary of Skid Trailer Test Results. In general, the skid trailer SN_{40} values obtained at the BPT test sites varied widely between the June 1976 and June 1977 tests. These differences in yearly SN_{40} values are shown for each type of aggregate studied in Figures 27-33. For the test sections illustrated, the 1977 average SN_{40} value is about 40, which is about 14 "points" below the average 1976 SN_{40} values. The reason for this large difference in the yearly SN_{40} values is unknown. The 1977 SN_{40} data are summarized for each BPT test site as shown in Table XVI, and are used in this analysis. The equation relating the 1976 and 1977 SN_{40} values is presented in another section of this report.

On the basis of the job average SN_{40} for these test sections, the pavements would be ranked from high to low as: 1) Malvern novaculite, route 70-10; 2) Texarkana gravel, route 71-1; 3) Jenny Lind sandstone, route 71-13; 4) Johnson limestone, route 45-5; 5) Big Rock syenite, route 88-9; and 6) Johnson limestone, route 71-13 and 71-14. This list is only the SN_{40} ranking of the pavement test sections that were used to illustrate the variability of the skid trailer test results, and is

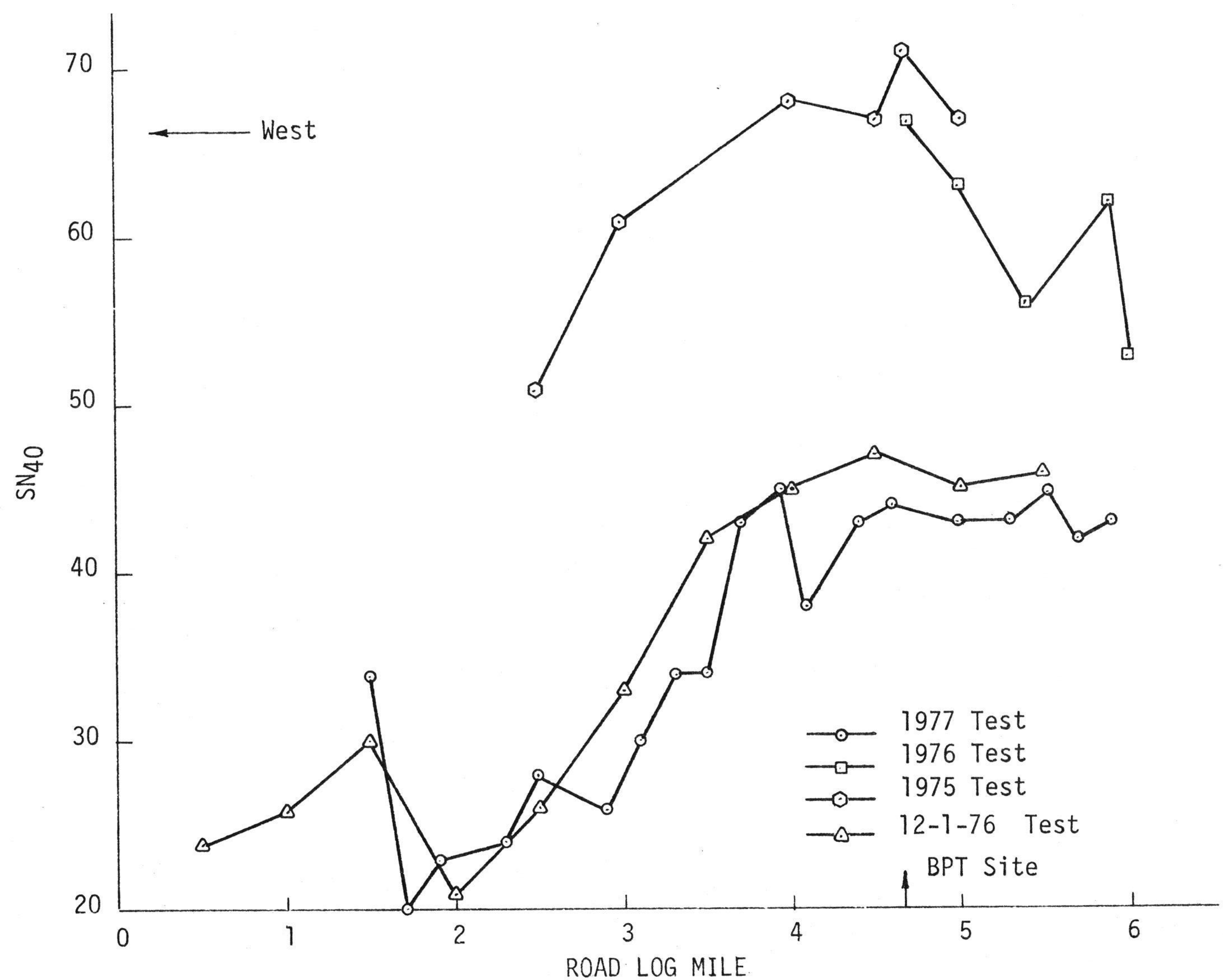


Figure 33. AHD Skid Trailer SN_{40} vs Road Log Mile - England Synthetic, 294-2, Furlow West

not necessarily representative of each type of aggregate investigated.

An attempt was made to correlate skid trailer test results with traffic. The relationship between the 1977 SN_{40} values and the estimated total wheel passes is shown in Figure 34. No data points are shown because of the low coefficient of correlation (R = 0.32) obtained. The relationship between SN_{40} and the average annual daily traffic gave an R value of only 0.21.

British Portable Tester Results

The BPT field test results of the average of the nine tests at each site are shown in Table XVI. The BPN values are reported for the IWP, BWP, and OWP tests along with the average BPN for the site. A fairly good relationship between the SN_{40} test site value and the average BPN value was obtained. Figure 35 is a graph of test results grouped by type of aggregate. The best fitted equation ($\text{Log BPN} = 1.27 + 0.313 \text{ Log } SN_{40}$) was obtained by regressing the BPN values for IWP and OWP against the SN_{40} value for each test site. However, the BWP test value (shown darkened) is also plotted in Figure 35 to show the variation in BPN values across the pavement surface. At most test sites, the BPN test value was highest for the between the wheel paths test and lowest for the outer wheel path test.

A plot of BPN number versus coefficient of friction at 30 mph is presented in Figure 6 for several different research projects. Comparison of the two graphs indicates a similar trend, and the BPN correlation with SN_{40} of Figure 35 approximates the slope of that shown for the 1962 Tappahannock study of Figure 6.

The test results from the different types of study aggregates used in pavements were grouped together for the plotting of Figure 35, which

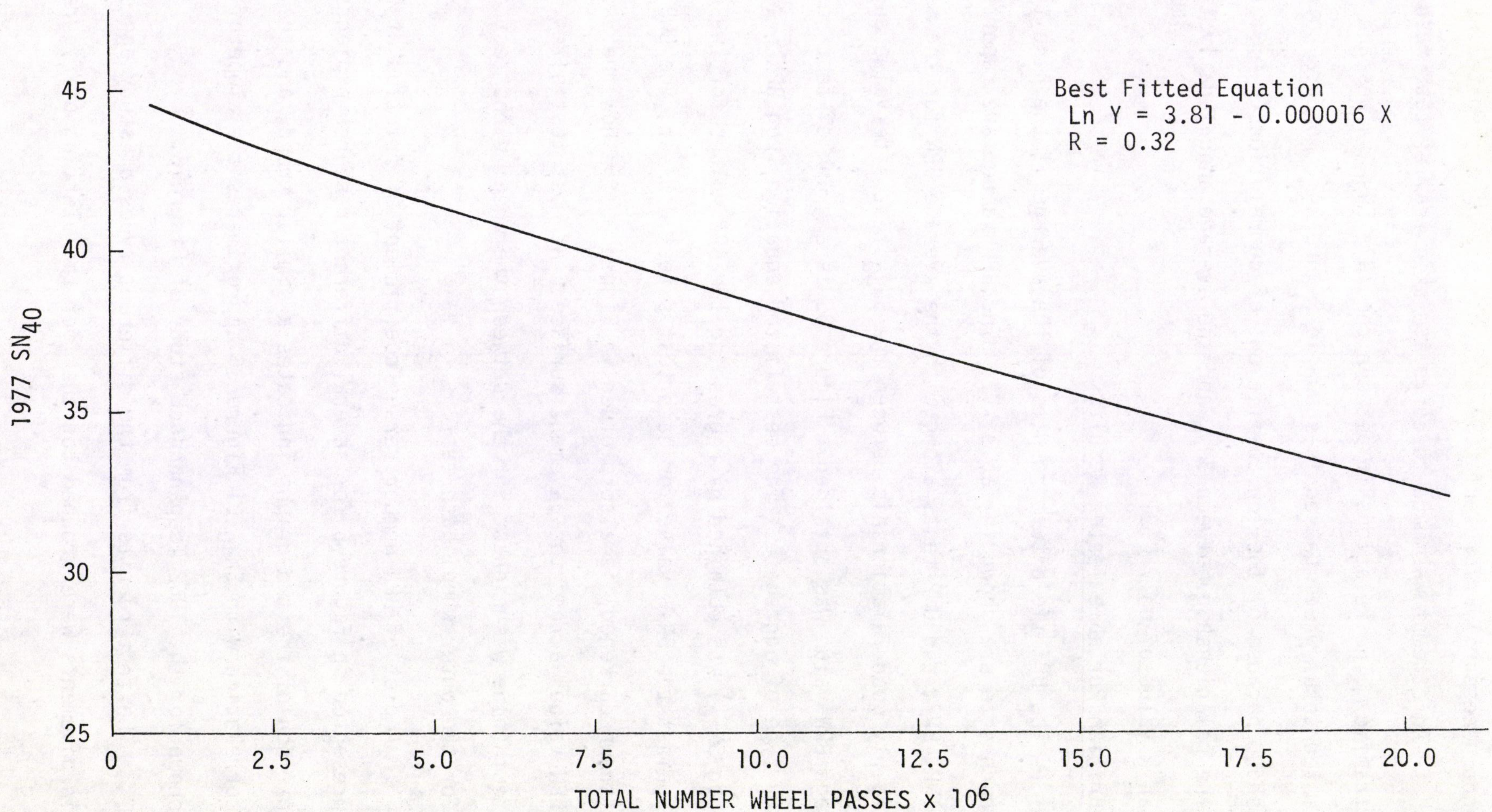


Figure 34. AHD Skid Trailer SN_{40} (1977) vs Total Number Wheel Passes

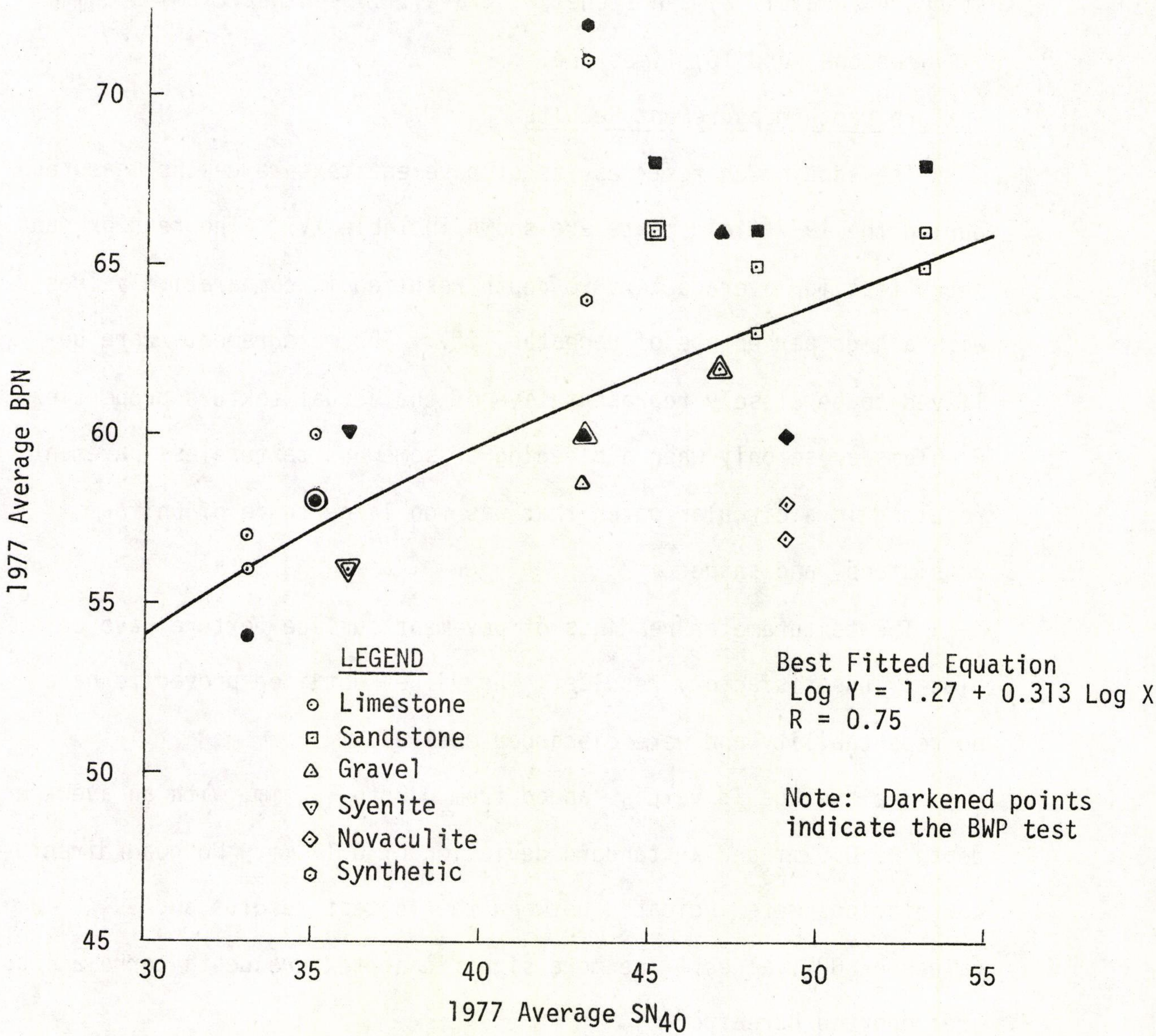


Figure 35. 1977 Average BPN vs 1977 Average SN_{40} - For IWP, BWP and OWP by Type of Aggregate

relates BPN values to SN_{40} values. Examination of the types of aggregate in this figure as they are plotted along the SN_{40} axis indicates the following rank from high skid number to low skid number for the 10 aggregates studied in the field: 1) sandstone, 2) novaculite, 3) sandstone, 4) gravel, 5) sandstone, 6) gravel, 7) synthetic, 8) syenite, 9) limestone, and 10) limestone.

Texture Depth Measurement Results

The sand patch test results of pavement texture depths measured during the 1977 field tests are shown in Table XVI. The regular sand patch test for average texture depth resulted in comparative values with a high percentage of repeatability. The measurements were believed to be closely representative of the actual texture properties. Problems arose only when a bleeding or somewhat textureless pavement resulted in a circular patch that was too large to be of uniform consistency and shape.

The texturemeter readings of pavement surface texture gave completely unsatisfactory results. The values obtained proved to have no repeatability and were discarded entirely.

The average Td values ranged from 0.2 to 1.1 mm, with an average depth of 0.5 mm and a standard deviation of 0.17 mm. No good direct correlations were indicated between the Td test results and SN_{40} values or BPN values. The more significant texture depth correlations are reported hereafter.

Laboratory Test Results

The laboratory tests performed were the polishing tests on the different aggregates and asphalt mixtures and the stripping or durability test performed on the same materials. The laboratory tests were carried out during the 4-year period from March 1, 1974, to March 1, 1978. Most of the testing work was done by civil engineering graduate and undergraduate students at the University of Arkansas.

Polishing Tests

Accelerated polishing tests were performed on each aggregate sample, asphalt mixtures of these aggregate samples, asphalt mixtures blended with two study aggregates, and pavement cores. The aggregate samples were tested both by the British Wheel method of polish and by the Arkansas Accelerated Polishing Device. All of the asphalt mixtures were polished on the APD.

Aggregate Polish Results. The results of the correlation work between the Arkansas Accelerated Polishing Device and the Texas British Wheel are shown in Figure 36. The 18 study aggregates are identified on the graph with their ultimate polish test values. The best fitted equation relating the two different methods of polish is: TPV = 11.1 + 0.806 APV, with an R value of 0.87. This equation permits the calculation of APV in terms of the TPV and vice versa.

For example, for the minimum aggregate polish values proposed by the Texas Highway Department as shown in Table IV, the Arkansas Polish Value equivalents are, respectively: TPV = 30, APV = 24; TPV = 33, APV = 27; and TPV = 35, APV = 30. The initial and final aggregate polish values obtained by the Texas British Wheel and the Arkansas Accelerated Polishing Device are shown in Table XVII. These test results are used to compare the APV results obtained from testing Marshall specimens molded with the same aggregate.

A polishing curve for each aggregate sample was determined in the

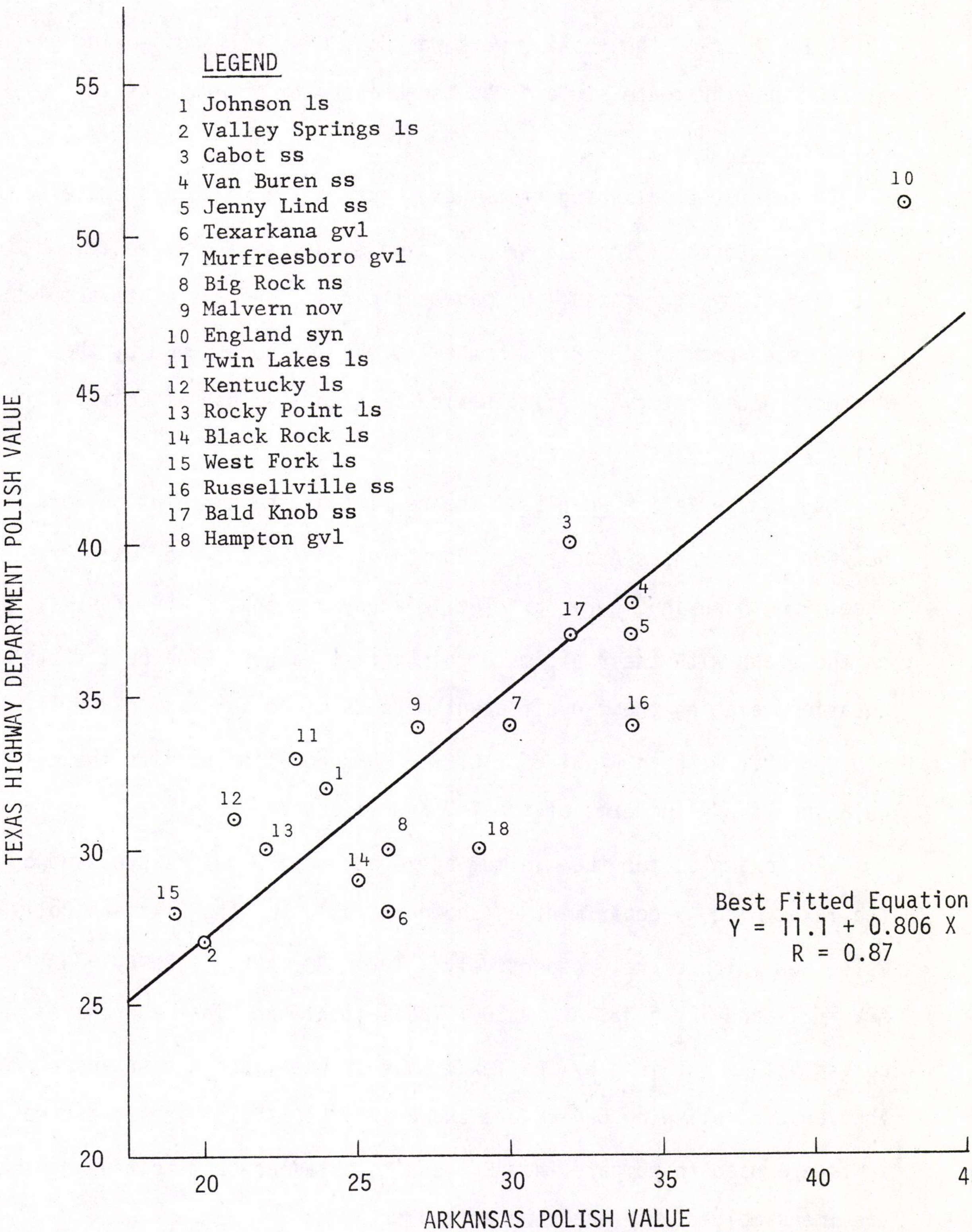


Figure 36. Relationship Between the Texas Highway Department Polish Value and the Arkansas Polish Value for Aggregates

Table XVII

Laboratory Polish Test Results - Aggregate and

Marshall Arkansas Polish Values

No.	Aggregate Used	Aggregate Sample				Marshall Sample	
		Texas Polish Value		Arkansas Polish Value		Arkansas Polish Value	
		initial	final	initial	final	initial	final
1	Johnson ls	45	32	36	24	36	23
2	Valley Springs ls	48	27	32	20	37	21
3	Cabot ss	49	40	41	32	36	32
4	Van Buren ss	50	38	42	34	36	32
5	Jenny Lind ss	47	37	42	34	37	25
6	Texarkana gvl	41	28	35	26	40	28
7	Murfreesboro gvl	46	34	37	30	39	28
8	Big Rock ns	44	30	32	26	40	28
9	Malvern now	50	34	40	27	38	28
10	England syn	52	51	46	43	38	25
11	Twin Lakes ls	47	33	37	23	38	24
12	Kentucky ls	45	31	32	21	38	26
13	Rocky Point ls	45	30	33	22	40	26
14	Black Rock ls	46	29	38	25	40	28
15	West Fork ls	44	28	34	19	36	21
16	Russellville ss	49	34	41	34	38	30
17	Bald Knob ss	39	37	39	32	38	30
18	Hampton gvl	41	30	33	29	38	27

laboratory. The Arkansas Polish Value varied as a function of the logarithm of polishing hours with the Accelerated Polishing Device. Typical polishing curves for each aggregate type are shown in Figure 37. The aggregate's initial APV ranged from 32 to 46, and that their frictional resistance was reduced greatly during the first hour or two of polishing time. The APV continued to decline with continued polish and the final polish value ranged from 20 to 43.

The England synthetic was the most polish resistant of the aggregates tested, followed by sandstone, novaculite, gravel, syenite, and limestone. Examination of these typical curves in Figure 37 indicates that the range in APV from limestone to sandstone is 35 to 20, or 15 points. The polishing curves are not smooth curves, but fluctuate up and down with polishing time indicating an overall decrease of friction with the decrease in microtexture of the aggregate surface and macrotexture of the aggregate particles.

The relationship between the APV and polish time (in hours) with the APD for each aggregate sample was determined and is shown in Table XVIII. The form of the polishing curve equation is: APV = A + B Log Time, where A is the intercept and B is the slope of the line. When the polishing time is equal to 1.0 hour, the APV is equal to the intercept value (A) shown in Table XVIII, as the logarithm of 1.0 is zero.

The slope of the line may be associated with the rate of polish of the particular sample, i.e., the most polish resistant aggregate was the England synthetic which has a slope of -0.95, and the least polish resistant aggregate (based on data of Table XVII) was the West Fork limestone with a -3.48 slope. These polish curve data and equations are based on a 20-hour test cycle of the APD; if this test cycle were changed, a dif-

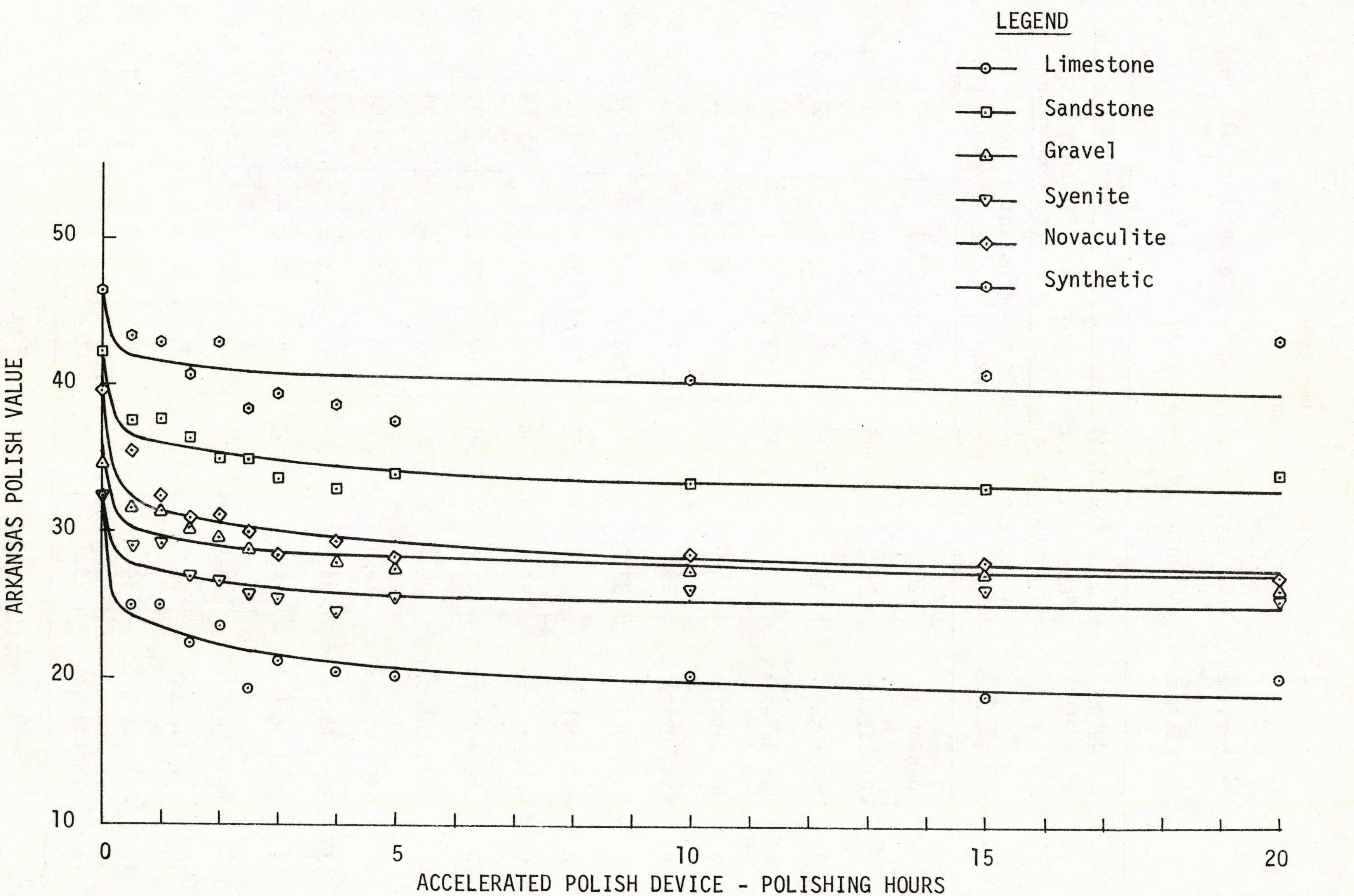


Figure 37. Typical Arkansas Polish Values vs Arkansas Accelerated Polish Device Polish Hours for Aggregate

Table XVIII

Laboratory Polish Test Results - Aggregate and Marshall Specimens, Arkansas Polish Value vs Hours Polished

No.	Aggregate Used	Aggregate			Marshall		
		Coefficients Best Fitted Equation[1]			Coefficients Best Fitted Equation[1]		
		constant (A)	slope (B)	C.C.[2] (R)	constant (A)	slope (B)	C.C.[2] (R)
1	Johnson ls	26.2	-3.10	0.94	28.3	-2.57	0.91
2	Val.Spr. ls	23.2	-3.19	0.95	29.0	-3.18	0.92
3	Cabot ss	35.2	-2.14	0.96	32.2	-1.51	0.83
4	Van Buren ss	35.3	-2.10	0.93	32.1	-1.54	0.79
5	Jen.Lind ss	36.0	-2.21	0.93	30.0	-2.62	0.96
6	Tex. gvl	29.9	-1.94	0.92	33.5	-2.32	0.87
7	Mur. gvl	32.4	-1.87	0.92	33.4	-2.09	0.85
8	Big Rock ns	27.4	-1.75	0.89	30.4	-3.07	0.99
9	Malvern nov	31.6	-3.04	0.95	30.0	-2.73	0.97
10	England syn	41.6	-1.41	0.62	30.2	-2.94	0.98
11	Twin Lakes ls	27.7	-3.32	0.99	30.0	-3.08	0.95
12	Kentucky ls	23.4	-2.78	0.98	29.6	-3.01	0.94
13	Rocky Point ls	24.2	-2.89	0.98	31.3	-3.37	0.96
14	Black Rock ls	29.1	-3.21	0.98	31.7	-2.95	0.96
15	West Fork ls	22.6	-3.48	0.97	27.1	-3.12	0.91
16	R'ville ss	35.5	-1.73	0.95	33.6	-1.92	0.83
17	Bald Knob ss	34.9	-1.69	0.94	33.4	-1.97	0.88
18	Hampton gvl	29.7	-0.95	0.88	31.0	-2.65	0.93

[1] Form of Equation: APV = A + B log Time (hours)

[2] Coefficient of Correlation

ferent polish curve would be obtained. The polish curve for each aggregate sample is presented subsequently and discussed with the asphalt mixture polish curve of the same aggregates.

Asphalt Mixtures Polish Results. Several different mixtures were tested for their polishing tendencies with the APD: Marshall specimens molded for each study aggregate, Marshall specimens molded with blends of two study aggregates, and pavement cores taken from the study roads.

These combined mixtures totaled to about 65 polishing curves, representing the 18 aggregates being studied. The best fitted equation representing each Marshall mixture is shown in Table XVIII, with the aggregate polish equations previously discussed. The coefficient of correlation of the Marshall specimen curves ranged from 0.79 for the Van Buren sandstone to 0.99 for the Big Rock syenite. The Marshall specimen's initial and final APV's obtained are shown with the aggregate values in Table XVII; thus the end test points of each type of test specimen can be compared.

Typical Marshall specimen polishing curves for each type of aggregate are shown in Figure 38. These curves were drawn on a four-cycle semilogarithm graph paper. The initial polish value is assumed to occur at 0.001 hours of polishing time. The assumption is necessary because the logarithm of zero is infinity.

The curves shown in Figure 38 are for the best fitted equations as determined by regression analysis. The more polish resistant Marshall mixtures for these typical curves, ranked from high to low at the end of the 20-hour polishing cycle, are gravel, sandstone, novaculite, synthetic, syenite, and limestone.

Marshall specimens were prepared from blends of the study aggregate samples. The blends were mixed together mostly in equal proportions

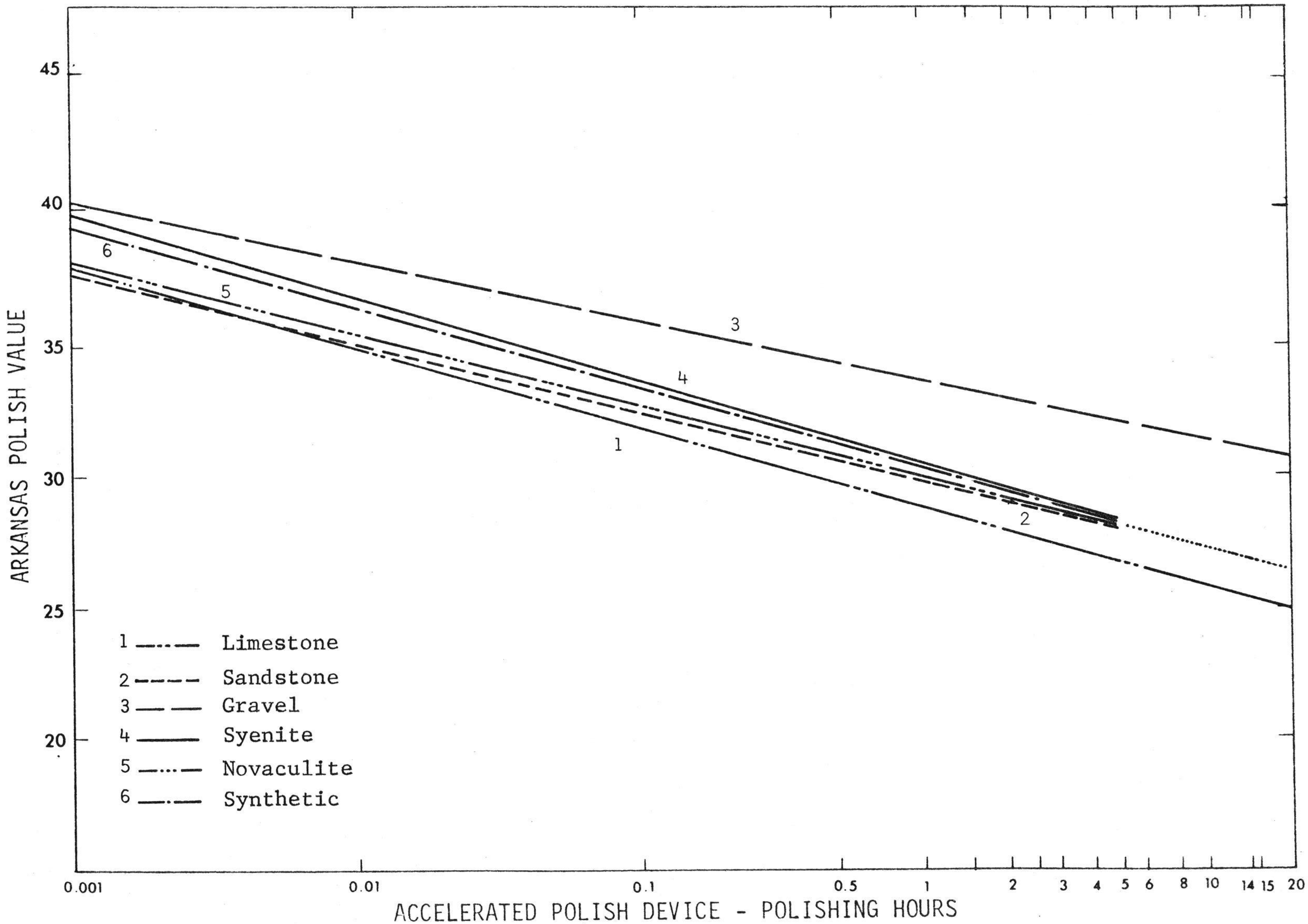


Figure 38. Typical Arkansas Polish Values vs Arkansas Accelerated Polish Device Polish Hours for Marshall Specimens

by weight, and molded for accelerated polish testing. All of the aggregates which were selected for field testing were used in the blending work. The 15 aggregate blends and their best fitted polishing equations are shown in Table XIX. The aggregate used and the number of blends incorporated into a mix are, respectively: Johnson ls (3), Valley Springs ls (6), Cabot ss (2), Van Buren ss (2), Jenny Lind (1), Texarkana gvl (1), Murfreesboro gvl (3), Big Rock ns (3) Malvern nov (4), and England syn (4). The R values for the best fitted lines range from 0.83 to 0.99, and indicate a fairly good fit for the test points for these blends.

The results of the laboratory polishing and texture measurements on the pavement core samples are shown in Table XX. This table contains the initial (as received from the field) APV and Td values and also the final APV and Td values after 20 hours of polishing on the APD. Each core sample consisted of 2 IWP specimens, 2 BWP specimens, and 2 OWP specimens, for which the initial APV and Td values were obtained. Only 3 of these 6 specimens were polished, one each from the IWP, BWP, and OWP locations. Thus, the final APV and Td values reported in Table XX are only for tests on three specimens for each core sample.

On the basis of the data reported in Table XX for the initial core APV and the data shown in Table XVI for the field BPN values from the same point, a relationship between APV and BPN was established. The best fitted equation obtained by the regression analysis is APV = 7.02 + 0.467 BPN. The coefficient of correlation for this relationship is 0.87. Thirty-two of the 60 BPT test sites are included in this correlation work.

The relationship established between field BPT test results and laboratory BPT test results is shown in Figure 39. With the relationship established, it is possible to relate APV values in the laboratory to BPN

Table XIX

Laboratory Polish Test Results - Blended Aggregate Marshall Specimens, Arkansas Polish Value vs Hours Polished

No.	Aggregate (50% of each unless noted)	Coefficients Best Fitted Equation[1] Form: APV = A + B log Time constant (A)	slope (B)	Corr. Coef.[2] (R)
1	Johnson ls + Van Buren ss	30.3	-2.67	0.98
2	Johnson ls + Malvern nov	30.6	-3.20	0.99
3	Johnson ls + England syn	31.7	-2.42	0.90
4	Valley Spgs. ls + Cabot ss	31.0	-1.97	0.89
5	Valley Spgs. ls + Murf. gvl	30.3	-2.77	0.97
6	Valley Spgs. ls + 30% Malvern	27.8	-1.56	0.92
7	Valley Spgs. ls + 60% Malvern	28.4	-1.19	0.85
8	Valley Spgs. ls + 30% England	27.8	-1.37	0.88
9	Valley Spgs. ls + 60% England	28.3	-1.42	0.89
10	Cabot ss + Big Rock ns	31.9	-1.79	0.83
11	Van Buren ss + Texarkana gvl	30.9	-2.37	0.98
12	Jenny Lind ss + Murfreesboro	31.4	-2.79	0.97
13	Texarkana gvl + England syn	31.2	-3.16	0.93
14	Murfreesboro gvl + Big Rock syn	31.1	-3.33	0.95
15	Big Rock ns + Malvern nov	31.4	-2.27	0.91

[1]Form of Equation: APV = A + B log Time (hours)

[2]Coefficient of Correlation

Table XX

Laboratory Polishing Test Results - Core Samples

BPT Test Site No.	Aggregate	Route-Section	Polish Values IWP/BWP/OWP/Avg		Texture Depths[1] IWP/BWP/OWP/Avg	
			Initial	Final	Initial	Final
2	Johnson	16-3	36/33/36/35	23/26/26/25	4.7/3.7/4.7/4.4	3.6/3.2/3.7/3.5
4	Johnson	71-17	33/32/30/31	24/27/24/25	4.5/3.8/4.1/4.2	3.7/4.1/3.4/3.7
5	Johnson	45-5	34/37/35/35	23/27/24/25	5.0/5.4/4.5/5.0	3.4/4.4/2.9/3.6
7	Johnson	62-2	33/35/33/33	29/28/27/28	5.8/5.0/4.1/5.0	4.4/4.9/3.2/4.2
10	Valley Spr.	65-1	35/30/33/33	27/23/25/25	5.6/6.0/4.8/5.5	4.4/4.4/3.4/4.1
11	Valley Spr.	62-7	35/36/33/34	23/26/26/26	4.4/4.7/3.9/4.4	3.2/3.2/3.2/3.2
12	Valley Spr.	62-7	35/37/35/36	25/27/27/27	5.4/4.4/4.4/4.7	3.4/3.2/4.1/3.6
15	Valley Spr.	65-4	33/34/36/34	25/27/28/	4.5/6.4/4.8/5.3	3.4/6.3/3.4/4.4
17	Cabot	5-15	35/41/37/38	32/30/29/30	7.5/6.0/8.2/7.3	6.8/5.4/7.5/6.6
19	Cabot	5-14	41/41/40/40	30/30/29/30	7.5/6.0/6.8/6.8	6.8/5.0/6.3/6.0
20	Cabot	5-12	40/39/35/38	29/31/29/30	5.4/3.6/6.9/5.3	5.8/4.4/4.9/5.0
21	Van Buren	71-15	42/41/39/40	33/33/33/33	5.3/5.4/5.5/5.4	3.6/5.0/5.0/4.5
22	Van Buren	162-1	35/36/35/35	31/32/31/31	3.4/5.6/3.3/4.1	3.4/4.7/3.6/3.9

Table XX (continued)

Laboratory Polishing Test Results - Core Samples

BPT Test Site No.	Aggregate	Route-Section	Polish Values IWP/BWP/OWP/Avg		Texture Depths[1] IWP/BWP/OWP/Avg	
			Initial	Final	Initial	Final
24	Jenny Lind	71-10	34/35/33/34	32/32/31/32	3.5/3.8/3.7/3.6	3.8/4.1/3.4/3.8
25	Jenny Lind	71-13	37/39/37/38	29/32/31/31	4.1/4.2/4.4/4.3	4.1/3.6/3.6/3.8
27	Jenny Lind	71-13	40/40/40/40	31/32/32/32	5.7/5.2/4.7/5.3	4.7/5.0/4.1/4.6
29	Jenny Lind	71-14	35/34/34/34	28/30/30/29	3.0/2.5/3.1/2.8	2.9/2.9/2.9/2.9
32	Jenny Lind	96-3	39/42/39/	32/33/33/33	6.0/7.2/5.6/6.3	5.0/6.3/4.7/5.3
33	Texarkana	71-1	37/37/36/36	25/25/23/24	6.5/4.1/5.8/5.5	6.3/5.4/5.8/5.8
35	Texarkana	71-1	34/36/33/34	22/26/22/23	6.3/4.8/5.5/5.6	6.8/5.4/5.0/5.7
38	Texarkana	82-2	30/33/32/32	25/24/25/25	5.5/4.2/6.1/5.3	8.2/6.3/7.5/7.3
40	Murfreesboro	71-2	34/35/34/34	26/30/27/28	5.2/4.4/4.8/4.8	5.8/4.7/5.4/5.3
41	Murfreesboro	32-3	40/43/38/40	33/33/31/32	6.0/4.8/5.8/5.6	6.8/4.4/6.3/5.8
44	Murfreesboro	27-2	36/37/36/36	29/30/27/29	6.3/5.8/6.6/6.3	5.0/5.0/5.0/5.0
46	Murfreesboro	4-3	37/41/40/39	31/29/29/30	6.5/6.3/6.4/6.4	6.3/6.3/5.4/6.0
49	Big Rock	15-9	28/33/31/30	24/24/25/24	5.6/4.1/5.4/5.0	4.7/3.8/5.0/4.5

Table XX (concluded)

Laboratory Polishing Test Results - Core Samples

BPT Test Site No.	Aggregate	Route-Section	Polish Values IWP/BWP/OWP/Avg		Texture Depths[1] IWP/BWP/OWP/Avg	
			Initial	Final	Initial	Final
51	Big Rock	88-9	33/34/32/33	25/26/24/25	5.0/5.8/5.4/5.4	3.8/4.7/3.8/4.1
52	Big Rock	11-5	33/35/32/33	26/27/26/26	5.2/4.7/4.9/5.0	4.1/4.4/3.8/4.1
54	Big Rock	70-8	30/32/28/30	24/26/30/27	6.9/6.0/5.8/6.3	5.4/5.0/6.8/5.7
56	Malvern	5-7	33/38/35/35	30/34/28/31	9.5/6.4/9.0/8.4	7.5/5.8/8.2/7.2
57	Malvern	70-9	35/37/34/35	25/25/26/25	8.2/4.8/7.5/6.9	6.3/5.4/6.3/6.0
60	England	294-2	40/40/34/38	31/32/32/32	11.9/16.4/16.1/14.8	5.4/8.2/10.9/8.2

[1]Texture Depths in mm x 10^{-1}

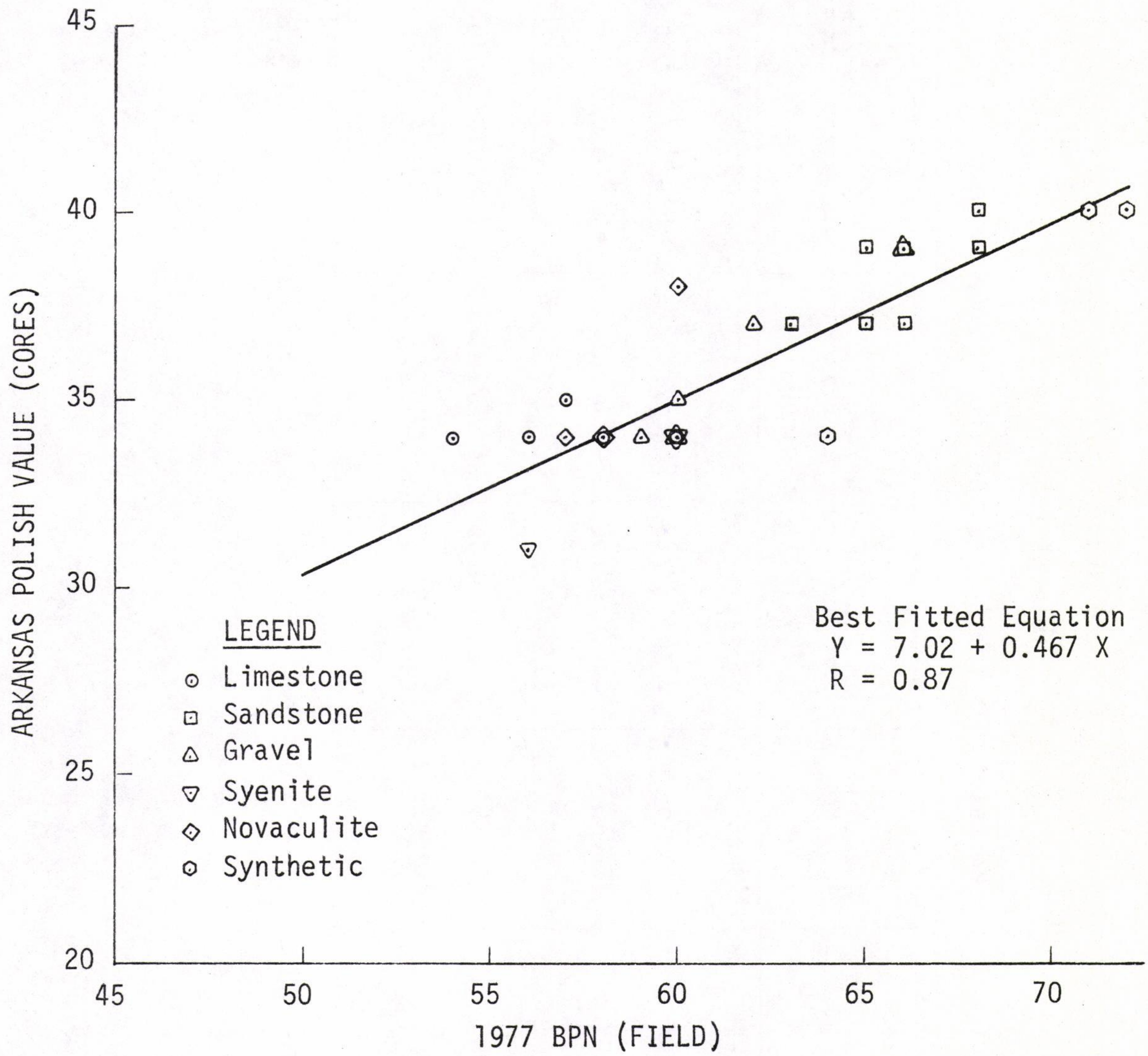


Figure 39. Relationship Between Arkansas Polish Value (core tests) and 1977 BPN (field) for Asphalt Pavements

values in the field. Because the field BPN values have been related to SN_{40} values, as shown in Figure 35, the interrelationship between APV values and SN_{40} values can be established. The best fitted equations to be used are:

a) BPN vs SN_{40} ($R = 0.75$)

$$\text{Log BPN} = 1.27 + 0.313 \text{ Log } SN_{40}$$

b) APV vs BPN ($R = 0.87$)

$$\text{APV} = 7.02 + 0.467 \text{ BPN}$$

Thus, if an SN_{40} value of 35 is desired, the laboratory ACHMSC sample would require an APV of 33 to provide the desired level of pavement friction.

An examination of the data points plotted in Figure 39 indicates that all of the pavements evaluated (based on the core tests) would have an SN_{40} of 35 or greater except the syenite test section. The foregoing relationships for APV, BPN, and SN_{40} are based only on the 1977 test results.

The core Td test results shown in Table XX are for initial polish and after accelerated polishing. Of interest is the change in Td resulting from the action of the APD. The average initial Td was 0.56 mm with a standard deviation of 0.20 mm; after accelerated polishing, the average Td was 0.49 mm with a standard deviation of 0.13 mm.

The Td values of the sandstone and gravel cores were not affected very much by the APD action; in fact, some of them increased. This increase in Td possibly resulted from a "pop out" of one of the larger aggregate particles from the surface of the core.

Comparison of Polish Test Results

The separate results of the polishing tests performed in the laboratory on aggregates, Marshall mixtures, Marshall blends, and pavement cores have been presented. These polish test results from the different mixtures can be compared most easily by a graphic presentation.

Therefore, the polish test curves relating APV and hours of polish for cores and parent aggregate follow. The polishing curves for the 10 aggregates which were evaluated in the field, as test study pavements are discussed briefly. The polishing curves for the other eight aggregates (Twin Lakes ls, Kentucky ls, Rocky Point ls, Black Rock ls, West Fork ls, Russellville ss, Bald Knob ss, and Hampton gvl) are in Appendix E.

The polish test curves relating APV and hours of polish for blends of the various aggregates tested are in Appendix F. The polish curves are arranged in Appendix F in the same order that they are shown in Table XIX.

Johnson Limestone Aggregates and Mixtures. The Johnson limestone aggregate's polishing curves shown in Figure 40 are for the pure aggregate specimens, Marshall specimens, and a typical core specimen. The best fitted equations for the three curves are shown in the figure, as well as the R value obtained. Only the test data points for the aggregate and Marshall polish curves are indicated. The Marshall mix has a higher polish resistance than the pure aggregate specimen. However, the typical core test results indicated that the core was more polish resistant than the Marshall mix.

Figure F1 shows the polish values curve results for an equal blend of Johnson limestone and Van Buren sandstone. In this figure, the test data points are plotted for the blend and the pure Marshall mixtures' curves are drawn with no data points. The Johnson Marshall polish curve

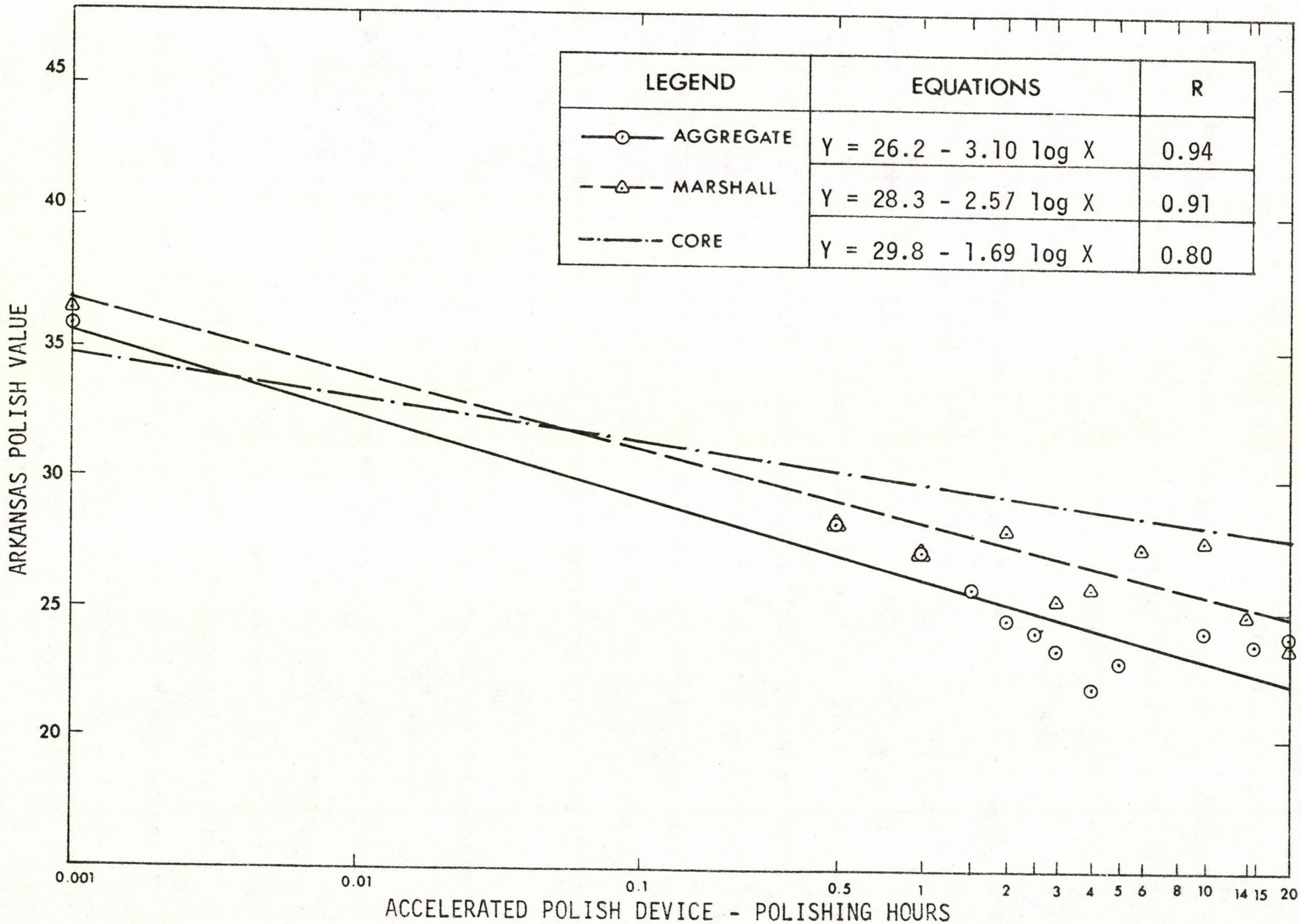


Figure 40. Arkansas Polish Value vs. Hours of Polish - Aggregate, Marshall, and Typical Core - Johnson Limestone

is also plotted on Figure 40, where the laboratory specimens' polish curves are compared with the field core polish curve. (For all of the samples reported on, the Marshall specimen polish data of the parent material and the aggregate specimen polish data are plotted on the figure that compares laboratory specimens with field core specimens.)

In Figure F1, the polish curve for the blend of Johnson and Van Buren aggregates in a Marshall specimen lies approximately midway between the two polish curves for the separate Marshall mixes. These results appear to agree with those of Mullen et al, (14) shown in Figure 5.

An equal blend of Johnson limestone and Malvern novaculite was used for the Marshall polish curve shown in Figure F2. The blended mixture had a higher APV than either parent material until after 15 hours of polishing. The plotted points representing the blended mixture are somewhat typical of a majority of the polishing curves, as the specimen would polish, then after continued wheel passes the specimen would "roughen up", giving a higher polish value, but with continued polishing, the specimen would again polish and indicate a lower APV.

Figure F3 presents the results of the polishing test for a blend (equal volumes) of Johnson limestone and England synthetic in a Marshall mixture. The blend indicated a higher skid resistance than either of the parent materials when they were tested separately in the asphalt mixture. Examination of the data of Table XVIII indicates that the England synthetic aggregate specimen was much more resistant to polish than was its Marshall specimen.

<u>Valley Springs Aggregates and Mixtures</u>. The Valley Springs limestone typical core specimen polishing curve is shown in Figure 41. As was observed for the Johnson limestone, the typical core specimen is more

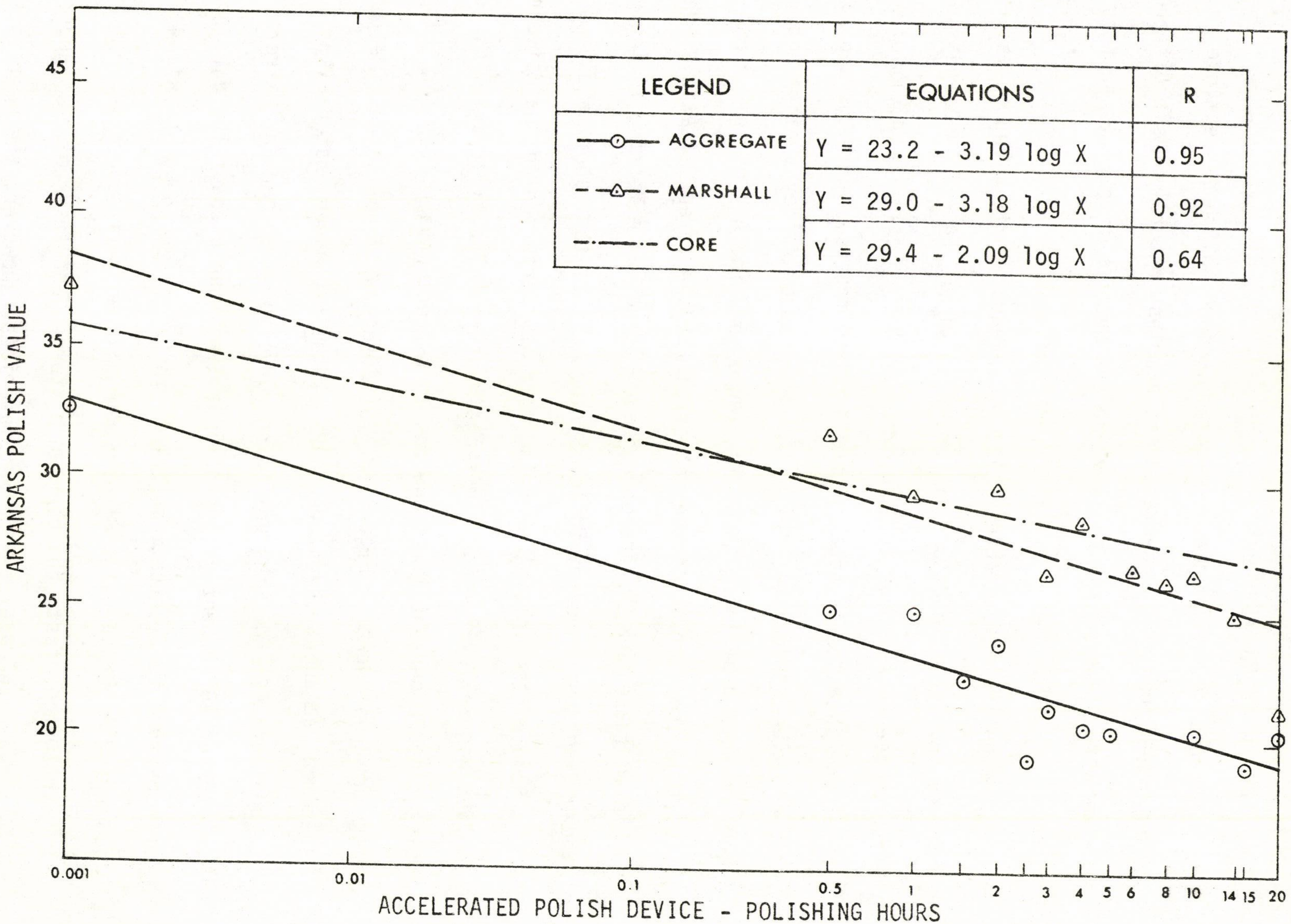


Figure 41. Arkansas Polish Value vs. Hours of Polish - Aggregate, Marshall, and Typical Core - Valley Springs Limestone

polish resistant than the Marshall specimen, which is more polish resistant than the pure aggregate specimen.

Because this aggregate seemed to be more polish susceptible than any of the other field pavement test samples, a total of six different blends were prepared and polished to evaluate their effects on this limestone.

The polishing characteristics of an equal parts blend of Valley Springs and Cabot sandstone are shown in Figure F4. This blend indicates an increase in APV over that of the pure Valley Springs specimen. The final polish value is about midway between the values of the two parent materials. The polish results of the blend of Valley Springs and Murfreesboro gravel are shown in Figure F5. The APV of this blend is less than the average of the values of the two parent materials, which would indicate that Murfreeboro gravel is not as effective in increasing the polish resistance of Valley Springs as was the Cabot sandstone.

Some of the initial blending work was performed on the Malvern novaculite and Valley Springs limestone mixture. The first trial mix consisted of 70 percent limestone and 30 percent novaculite, and this polishing curve is shown in Figure F6. At 20 hours of polishing time, the specimen had about the average APV of the two parent materials.

Figure F7 shows the polishing curve for a 60 percent Malvern novaculite and 40 percent Valley Springs limestone blend. The data points are observed to fluctuate more than those for the previous blend, as reflected by the R value of 0.85 for this best fitted curve. At the end of the 20 hour test, the blended mixture's end test point is about midway between those of the parent materials. However, the best fitted curve indicates the APV of the blend is greater than that of either parent material after about 14 hours of the APD operation.

The second series of the Valley Springs blends were made with the England synthetic aggregate. Polish test results of 70-30 percent (by volume) blends of Valley Springs and England are shown in Figure F8. The polish rate of this blend is less than that of either parent material and its ultimate APV is just less than the value obtained for a pure England Marshall specimen.

Figure F9 shows the results of a 40-60 percent (by volume) blend of Valley Springs and England. This polish curve indicates the ultimate APV of the blend is equal to that of the parent England material when tested as a Marshall specimen.

<u>Cabot Sandstone</u>. The polish curve for a typical core from a Cabot sandstone pavement is shown in Figure 42. The upper curve is for the Cabot sandstone aggregate polish test, and the lower curve is for the Marshall specimen molded with Cabot sandstone. The core's ultimate APV was between the values obtained by the aggregate polish test and the Marshall polish test. These curves indicate that the in-place mixture (core) was actually better than the Marshall specimen, but not as good as the aggregate specimen.

The results of blending Cabot sandstone with Valley Springs limestone are shown in Figure F4. The Cabot sandstone was also blended in equal proportion with the Big Rock syenite aggregate. This polishing curve is shown in Figure F10. The ultimate polish of the blend is better than that of the syenite, but does not equal that of the sandstone. The APV of this blend is superior to the value of either parent material up to about three hours of polishing time.

<u>Van Buren Sandstone</u>. The polishing curves for this material tested

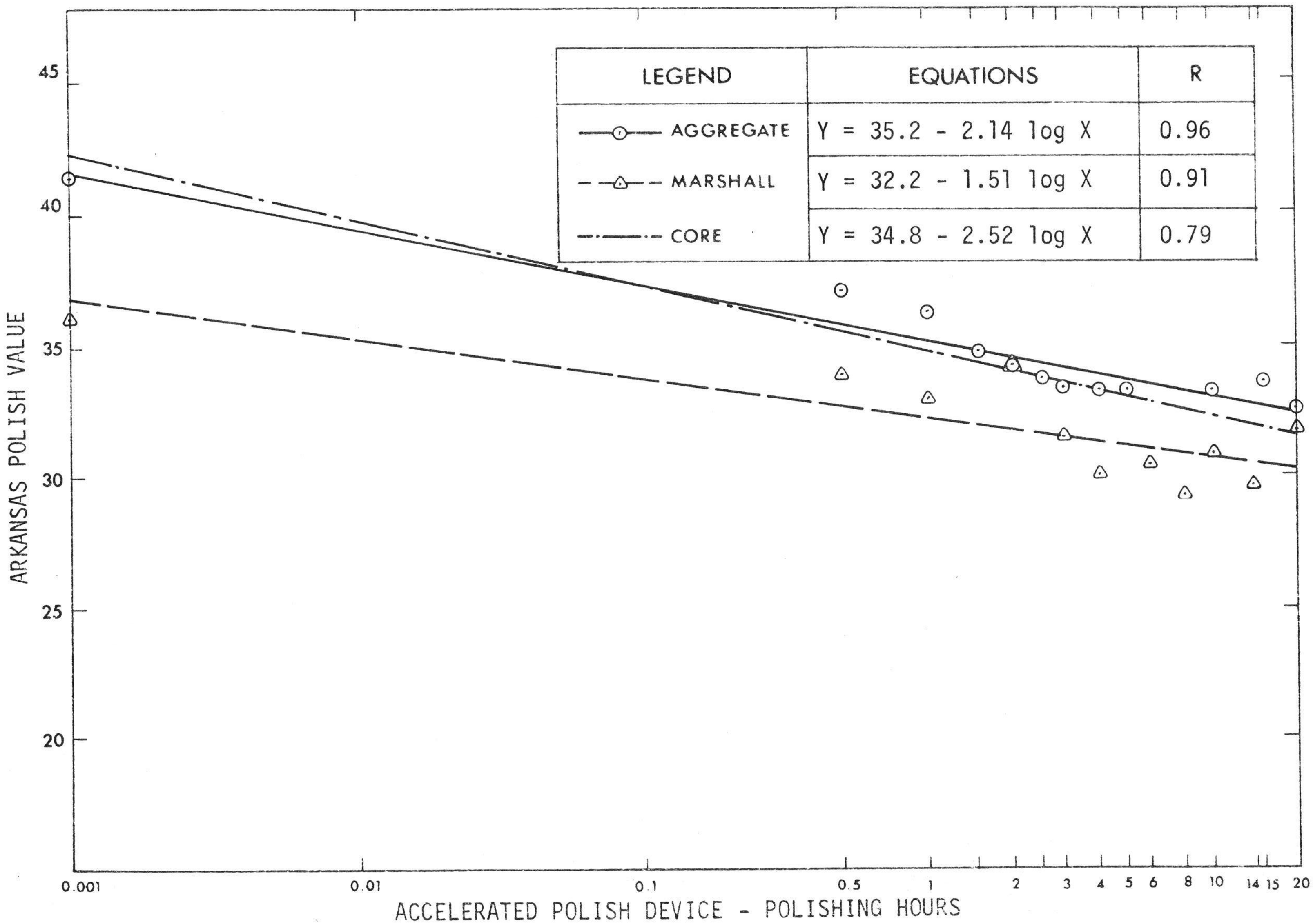


Figure 42. Arkansas Polish Value vs. Hours of Polish - Aggregate, Marshall, and Typical Core - Cabot Sandstone

as an aggregate, Marshall specimen, and a typical pavement core are shown in Figure 43. This core was taken from an ACHMSC Type 3 mix and indicates a higher polish resistance than the parent aggregate. The Marshall specimen indicates a lower polish resistance than the parent aggregate.

The polish curve obtained from a blend of Van Buren sandstone and Texarkana gravel is shown in Figure F11. The blend of these two materials indicates a lower polish value than either one of the parent materials. The polish curve (Figure F11) for the Texarkana gravel Marshall specimen indicates a better resistance to polish than that of the Van Buren sandstone Marshall specimen, although it polishes at a faster rate.

Jenny Lind Sandstone. The results of the polishing test on a typical core specimen of this aggregate are shown in Figure 44. The polish curve for the core is parallel to, but slightly lower than, the pure aggregate polish curve. The Marshall specimen polish curve is considerably lower than the aggregate curve.

A blend of Jenny Lind sandstone and Murfreesboro gravel indicates a polish curve approximately midway between the curves of the two parent materials. This graph is shown in Figure F12. Of interest is the fact that the Murfreesboro gravel Marshall specimen has a higher polish resistance than is indicated for the Jenny Lind sandstone Marshall specimen.

Texarkana Gravel. The polish curve relationships between the typical core, aggregate specimen, and Marshall specimen are shown in Figure 45. The Marshall specimen test data are fairly scattered, with an R value of 0.87, but indicate a higher resistance to polish than that of the pure aggregate specimen. However, the typical core polish curve

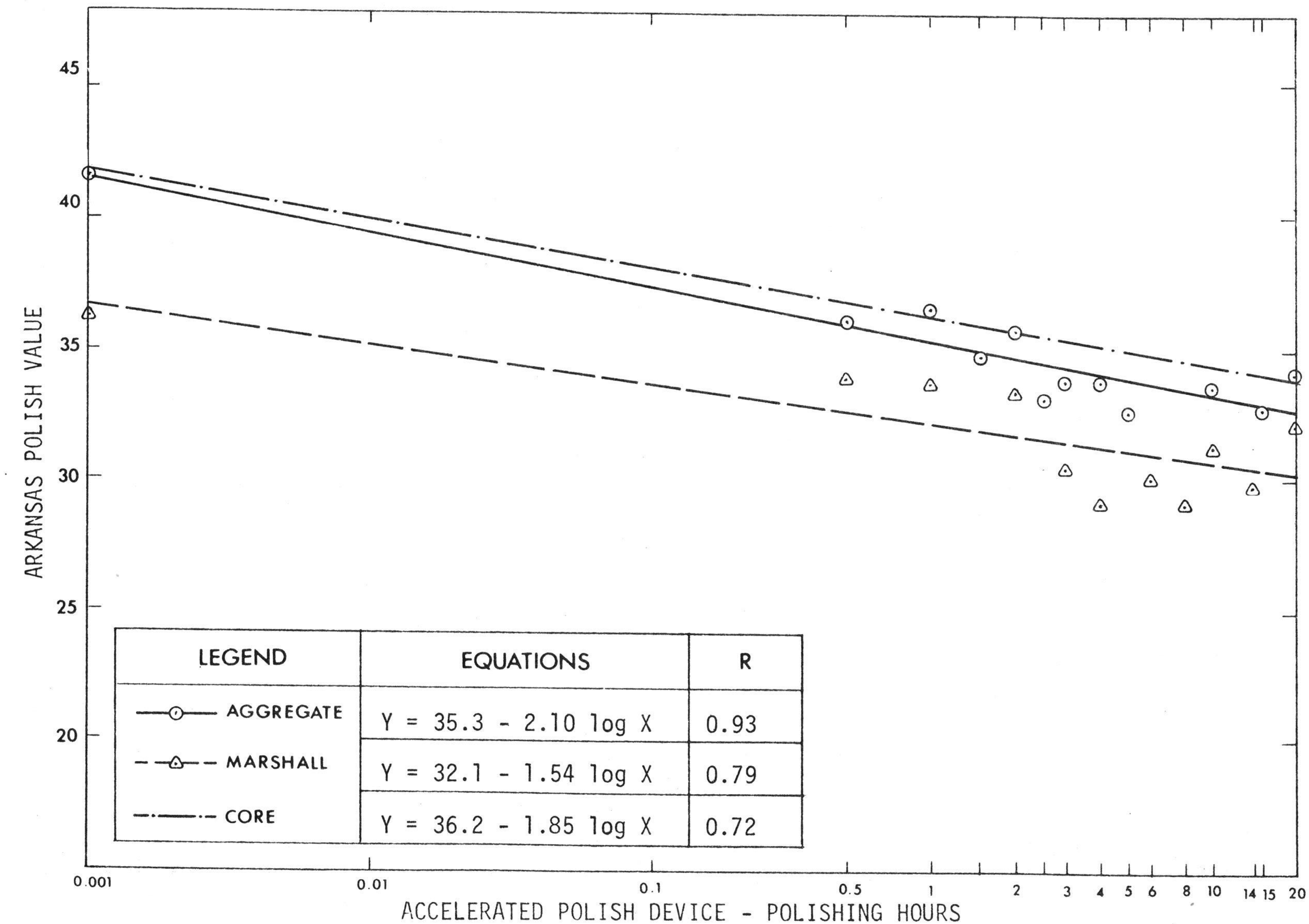


Figure 43. Arkansas Polish Value vs. Hours of Polish - Aggregate, Marshall, and Typical Core - Van Buren Sand e

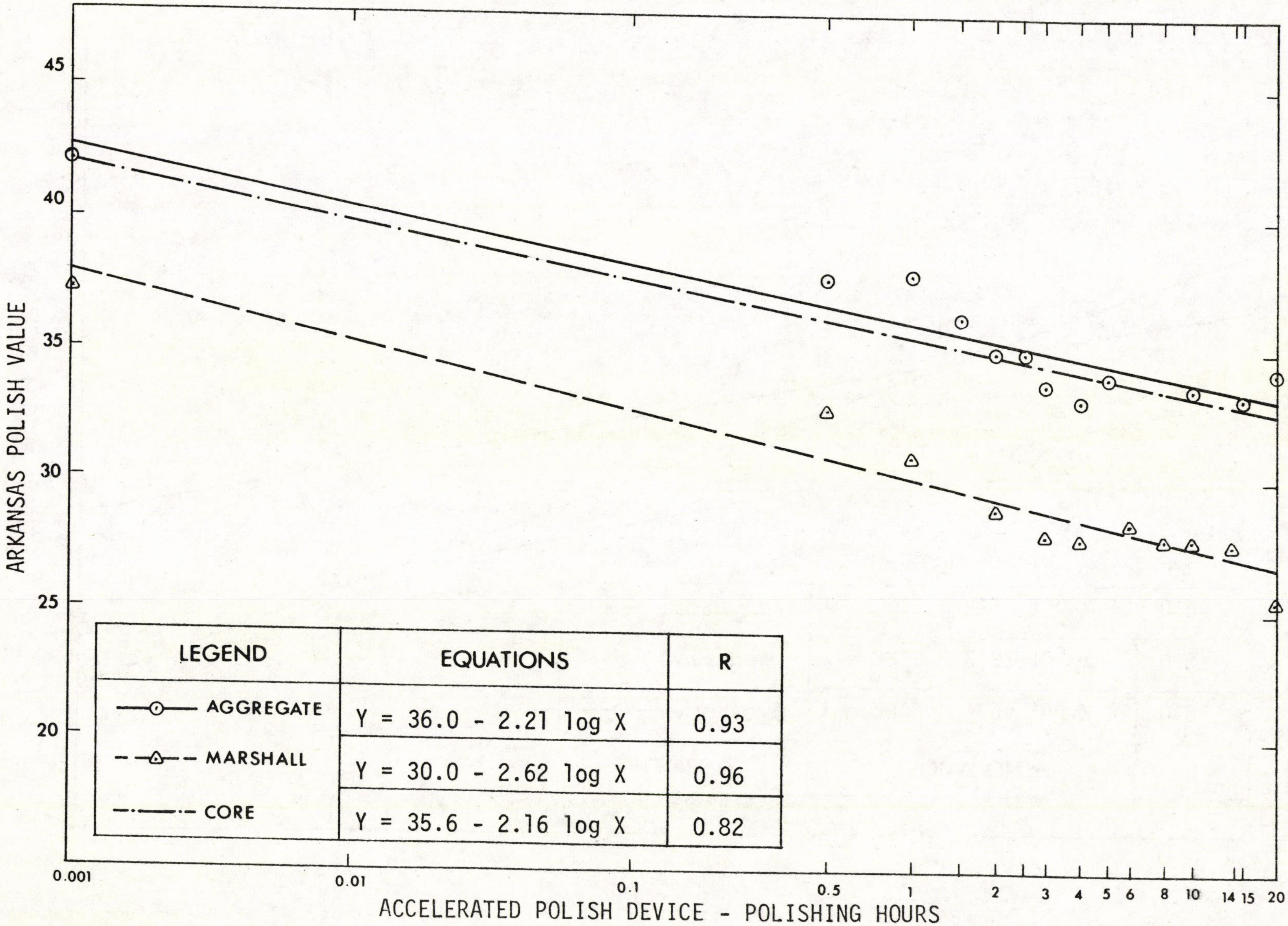


Figure 44. Arkansas Polish Value vs. Hours of Polish - Aggregate, Marshall, and Typical Core - Jenny Lind Sandstone

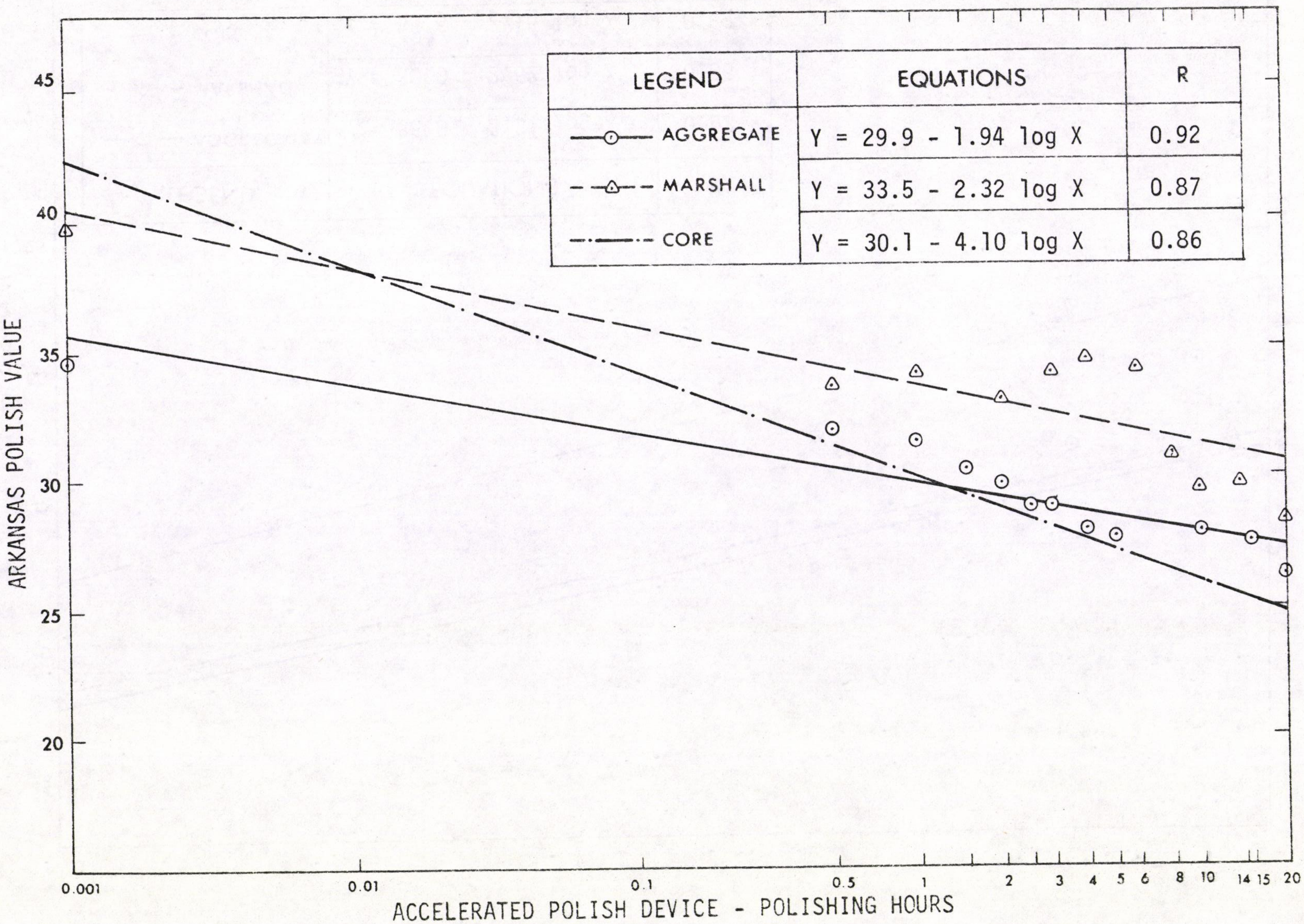


Figure 45. Arkansas Polish Value vs. Hours of Polish - Aggregate, Marshall, and Typical Core - Texarkana Gravel

shows rapid polish and is 5 or more APV numbers below the aggregate specimen curve at the end of the 20 hours of polish.

A blend of equal parts by volume of the Texarkana gravel and England synthetic was prepared and the polish curve for this mixture is shown in Figure F13. The ultimate APV of the blend is far below that of the Texarkana gravel and approaches the polish value of the England synthetic Marshall specimen.

Murfreesboro Gravel. The polish curves for the typical core, aggregate, and Marshall specimen are shown in Figure 46. The Marshall specimen indicates a greater resistance to polish than does the aggregate specimen. However, the typical core specimen was more resistant to polish than the Marshall specimen until the end of the 20-hour test, when they were equal.

The Big Rock syenite was blended with the Murfreesboro gravel on an equal weight basis. The polish curve for this blend is shown in Figure F14, with the Marshall specimen curves of the parent materials. The polishing resistance characteristics of this blend are between those of the parent materials.

Big Rock Syenite. The comparative polish curves for the aggregate, Marshall specimen, and typical core of this material are shown in Figure 47. As was true for the limestone aggregates tested, the Marshall specimen indicated a better resistance to polishing than did the aggregate specimen. The typical core polish test curve is about midway between the two laboratory specimens' polish curves.

The polishing characteristics of Big Rock syenite and Malvern novaculite in a blend were determined by using equal parts of each material. The results of the polish test of this blend are shown in Figure F15,

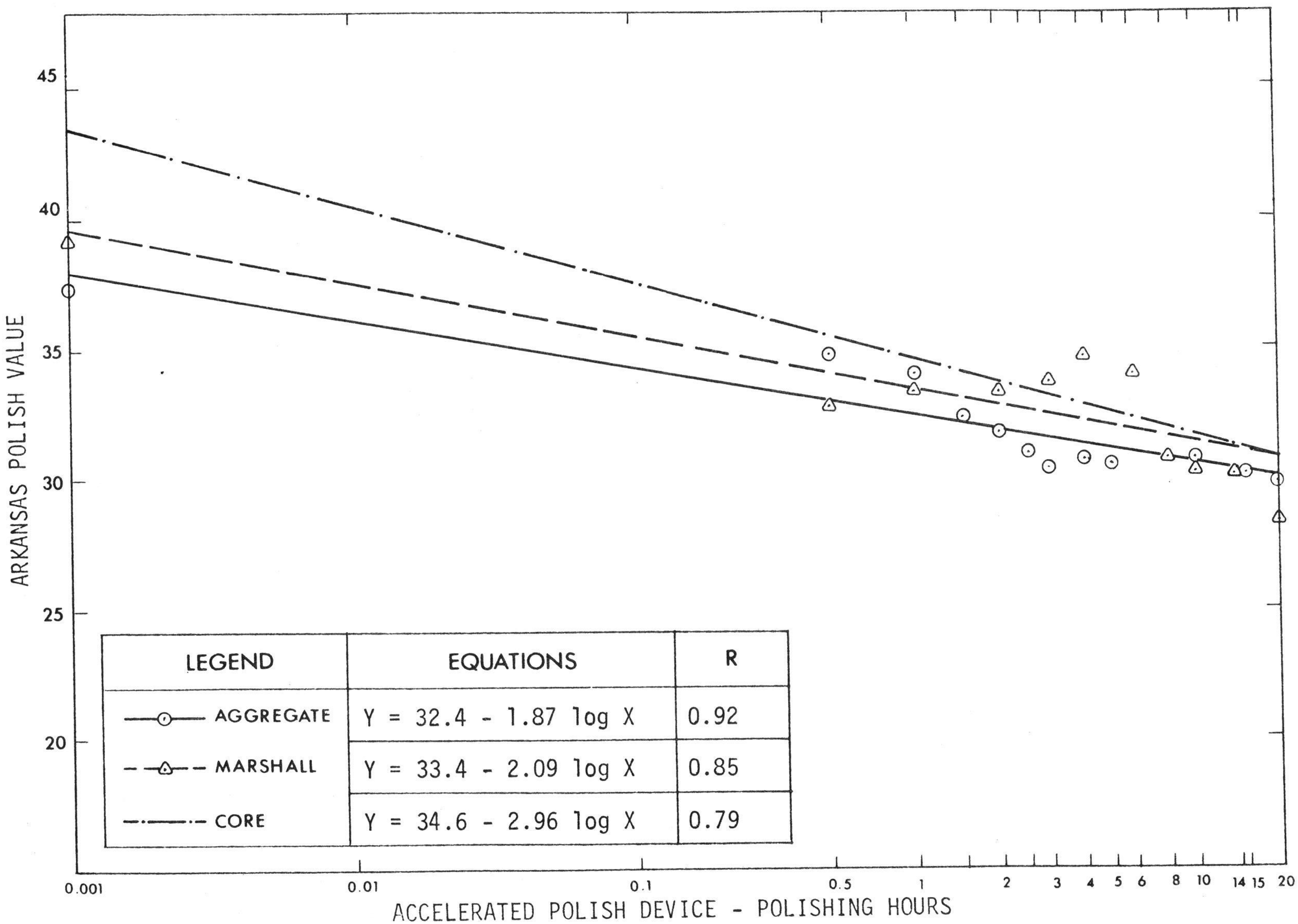


Figure 46. Arkansas Polish Value vs. Hours of Polish - Aggregate, Marshall and Typical Core - Murfreesboro G 1

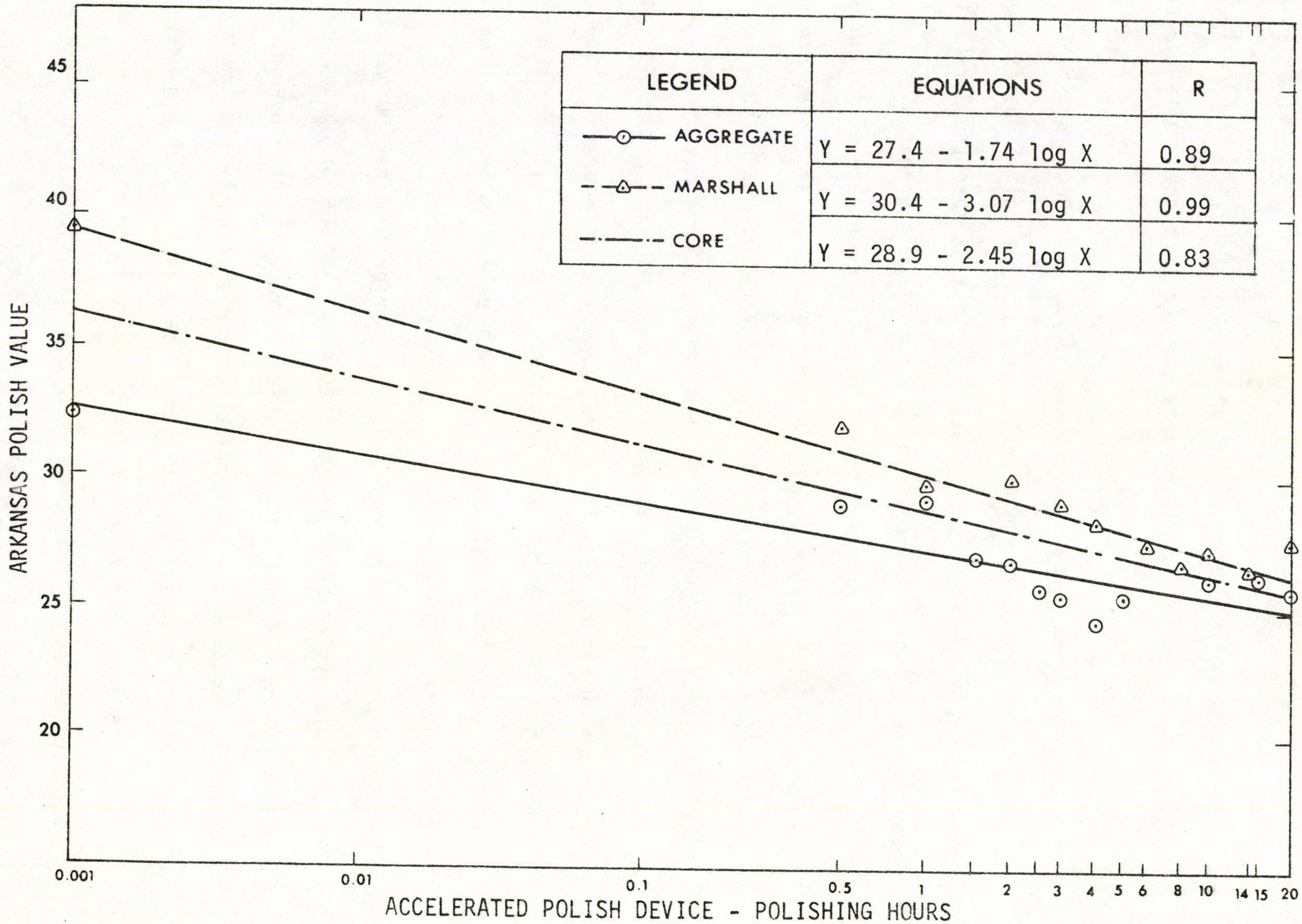


Figure 47. Arkansas Polish Value vs. Hours of Polish - Aggregate, Marshall, and Typical Core - Big Rock Syenite

with the Marshall polish curves of the parent materials. The Malvern novaculite has a lower rate of polish than the Big Rock syenite, but they are equal in polish value at the end of the 20-hour polish cycle. Their blend's best fitted curve indicates a slightly higher polish value than that of either parent material.

Malvern Novaculite. The typical core, aggregate, and Marshall specimen polish curves are shown in Figure 48. The polish curves show that the aggregate specimen is more polish resistant than the Marshall specimen, as was usually the case for the sandstone aggregates.

The typical core specimen indicates a lower rate of polish and is in fact superior to the aggregate specimen. Similar trends are shown for the cores from Murfreesboro gravel, Van Buren sandstone, Valley Springs limestone, and Johnson limestone.

England Synthetic. The polish curves for this artificial aggregate are shown in Figure 49. The aggregate specimen is more resistant to polish than the pavement core specimen. The Marshall specimen of this material indicates a substantially lower polish value than does the core or the aggregate specimen.

Some problems were encountered in performing the APD polish test on the pure synthetic aggregate specimen. The polishing and scrubbing action of the APD rubber tires caused degradation and pop-outs of the aggregate particles. The difficulty is reflected by the low R value of 0.62 for this best fitted equation. The polish values measured with the BPT on the aggregate specimen are believed to be higher than they should have been because of the uneven surface condition of the specimen.

Ultimate Polish and Weather Effects

One cycle of APD testing of particular interest was the determina-

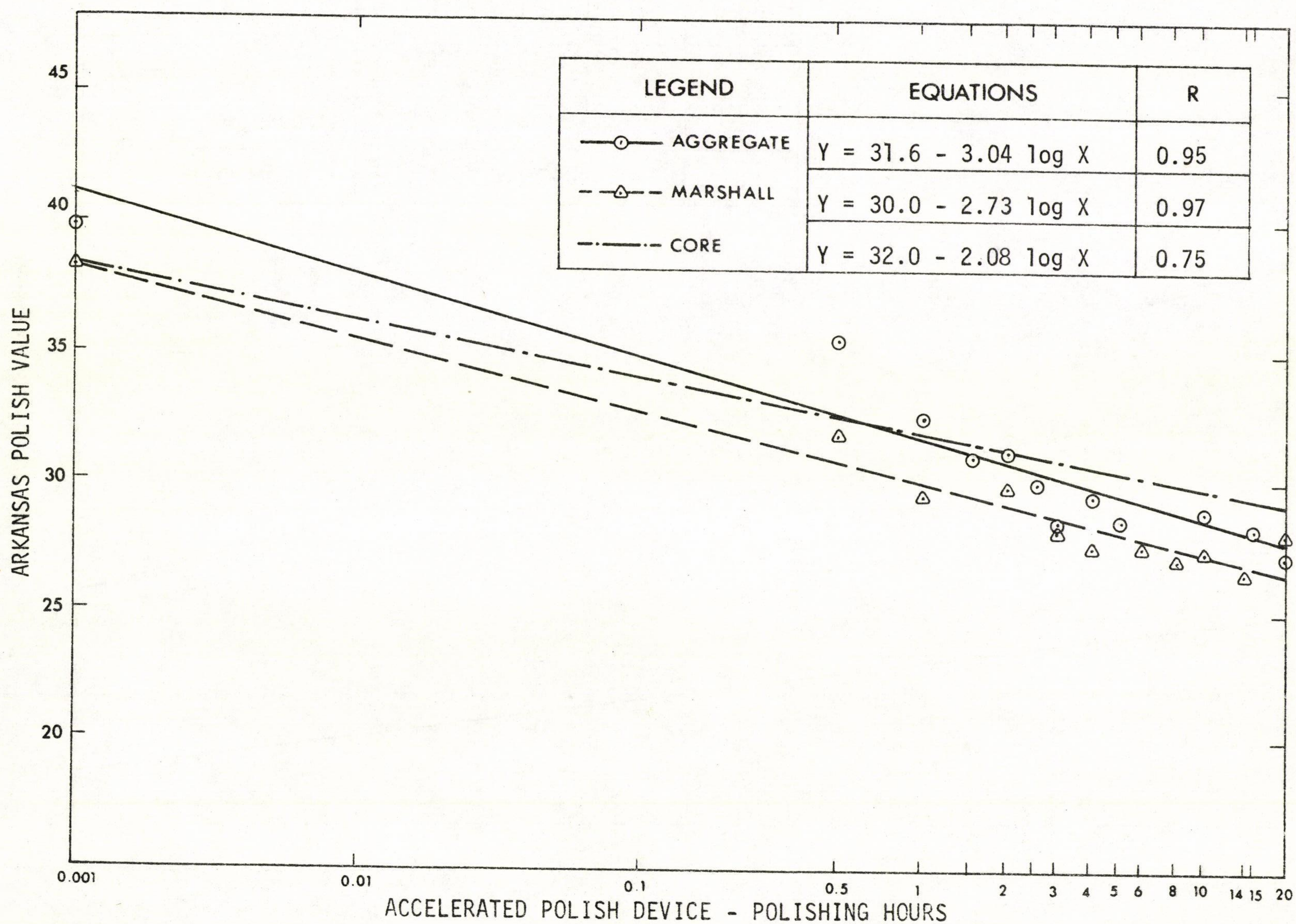


Figure 48. Arkansas Polish Value vs. Hours of Polish - Aggregate, Marshall, and Typical Core - Malvern Novaculite

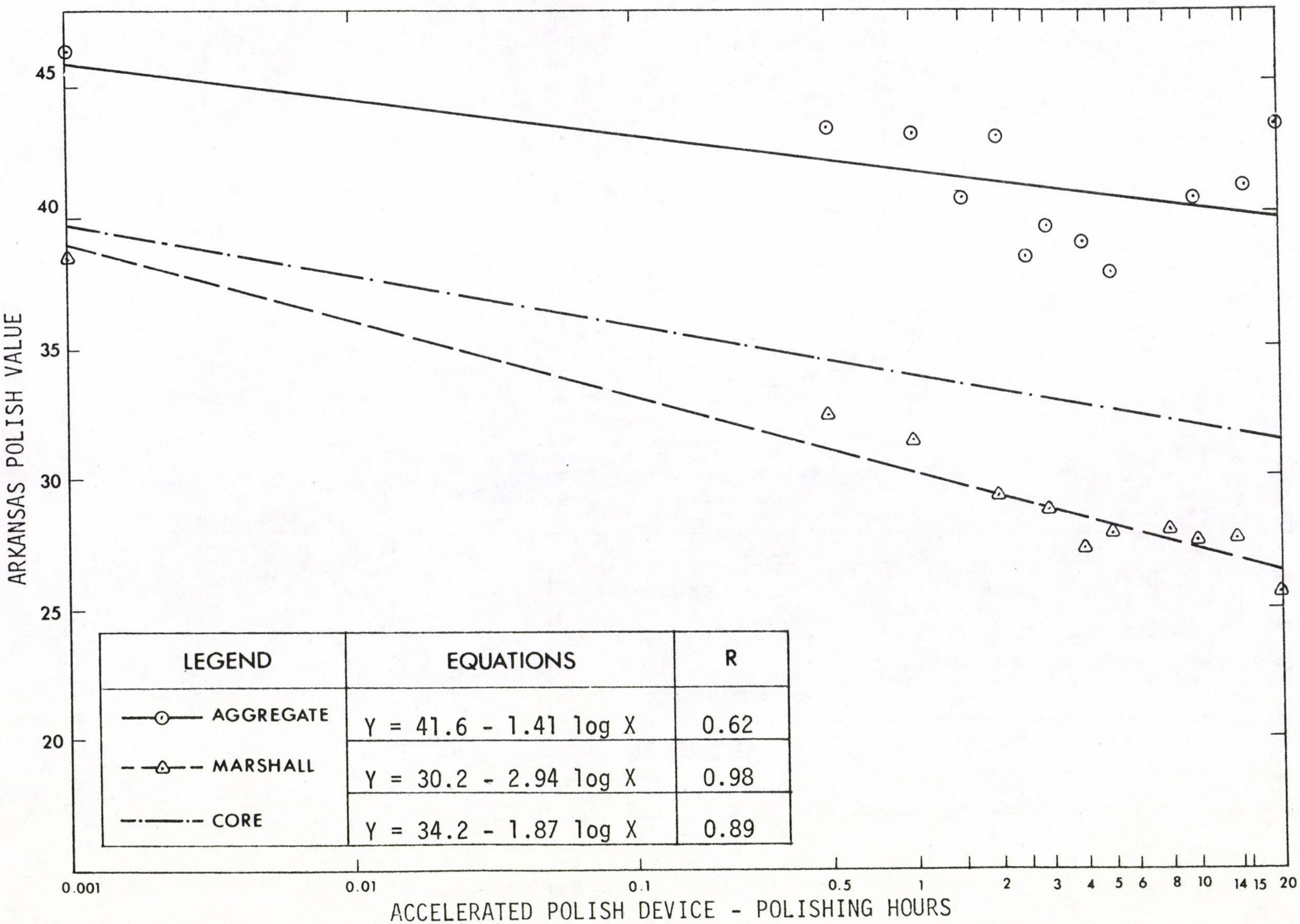


Figure 49. Arkansas Polish Value vs. Hours of Polish - Aggregate, Marshall, and Typical Core - England Synthe

tion of the absolute maximum polish that could take place in an asphalt mixture. After the specimens from the Valley Springs limestone and Twin Lakes limestone were polished for the regular 20-hour test cycle, they were removed from the device and the top 1/4 in. was sawed off with the diamond-studded blade of the masonry saw. The specimens, with the sawed surface on top, were then reinstalled on the APD. The "initial" APV with the BPT averaged about 32 and 30, respectively, for the Valley Springs and Twin Lakes specimens. After 30 minutes of APD time, their APV's were equal at 17; after 20 hours of polish the Valley Springs APV was 14 and the Twin Lakes APV was 16. This polish cycle was then continued for a total of 40 hours at which time the specimens had each roughened 2 points for an APV of 16 and 18, respectively. This experiment showed the minimum APV of these Marshall mixtures to be around 15.

All of the APD Marshall specimens, after polishing, were placed outdoors in open boxes to observe the effect of weathering on their polish value. After remaining outdoors from April 1977 to January 1978 (approximately 9 months), the APD specimens were brought indoors and their APV determined. The sawed specimen of Valley Springs had an APV of 21 and the Twin Lakes specimen had an APV of 24. This phenomenon of specimens increasing in polish value after weathering was observed in all cases. As a matter of fact, the polish values of the Marshall specimens were equal to or greater than the initial APV before their APD polish test.

Durability Tests

The tests performed to examine the durability of the aggregate's and asphalt mixtures were: Los Angeles abrasion, sodium sulfate soundness, particle shape, APD wear rate, film stripping, and immersion-compression. The results of these tests and the voids analysis of the core specimens are presented hereafter.

Core Specimen Physical Tests. The cores from the pavements were evaluated for their compacted bulk specific gravity, total thickness after trimming, and the thickness of the surface course. The average values for each core sample are shown in Table XXI. In addition, the amount of material that was abraided off of the top 1/16 in. of the polished core specimen was evaluated by weighing each specimen and measuring its height before and after the polishing test was performed.

The information in Table XXI on core air voids and voids filled with asphalt was estimated from the AHD Marshall mix design report for each test pavement section. The information can be used only to compare values among the wheel path (WP) and the between wheel paths specimens of each core site.

Usually the core air void content was less for the WP specimens than for the BWP specimens. In general, the amount of material worn off the top of the specimens was less for the WP specimens than for the BWP specimens. This differential wear characteristic is attributed to the fact that most loose material on the pavement surface has been removed from the wheel paths by traffic.

It may be possible to obtain some index of the potential wearability of the different aggregate mixtures from this type of analysis, i.e., the samples that had more material abraided away are not as durable as (but possible are more skid resistant than) samples that retained most of their height and weight.

The average weight loss for the BWP specimens was about 13 percent, whereas the WP specimens only lost 7 percent; the height loss was 21 per-

Table XXI

Laboratory Test Results - Physical Properties and Wearing Loss of Core Specimens

BPT Test Site No.	Aggregate	Batch Plant[1] % AC	Bulk Sp.Gr.	Air Voids Estimated		Voids Filled		Thickness		Wear Loss - APD[4]			
				BWP[2]	WP[3]	BWP	WP	Total in.	Surf. in.	Δwt. (%) BWP	Δwt. (%) WP	Δht. (%) BWP	Δht. (%) WP
2	Johnson ls	5.2	2.212	8.6	8.7	56.9	56.6	2.5	1.2	18	10	32	17
4	Johnson ls	5.5	2.310	5.0	2.7	71.0	82.5	2.5	1.1	20	9	35	5
5	Johnson ls	5.4	2.358	2.0	1.1	86.5	91.6	2.5	1.4	12	11	18	16
7	Johnson ls	5.6	2.272	5.5	5.3	69.2	69.3	2.5	1.3	5	1	3	5
10	Valley Spr. ls	5.1	2.385	2.0	2.9	85.9	80.4	2.2	1.8	6	10	18	17
11	Valley Spr. ls	5.6	2.381	3.8	2.5	77.4	84.1	2.2	1.2	5	8	13	22
12	Valley Spr. ls	5.6	2.350	5.6	3.5	69.7	81.0	2.3	1.7	10	7	14	16
15	Valley Spr. ls	5.1	2.375	6.9	2.1	62.7	85.4	2.2	0.8	10	8	16	6
17	Cabot ss	5.3	2.324	3.4	3.8	78.2	76.4	1.9	1.0	20	10	30	20
19	Cabot ss	5.4	2.337	4.7	2.5	73.3	83.7	2.3	1.2	2	5	21	18
20	Cabot ss	5.3	2.278	7.8	5.0	60.0	70.7	2.6	1.3	52	18	51	26
21	Van Buren ss	5.3	2.293	6.7	3.5	63.7	77.7	2.5	1.3	2	2	2	2

Table XXI (continued)

Laboratory Test Results - Physical Properties and Wearing Loss of Core Specimens

BPT Test Site No.	Aggregate	Batch Plant[1] % AC	Bulk Sp.Gr.	Air Voids Estimated BWP[2]	Air Voids Estimated WP[3]	Voids Filled BWP	Voids Filled WP	Thickness Total in.	Thickness Surf. in.	Wear Loss - APD[4] Δwt. (%) BWP	Wear Loss - APD[4] Δwt. (%) WP	Wear Loss - APD[4] Δht. (%) BWP	Wear Loss - APD[4] Δht. (%) WP
22	Van Buren ss	5.6	2.345	2.8	1.5	81.9	89.6	2.5	1.4	14	7	14	7
24	Jenny Lind ss	5.4	2.354	3.3	1.5	79.2	89.6	2.6	1.4	10	8	19	13
25	Jenny Lind ss	5.3	2.357	2.9	1.7	81.0	87.8	2.1	1.3	5	3	14	18
27	Jenny Lind ss	5.6	2.345	2.9	2.5	81.8	83.6	2.5	1.5	1	1	16	13
29	Jenny Lind ss	5.4	2.365	0.9	0.7	93.4	94.9	2.5	1.2	7	5	2	18
32	Jenny Lind ss	5.2	2.294	3.8	4.1	75.5	74.2	1.6	0.9	7	6	19	22
33	Texarkana gvl	5.3	2.320	5.0	4.3	70.7	74.0	2.3	1.1	32	16	40	14
35	Texarkana gvl	5.2	2.385	3.1	2.4	79.4	84.3	2.5	1.2	20	4	38	46
38	Texarkana gvl	5.4	2.378	2.8	3.0	81.8	81.9	2.5	1.3	27	17	37	22
40	Murfreesboro gvl	5.4	2.387	0.5	0.3	96.2	97.7	2.4	1.3	9	6	14	19
41	Murfreesboro gvl	5.6	2.234	6.3	5.9	66.0	67.8	2.2	1.5	5	1	16	3
44	Murfreesboro gvl	5.7	2.313	2.8	2.0	82.4	86.5	2.5	1.5	1	1	6	5

Table XXI (concluded)

Laboratory Test Results - Physical Properties and Wearing Loss of Core Specimens

BPT Test Site No.	Aggregate	Batch Plant[1] % AC	Bulk Sp.Gr.	Air Voids Estimated BWP[2]	Air Voids Estimated WP[3]	Voids Filled BWP	Voids Filled WP	Thickness Total in.	Thickness Surf. in.	Wear Loss - APD[4] Δwt. (%) BWP	Wear Loss - APD[4] Δwt. (%) WP	Wear Loss - APD[4] Δht. (%) BWP	Wear Loss - APD[4] Δht. (%) WP
46	Murfreesboro gvl	5.7	2.309	4.2	3.2	74.9	80.5	2.0	1.1	-	1	48	11
49	Big Rock ns	5.8	2.276	6.5	4.7	66.3	73.8	1.1	0.4	17	8	34	26
51	Big Rock ns	6.1	2.332	2.4	2.2	85.6	86.3	1.8	0.6	8	7	-	17
52	Big Rock ns	5.9	2.332	2.9	3.0	82.2	81.8	1.9	0.8	9	7	19	11
54	Big Rock ns	5.8	2.194	10.7	7.8	53.3	62.1	1.9	0.6	22	14	22	10
56	Malvern nov	5.8	2.278	6.8	3.9	65.3	78.8	1.4	0.8	14	11	18	6
57	Malvern nov	5.9	2.196	9.3	6.8	57.2	62.5	1.3	0.8	16	6	6	10
60	England syn	-	2.126	-	-	-	-	1.2	0.5	3	3	38	14

[1]From last report of inspection at the Asphalt Plant

[2]Between Wheel Path

[3]Wheel Path (average of IWP and OWP)

[4]Wear loss based on top 1/16 inch of specimen

cent and 15 percent, respectively. No valid comparison of wear among the different types of aggregates is possible because of their different ages, weather conditions, asphalt content and composition, and fine aggregate variations.

Film Stripping Test Results. All of the aggregate samples were tested for film stripping by the SIS and DIS methods of test. The results of the tests are shown in Table XXII. No stripping was observed for the static immersion stripping test at 77 F. When the test temperature was raised to 140 F for 18 hours, extensive stripping was evident on some aggregates. By visual observation, the retained coatings ranged from 90 percent for the Valley Springs and Rocky Point limestones to 20 percent for the Russellville and Bald Knob sandstones.

A lesser amount of stripping was observed from the dynamic immersion test at 77 F. The values of retained coatings ranged from 60 percent for the Texarkana gravel and Jenny Lind sandstone, to 90 percent for the Rocky Point and West Fork limestones, Murfreesboro and Hampton gravels, and the Bald Knob sandstone.

The amount of stripping as measured by the surface reaction test is also shown in Table XXII. The stripped samples from the SIS (140 F) and DIS tests were measured quantitatively for the amount of surface area exposed from these stripping tests by the SRT.

The surface reation test average value from the SIS (140 F) test was 56 percent retained coating, whereas the DIS tests gave a 70 percent retained costing. These test results reflect the different test methods used to induce the stripping effects of water. In the DIS test, the coated aggregate particles are abraided against one another and the test jar as they revolve back and forth in the water. This action is supposed

Table XXII

Laboratory Test Results - Aggregate Shape and Film Stripping

No.	Aggregate	Static Immersion Ret. Coating - %		Dynamic Immersion Ret. Coating - %	Surface Reaction Ret. Coating - %		Particle Shape	
		77F	140F	77F	SIS 140F	DIS 77F	Flakiness Index	Elongation Index
1	Johnson ls	100	75	80	90	98	25	14
2	Valley Springs ls	100	90	85	96	99	25	13
3	Cabot ss	100	40	80	44	49	42	30
4	Van Buren ss	100	50	75	26	37	33	18
5	Jenny Lind ss	100	60	60	46	21	37	18
6	Texarkana gvl	100	70	60	47	-	28	24
7	Murfreesboro gvl	100	30	90	57	81	25	25
8	Big Rock ns	100	60	80	49	79	38	18
9	Malvern nov	100	70	75	48	84	31	24
10	England syn	100	40	85	38	92	20	6
11	Twin Lakes ls	100	70	80	62	72	24	12
12	Kentucky ls	100	80	85	95	99	24	23
13	Rocky Point ls	100	90	90	93	98	31	24
14	Black Rock ls	100	30	85	54	46	39	29
15	West Fork ls	100	70	90	94	98	21	10
16	Russellville ss	100	20	85	52	32	36	9
17	Bald Knob ss	100	20	90	18	14	33	33
18	Hampton gvl	100	70	90	80	72	42	12

to simulate the effects of traffic on the wet pavement surface. The SIS (140 F) test is supposed to simulate the effects of water being heated on the pavement surface during the summer months. As shown in the literature review, there is no agreement among researchers about which method is more accurate.

On the basis of the surface reaction test data from the SIS (140 F) film stripping tests, the aggregates ranked from good to poor are: limestone, gravel, syenite, novaculite, synthetic, and sandstone. On the basis of the DIS film stripping test, the aggregates ranked from good to poor are: limestone, synthetic, novaculite, gravel, syenite, and sandstone. An average of the results of the two stripping tests would rank the aggregates, with the average percentage retained coatings, as: limestone (85), novaculite (66), synthetic (65), syenite (64), gravel (64), and sandstone (34).

The Bald Knob sandstone indicated an average retained coating of 16 percent, the lowest value, and the Black Rock limestone had only a 50 percent retained coating; the highest retained coating was for the Valley Springs limestone, 98 percent.

These results are in general agreement with the information reported in the literature review, as shown in Table IX. In that report, the lowest retained coating values are for the Broken Bow (gravel) at 54 percent and the Keota (sandstone) at 56 percent, and the highest retained coating value is for the Stringtown (limestone) at 93 percent.

One of the more significant correlations was established between film stripping and insoluble residue of the aggregate. The curve relating insoluble residue and SRT (average) retained coating in percent is shown in Figure 50. The equation of the best fitted line is: ln insoluble

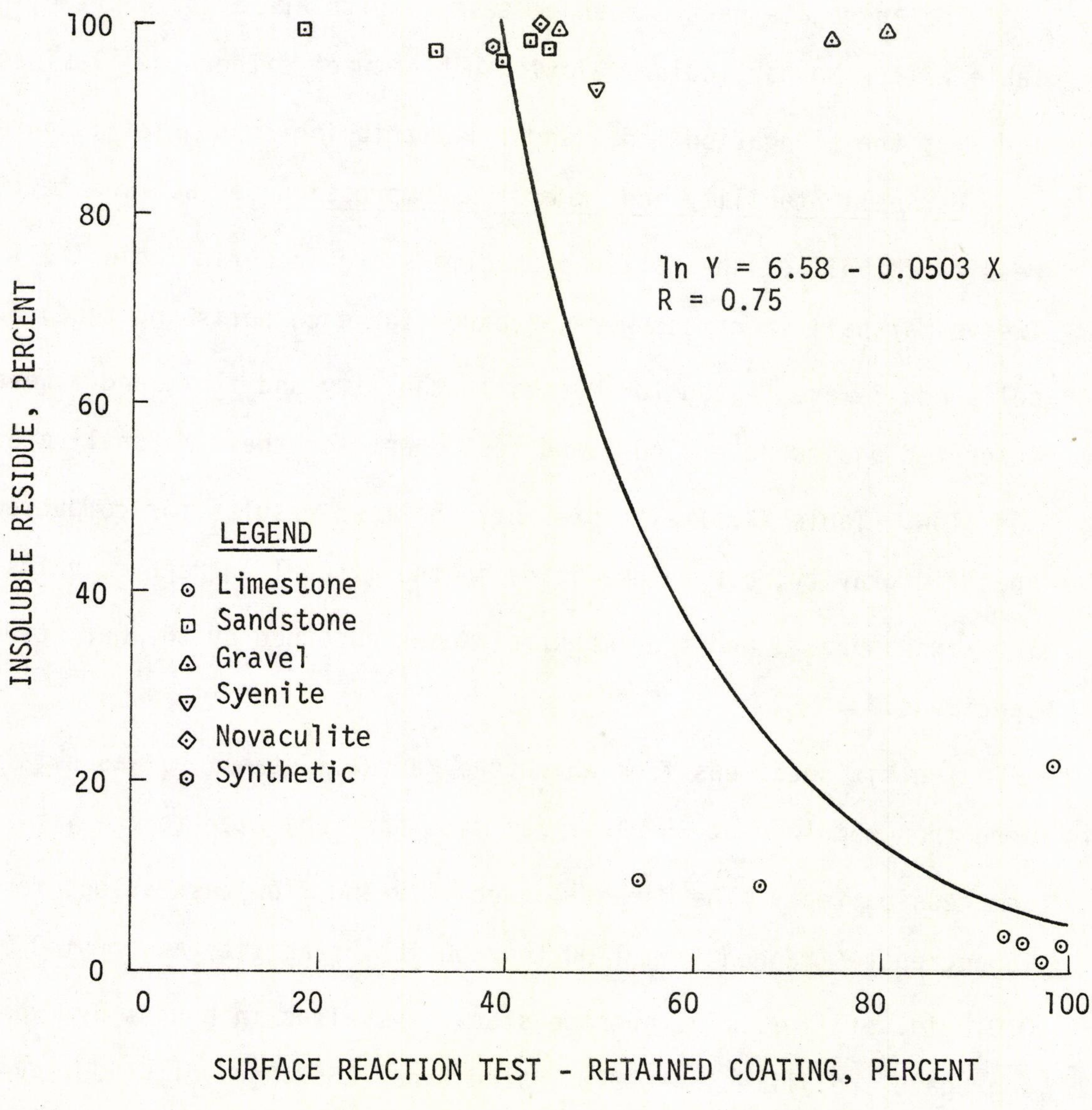


Figure 50. Relationship Between Insoluble Residue and SRT Retained Coating

residue = 6.58 - 0.0503 retained coating; with an R value of 0.75. The type of aggregate sample was coded in the plotting of the data for Figure 50 to indicate the relative position of each type of aggregate in relation to its stripping tendency.

The aggregate particle shape test results are also reported in Table XXII. No particular relationship between either the flakiness index or the elongation index and film stripping tendencies is noted.

Marshall Stability and Immersion-Compression Tests. The test results for all of the polished specimens are shown in Table XXIII. Twelve Marshall specimens were prepared for each polishing test, 6 were polished, 3 were tested for Marshall stability and flow, and 3 were vacuum saturated and cured at 140 F and tested wet for their Marshall stability and flow. Table XXIII also presents the test results for compacted bulk specific gravity, air voids, voids in the mineral aggregate, voids filled with asphalt, and the amount of water absorbed by the wet stability specimens.

The six specimens from which the Marshall stability was obtained were then remolded at 35 blows per side and subjected to the immersion-compression test. The Marshall stability and flow test values for the 18 aggregates ranged from 1800 to 2800 lbs. stability and from 0.010 to 0.018 in. of flow. The average stability values in pounds by type of aggregate are: limestone, 2180; sandstone, 2550; gravel, 1900; syenite, 2065; novaculite, 2005; and synthetic, 2255.

The immersion-compression test results for the various blended aggregate mixtures are also given in Table XXIII. The I-C test results for the eight blended mixtures which included limestone aggregate show an average retained strength of 91 percent. The average I-C results for

Table XXIII

Laboratory Test Results - Marshall Stability and Immersion-Compression

No.	Aggregate[1]	Blows per Side	Water Abs. %	AC %	Bulk Sp.Gr.	AV %	VMA %	Voids Filled (V_F) %	Marshall Test Stab. Dry (lb)	Marshall Test Flow 0.01 in.	I-C Ret. Str. %
1	Johnson ls	50	1.4	5.3	2.362	3.2	15.6	78.6	2050	13	95
1R	Johnson ls	35	1.9	5.3	2.334	4.4	16.6	73.3	1695	14	91
2	Valley Spgs. ls	50	0.3	5.3	2.443	0.8	13.5	93.9	2185	17	99
2R	Valley Spgs. ls	35	0.9	5.3	2.411	2.1	14.6	85.9	2045	18	93
3	Cabot ss	50	0.9	5.4	2.388	1.3	14.0	90.5	2405	17	100
3R	Cabot ss	35	1.0	5.4	2.371	2.0	14.6	85.8	2150	16	104
4	Van Buren ss	50	1.3	5.3	2.369	2.5	14.8	83.2	2485	14	90
4R	Van Buren ss	35	1.1	5.3	2.361	2.9	15.2	81.1	2435	14	102
5	Jenny Lind ss	50	2.2	5.2	2.325	4.2	16.0	74.1	2640	11	84
5R	Jenny Lind ss	35	2.6	5.2	2.309	4.8	16.6	71.0	2520	13	82
6	Texarkana gvl	50	1.3	5.7	2.357	2.8	15.9	82.7	1755	15	88
6R	Texarkana gvl	35	1.1	5.7	2.361	2.7	15.8	83.3	1750	14	92
7	Murfreesboro gvl	50	1.8	5.6	2.326	2.7	15.4	82.9	2105	14	85
7R	Murfreesboro gvl	35	1.9	5.6	2.313	3.1	15.8	80.3	1705	14	90
8	Big Rock ns	50	1.9	5.2	2.348	4.0	16.0	74.8	1975	11	85
8R	Big Rock ns	35	1.7	5.2	2.334	4.4	16.4	73.2	2065	15	83
9	Malvern nov	50	1.4	5.5	2.327	3.7	16.2	77.1	2005	15	92
9R	Malvern nov	35	1.7	5.5	2.307	4.2	16.7	74.5	1985	15	92

Table XXIII (continued)

Laboratory Test Results - Marshall Stability and Immersion-Compression

No.	Aggregate[1]	Blows per Side	Water Abs. %	AC %	Bulk Sp.Gr.	AV %	VMA %	Voids Filled (V_F) %	Marshall Test		I-C Ret. Str. %
									Stab. Dry (lb)	Flow 0.01 in.	
10	England syn	50	4.0	8.8	1.803	4.6	20.2	77.4	2255	14	88
10R	England syn	35	5.9	8.8	1.794	5.4	20.9	74.3	2370	14	85
11	Twin Lakes ls	50	0.9	5.4	2.430	1.9	87.0	14.8	2565	17	99
11R	Twin Lakes ls	35	1.4	5.4	2.407	2.9	15.8	81.3	2330	15	90
12	Kentucky ls	50	0.5	5.1	2.448	1.7	13.9	88.1	2000	16	104
12R	Kentucky ls	35	0.6	5.1	2.435	2.1	14.3	85.4	2075	19	100
13	Rocky Point ls	50	0.5	5.2	2.425	1.0	13.4	92.7	2240	16	92
13R	Rocky Point ls	35	0.6	5.2	2.410	1.6	13.9	88.5	2010	18	100
14	Black Rock ls	50	0.6	5.3	2.471	2.4	15.3	84.1	2065	18	89
14R	Black Rock ls	35	0.5	5.3	2.475	2.3	15.1	85.1	1920	21	112
15	West Fork ls	50	0.6	5.5	2.406	2.2	15.2	85.6	2170	10	110
15R	West Fork ls	35	0.9	5.5	2.393	2.8	15.6	82.4	2325	16	94
16	Russellville ss	50	0.9	5.2	2.396	2.5	14.7	83.2	2415	15	95
16R	Russellville ss	35	1.0	5.2	2.382	2.8	15.0	81.4	2370	15	90
17	Bald Knob ss	50	0.7	5.3	2.376	2.5	14.8	83.4	2775	13	91
17R	Bald Knob ss	35	1.3	5.3	2.345	3.8	15.8	76.4	2430	15	77
18	Hampton gvl	50	0.8	5.1	2.380	2.4	14.3	83.2	1855	15	96
18R	Hampton gvl	35	1.0	5.1	2.364	3.0	14.8	80.1	1985	13	89

Table XXIII (continued)

Laboratory Test Results - Marshall Stability and Immersion-Compression

No.	Aggregate[1]	Blows per Side	Water Abs. %	AC %	Bulk Sp.Gr.	AV %	VMA %	Voids Filled (V_F) %	Marshall Test Stab. Dry (lb)	Marshall Test Flow 0.01 in.	I-C Ret. Str. %
19	J ls + VB ss	50	1.4	5.3	2.367	2.8	15.1	81.4	2285	14	87
19R	J ls + VB ss	35	2.2	5.3	2.329	4.4	16.5	73.4	1840	13	92
20	J ls + M nov	50	1.2	5.4	2.365	2.5	15.1	83.2	2060	16	88
20R	J ls + M nov	35	1.5	5.4	2.335	3.8	16.1	76.8	1850	15	97
21	J ls + E syn	50	4.2	6.3	2.338	7.0	19.7	64.4	2255	13	77
21R	J ls + E syn	35	6.1	6.3	2.012	8.6	21.0	59.2	1980	13	86
22	VS ls + Cabot gvl	50	-	5.4	2.404	1.6	14.4	88.8	2455	17	-
22R	VS ls + Cabot gvl	50	-	5.4	2.396	2.0	14.7	86.6	2440	18	-
23	VS ls + Mur gvl	50	0.7	5.4	2.397	1.9	14.9	87.3	2160	16	92
23R	VS ls + Mur gvl	35	1.0	5.4	2.370	3.0	15.6	80.7	2130	17	85
24	VS ls + 30%[2] M nov	50	-	5.4	2.391	3.6	16.3	77.9	2285	13	-
24R	VS ls + 30% M nov	35	-	5.4	2.392	3.6	16.2	77.8	2460	13	96
25	VS ls + 60% M nov	50	-	5.4	2.368	4.0	16.5	75.8	2450	12	-
25R	VS ls + 60% M nov	35	-	5.4	2.358	4.4	16.8	74.4	2090	13	98
26	VS ls + 30% E syn	50	-	6.5	2.241	1.5	15.8	90.5	2000	14	-
26R	VS ls + 30% E syn	35	-	6.5	2.242	1.5	15.8	90.5	2170	16	95
27	VS ls + 60% E syn	50	-	7.5	2.056	1.6	16.1	90.1	2355	15	-
27R	VS ls + 60% E syn	35	-	7.5	2.045	1.8	16.8	89.4	2290	15	98

Table XXIII (concluded)

Laboratory Test Results - Marshall Stability and Immersion-Compression

No.	Aggregate[1]	Blows per Side	Water Abs. %	AC %	Bulk Sp.Gr.	AV %	VMA %	Voids Filled (V_F) %	Marshall Test Stab. Dry (lb)	Marshall Test Flow 0.01 in.	I-C Ret. Str. %
28	Cabot ss + BR ns	50	-	5.4	2.384	1.7	14.3	88.2	2085	14	-
28R	Cabot ss + BR ns	50	-	5.4	2.381	1.5	14.2	89.2	2220	16	-
29	VB ss + Tex gvl	50	1.0	5.5	2.376	2.0	14.9	86.2	1855	15	95
29R	VB ss + Tex gvl	35	1.4	5.5	2.348	3.2	15.9	79.9	1990	17	95
30	JL ss + Mur gvl	50	0.9	5.4	2.355	2.1	14.6	85.6	2145	13	104
30R	JL ss + Mur gvl	35	1.9	5.4	2.309	4.0	16.2	76.1	1825	14	85
31	Tex gvl + E syn	50	3.3	6.5	2.072	3.5	16.8	79.0	1965	14	90
31R	Tex gvl + E syn	35	4.3	6.5	2.032	5.4	18.3	71.0	1250	15	133
32	Mur gvl + BR ns	50	-	5.4	2.370	1.6	14.2	88.7	1870	14	-
32R	Mur gvl + BR ns	50	-	5.4	2.358	2.1	14.6	85.8	2050	16	-
33	BR ns + M nov	50	1.1	5.4	2.350	3.0	15.4	80.8	1790	14	86
33R	BR ns + M nov	35	1.9	5.4	2.311	4.2	16.5	74.7	1325	17	92

[1]Aggregate abbreviations used in blends are: J = Johnson, VB = Van Buren, M = Malvern, E = England, VS = Valley Springs, Mur = Murfreesboro, BR = Big Rock, Tex = Texarkana and JL = Jenny Lind

[2]Percent of total coarse aggregate in mix i.e. VS = 70% and M = 30%, etc.

the two blended mixtures of sandstone aggregate indicate a retained strength of 95 percent; the one mixture of gravel aggregate has an average of 112 percent retained strenght. The syenite and novaculite blend has an I-C retained strength of 89 percent.

When the results of the I-C test were correlated with the several other variables the most significant relationship obtained was with the air voids in the mixture. The relationship between air voids and retained strength for the 35 blow specimens is shown in Figure 51. The equation of the best fitted line is: log air voids = 5.7 - 2.65 log retained strength; the coefficient of correlation is 0.72.

At this point, it should be emphasized that the fine aggregate fraction of all of the asphalt mixtures, 47 percent of the total aggregate, consisted of limestone screening and some fine river sand. The differential effect of this calcium carbonate fine aggregate on the dry stability and wet stability of each of the individual mixtures was not evaluated in this investigation. The literature review (98) indicated that immersion-compression retained strength of limestone mixtures often is greater than 100 percent, i.e., the wet stability of the specimen is higher than the dry stability of the specimen.

The average I-C retained strength for the 50 blow Marshall mixtures, by type of aggregate is: limestone, 98; sandstone, 92; gravel, 90; syenite, 83; novaculite, 92; and synthetic, 85.

The relative stripping resistance of the various aggregates as determined from the immersion-compression tests, with their average retained strengths, are: limestone, 98 percent; novaculite, 92 percent; standstone, 92 percent; gravel, 90 percent; synthetic, 86 percent; and syenite, 82 percent.

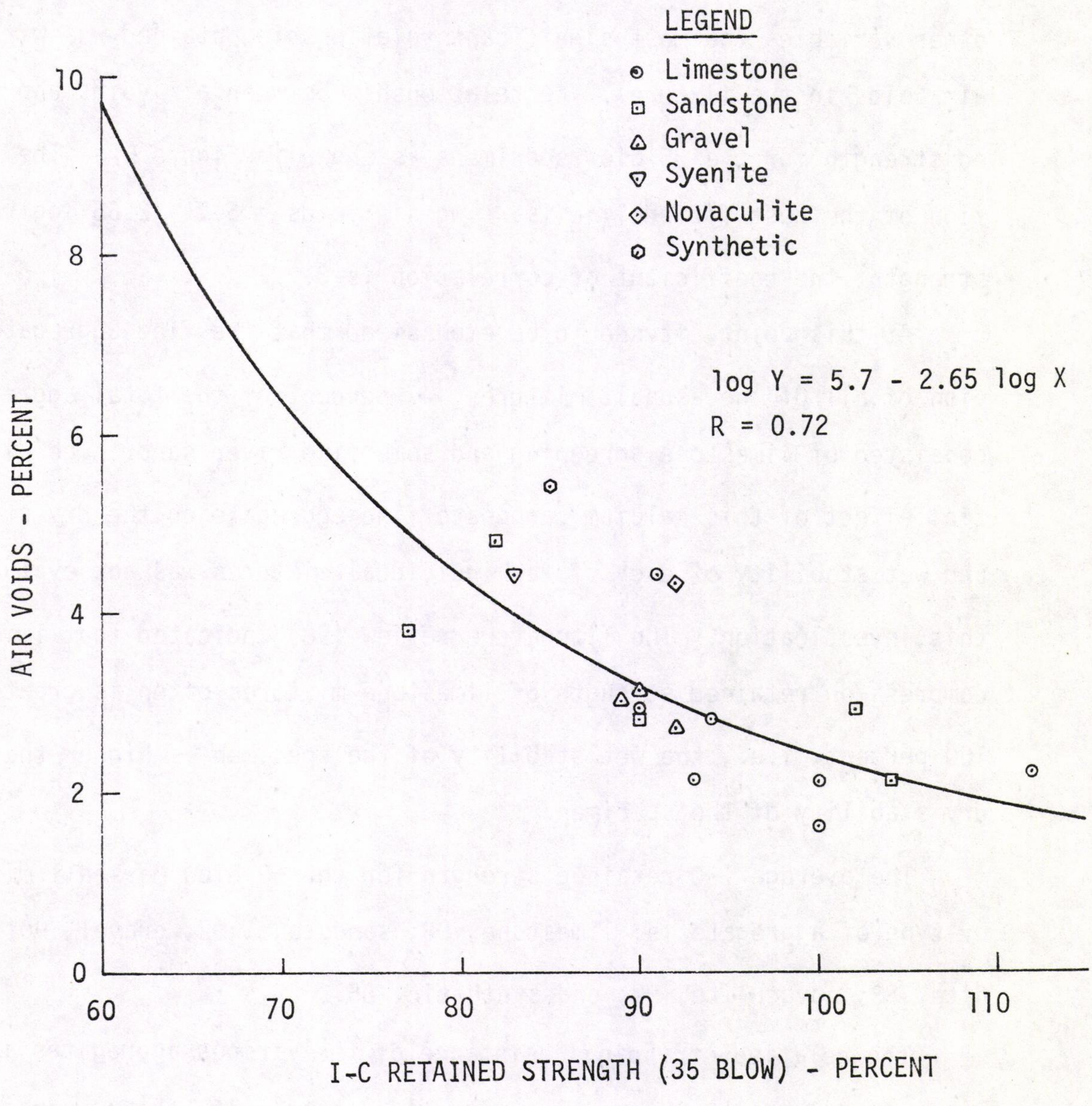


Figure 51. Relationship Between Air Voids and Immersion-Compression Retained Strength

Summary

To evaluate the relationships, if any, among the results of the various field and laboratory tests, the data obtained from all the tests were analyzed statistically by a computerized curvilinear regression program utilizing the least squares method. The results of this type of study yield the "best fit" curve relation between two variables.

The regression analysis program used compares the data by four curve fitting methods: direct or indirect linear, logarithm versus linear, logarithm versus logarithm, and natural logarithm versus linear. The results from the best fit curve evaluation are reported in the form of an equation and a coefficient of correlation detailing the accuracy of the test. The correlation coefficient is a measure of how close the resulting curve is to a perfect relationship, i.e., the data points all being directly on the regressed curve. A perfect relationship would result in a correlation coefficient of one; therefore, the closer to one, the higher the accuracy of the relationship.

Significant Correlations of Test Data

Justifiable correlations usually are associated with coefficients of the magnitude of 0.65 and 0.70 or greater. Also, in a regression analysis, the number of data values entered into the variable breakdown is proportional to the accuracy of the final solution. In general, 10 points is considered the minimum number of values that will justify a representative sample of a variable.

Some individual types of aggregate were believed to have unique properties that might cause them to correlate differently from aggregates as a whole; they were analyzed separately and compared with the other types of aggregates. Because of the limited data associated with syenite and novaculite, the two were sometimes combined in the analysis. The individual significant correlations and the best fitted equation relating each variable are shown in Table XXIV.

Traffic Correlations. For the purpose of computing the amount of polishing the pavements had already achieved, and in attempt to correlate the APD and actual traffic polishing, traffic data were accumulated for each of the study roads.

Average annual 24 hour traffic volume (AADT) for each year the pavement was in use was taken from yearly traffic volume maps prepared by the Arkansas Highway Department Division of Planning and Research. Total traffic volume was computed from the day the study sections were opened until April 1, 1977, which is approximately the date of the core removals (February and March), and the date of the final 1977 field testing (May and June).

The daily traffic data from the traffic volume maps contained two-direction total vehicle loading for each road. The assumption was made that half the traffic flowed in each direction, and that each vehicle (on the AADT maps) consisted of two axles. Therefore, the values selected from the traffic volume maps were read as "wheel passes." The resulting data are shown in Table XV.

The total estimated number of wheel passes was correlated with SN_{40} values, BPN values, and pavement texture depths. The results of these correlations are shown in Table XXIV as number 1 through number 14. The highest correlation value obtained with the SN_{40} data was $R = 0.61$ for the 15 BPT sites on gravel pavements. The relationship between 1977 SN_{40} and total estimated wheel passes had an $R = 0.32$; this best fitted line is shown in Figure 34. When the average BPN

Table XXIV

Field and Laboratory Correlations

No.	Correlation (X vs Y)	Equation	Corr. Coef.	No. of Points
-	1977 Total Traffic vs SN_{40} @ BPT Sites			
1	- 1976 SN_{40} (All)	Log Y = 1.92 - 0.0499 Log X	0.23	60
2	- 1977 SN_{40} (All)	Ln Y = 3.81 - 0.000016 X	0.32	60
3	Limestone	Y = 39.2 - 7.72 Log X	0.59	15
4	Sandstone	Y = 51.9 - 4.93 Log X	0.20	17
5	Gravel	Y = 50.4 - 11.1 Log X	0.61	15
6	Syenite	Y = 38.0 - 0.763 X	0.56	8
7	Novaculite	Ln Y = 3.84 + 0.00868 X	0.60	4
-	1977 Total Traffic vs BPN			
8	- 1976 IWP BPN	Ln Y = 4.06 + 0.00000177 X	0.08	60
9	- 1976 OWP BPN	Ln Y = 4.06 + 0.00000154 X	0.07	60
10	- 1977 IWP BPN	Y = 70.5 - 3.02 Log X	0.22	60
11	- 1977 OWP BPN	Y = 74.9 - 4.50 Log X	0.30	60
12	- 1977 Average BPN	Y = 78.0 - 5.06 Log X	0.33	60
-	1977 Total Traffic vs Field Texture			
13	- IWP Texture	Log Y = -0.0284 - 0.0696 Log X	0.16	60
14	- OWP Texture	Y = 1.02 - 0.140 Log X	0.29	60

Table XXIV (continued)

Field and Laboratory Correlations

No.	Correlation (X vs Y)	Equation	Corr. Coef.	No. of Points
-	1977 AADT vs SN_{40} @ BPT Site			
15	- 1976 SN_{40} (All)	Log Y = 1.84 - 0.0297 Log X	0.11	60
16	- 1977 SN_{40} (All)	Ln Y = 3.79 - 0.0000172 X	0.19	60
17	Limestone	Y = 72.1 - 10.9 Log X	0.47	15
18	Sandstone	Y = 93.6 - 13.0 Log X	0.49	17
19	Gravel	Y = 85.0 - 11.6 Log X	0.41	15
-	1977 AADT vs BPN			
20	- 1976 IWP BPN	Ln Y = 4.04 + 0.00000869 X	0.22	60
21	- 1976 OWP BPN	Ln Y = 4.05 + 0.0000059 X	0.15	60
22	- 1977 IWP BPN	Y = 65.7 - 1.36 Log X	0.09	60
23	- 1977 OWP BPN	Y = 62.1 - 0.000552 X	0.24	60
-	1977 AADT vs Field Texture			
24	- IWP Field Texture	Ln Y = 0.504 - 0.0000419 X	0.26	60
25	- OWP Field Texture	Log Y = 0.360 - 0.198 X	0.36	60
-	1977 AADT (Td < 0.51 mm) vs			
26	- 1977 SN_{40}	Ln Y = 3.67 - 0.00000586 X	0.07	33
27	- 1977 IWP BPN	Ln Y = 4.07 + 0.00000475 X	0.15	33

Table XXIV (continued)

Field and Laboratory Correlations

No.	Correlation (X vs Y)	Equation	Corr. Coef.	No. of Points
28	- 1977 OWP BPN	Ln Y = 4.08 - 0.00000320 X	0.09	33
-	1977 AADT (Td > 0.51 mm) vs			
29	- 1977 SN_{40}	Y = 67.9 - 6.28 Log X	0.25	26
30	- 1977 IWP BPN	Ln Y = 4.17 - 0.0000156 X	0.37	26
31	- 1977 OWP BPN	Y = 88.0 - 7.81 Log X	0.40	26
32	1977 SN_{40} @ BPT Site vs 1976 SN_{40} @ BPT Site	Y = 17.0 + 0.903 X	0.78	60
-	1977 SN_{40} vs 1977 BPN			
33	- IWP BPN @ Site	Log Y = 1.33 + 0.283 Log X	0.72	60
34	Limestone	Y = 36.2 + 0.668 X	0.83	15
35	Sandstone	Log Y = 1.51 + 0.182 Log X	0.65	17
36	Gravel	Log Y = 1.00 + 0.474 Log X	0.87	15
37	Syenite and Novaculite	Log Y = 1.58 + 0.109 Log X	0.55	12
38	- OWP BPN @ Site	Log Y = 1.21 + 0.348 Log X	0.78	60
39	Limestone	Y = -48.0 + 69.0 Log X	0.89	15
40	Sandstone	Log Y = 1.34 + 0.277 Log X	0.72	17
41	Gravel	Ln Y = 3.47 + 0.0140 X	0.86	15
42	Syenite and Novaculite	Log Y = 1.65 + 0.0633 Log X	0.31	12

Table XXIV (concluded)

Field and Laboratory Correlations

No.	Correlation (X vs Y)	Equation	Corr. Coef.	No. of Points
-	1977 SN_{40} vs Field Texture			
43	- IWP Texture	Y = -0.821 + 0.853 Log X	0.40	60
44	- OWP Texture	Log Y = 1.21 + 0.348 Log X	0.78	60
45	1977 IWP BPN vs 1976 IWP BPN	Log Y = 0.459 + 0.734 X	0.63	60
-	1977 IWP BPN vs			
46	- IWP Texture	Y = 0.0424 + 0.00850 X	0.22	60
47	- OWP Texture	Y = 0.0675 + 0.00732 X	0.20	60
-	Surface Reaction Test vs			
48	- I-C 50 Blow	Y = 84.1 + 0.41 X	0.52	18
49	- I-C 35 Blow	Log Y = 1.84 + 0.0687 Log X	0.34	18
50	- Elongation Index	Y = 46.5 - 15.6 Log X	0.39	18
51	- Flakiness Index	Ln Y = 3.64 - 0.00366 X	0.39	18
52	- Insoluble Residue	Ln Y = 6.58 - 0.0503 X	0.75	18
53	I-C 50 Blow vs Air Voids	Y = 41.6 - 19.8 Log X	0.61	18
54	I-C 50 Blow vs Flakiness Index	Ln Y = 4.44 - 0.0110 X	0.33	18
55	I-C 35 Blow vs Air Voids	Log Y = 5.70 - 2.65 Log X	0.72	18
56	I-C 35 Blow vs Elongation Index	Ln Y = 1.36 + 0.0162 X	0.30	18

test results were correlated with wheel passes, the best fitted line indicated an R = 0.33 value as shown for correlation number 12. The outer wheel path texture depth correlation with the traffic showed an R = 0.29 value, and is correlation number 14.

The AADT counts for 1977 were used in correlations 15 through 31 as shown in Table XXIV. These daily traffic numbers were regressed against SN_{40} values, BPN values, pavement texture depths, and again with SN_{40} values divided for pavements having a Td less than 0.51 mm and for those with a Td greater than 0.51 mm. The best single correlation, number 18, related AADT values to SN_{40} on sandstone pavements with an R = 0.36 value. In the trial correlations 26 through 31, the AADT values were regressed against the SN_{40} values for low Td's and high Td's. The correlations are low, but the R values for pavements having a Td greater than 0.51 mm indicate a much better correlation (R = 0.25 to 0.40) than do those for pavements with a Td less than 0.51 mm (R = 0.07 to 0.15).

<u>Skid Trailer and British Portable Tester Correlations</u>. The correlations between SN_{40} values and BPN values ranged from an R = 0.31 to an R = 0.89. The best fitted equations were established for the various types of aggregates, and related both the IWP-BPN and OWP-BPN values versus SN_{40} data. The combination of IWP and OWP British Pendulum Number versus SN_{40} values was used to obtain the best fitted equation curve shown in Figure 35.

Two correlations of particular interest are those relating 1977 SN_{40} values and 1976 SN_{40} values (number 32) and relating 1977 BPN values and 1976 BPN values (number 47). The correlation between the two different years' SN_{40} tests is 1976 SN_{40} = 17.0 + 0.903 1976 SN_{40}, with an R = 0.78 value. However the 1977 BPN and the 1976 BPN test values (number 45) for the inner wheel path tests only indicated a correlation of R = 0.63 value. This BPN correlation is lower than expected; these yearly BPN correlations would be expected to be increased by using the additional 60 test values for the outer wheel path values in the regression analysis.

Stripping Correlations. The correlations between stripping test results are also shown in Table XXIV, numbers 48 through 56. The best correlation for the film stripping tests was between the SRT percent retained coating and insoluble residue content of the aggregate. The graph of this relationship is Figure 50.

The correlations between the SRT test results and the I-C test results (numbers 48 and 49) indicate a better correlation with the 50 blow Marshall specimen than with the 35 blow Marshall specimens. This best value, R = 0.52, has very little significance, however. Lower correlations were obtained between the SRT test data and the aggregate flakiness index or the elongation index test results.

The regression analysis for the I-C test results versus aggregate shape, as measured from the flakiness index and the elongation index, is low and is only presented for information. The correlation between I-C retained strengths and percent air voids indicated an R = 0.72 value for the 35 blow Marshall specimen. This relationship is reported in Figure 51.

Field Tests

The results of the field tests are shown in Table XVI and discussed with Figures 27 through 35. The data include SN_{40} values, BPN values, and pavement texture depths at the BPT sites. A good relationship between the 1977 SN_{40} values and BPN values was established.

Laboratory Tests

The correlation between the Texas Polish Value and the Arkansas Polish Value for mineral aggregate was established. The polishing test results are reported for the 51 aggregate and asphalt mixtures evaluated. The polishing curves for each sample are presented in Figures 36 through 49; in Appendix E, Figures E1 through E8; and in Appendix F, Figures F1 through F15.

The relative polishing characteristics of each aggregate, asphalt mixture, and core specimen were determined. A good relationship between BPN values and APV was established.

Each aggregate was evaluated as to its film stripping characteristics and physical properties. The relative stripping tendency of each asphalt mixture was determined from the immersion-compression test. A fairly good relationship is reported between the amount of insoluble residue in the aggregate and the amount of film stripping that was measured. A definite relationship was observed between the air void content and the immersion-compression retained strengths of Marshall specimens.

Chapter VI

CONCLUSIONS AND RECOMMENDATIONS

Within the limitations of the test procedures and for the range of materials and conditions utilized in this investigation, the following conclusions are made.

1. The Arkansas Accelerated Polishing Device is an excellent apparatus for polishing test specimens to their minimum friction values. Therefore, the device and associated test procedures are a valuable aid in predicting the wearing properties of aggregates and asphalt mixtures for use in highway pavements.

2. The fricitonal resistance of the pavement sections tested, as measured by the Arkansas Highway Department skid trailers, showed a considerable difference in SN_{40} values between the June 1976 tests and the June 1977 tests. The British Portable Tester results did not indicate such a wide difference. The true frictional resistance of the pavements studied therefore was not determined.

3. The data obtained from this investigation provide sufficient information to study and evaluate adequately the polishing characteristics of aggregates, asphalt mixtures, and pavement cores. The Arkansas Polish Value obtained in the laboratory can be used to establish the minimum skid number that will be provided by an asphalt mixture. Once the minimum desired skid value has been determined, specifications can be established for the polish values of asphalt mixtures or mineral aggregates for use in Arkansas pavements.

4. A good relationship is indicated between the polish value of the aggregate as measured by the Texas Highway Department, after polish

with the British Wheel, and the Arkansas Polish Value as measured by the BPT after polishing on the Arkansas Accelerated Polishing Device.

5. A good correlation was obtained between the skid trailer test values (SN_{40}) and the British Pendulum Number as measured with the British Portable Tester. Also, a good correlation was obtained between the BPN values (field) and Arkansas Polish Value (laboratory).

6. No significant relationship was observed between the skid trailer test results and the estimated total number of wheel passes over the pavement. Also, there was no significant correlation between SN_{40} values and average annual daily traffic.

7. The polish tests indicated that when sandstone, novaculite, or synthetic aggregate are used in asphalt mixtures, their APV after polishing is less than the APV obtained for the pure aggregate. On the contrary, when limestone, gravel, or syenite aggregate are placed in an asphalt mixture, the resulting APV value is higher than that obtained for the pure aggregate. The polish values obtained from tests on aggregate specimens do not indicate the actual polish resistance that is obtained when the materials are placed into a pavement.

8. Laboratory polish tests on pavement cores indicated a higher resistance to polish for most cores than was indicated from their Marshall polish test. The difference in these polish values is attributed to the fine aggregate used in the actual paving mixture and the different surface texture obtained from the core specimen and a Marshall specimen.

9. The overall test results indicated that sandstone and gravel mixtures are more polish resistant than syenite and limestone mixtures, whereas the novaculite and synthetic aggregates are of intermediate

resistance.

10. The film stripping tests, as evaluated by the surface reaction test, yield quantitative values that when correlated with field observations will be a valuable means of pre-evaluation of aggregates proposed for use in surface treatments.

11. The air void content of the compacted asphalt mixture greatly influences the results of the immersion-compression test. Specimens with greater than 5 percent air voids would have a retained strength of about 75 percent or less.

12. The stripping tests indicated that the Bald Knob sandstone is most susceptible to stripping. The Black Rock limestone indicated a tendency to strip in the SIS and DIS tests; however, in the Marshall mixture a satisfactory retained strength was obtained in the immersion-compression test.

On the basis of the results of this investigation, the following recommendations are presented.

1. The Bald Knob sandstone and Black Rock limestone aggregates should be investigated further in regard to their stripping tendencies.

2. Before the results of this investigation can be put into practical use, the relationship between traffic (total number of wheel passes or AADT) and the number of revolutions of the Arkansas Accelerated Polishing Device must be evaluated. The relationship between the hours of polish in the laboratory and traffic would enable the design engineer to select the pavement materials necessary to provide the skid resistance for that particular highway traffic. The Arkansas Accelerated Polishing Device and test procedure then could be used to pre-evaluate the asphalt mixture to ascertain that the desired long-term skid resistance was

provided.

3. The results of this investigation have demonstrated the capability of evaluating in the laboratory the ultimate polish (minimum friction) of an aggregate or asphalt mixture. Therefore, the minimum desired pavement frictional resistance in terms of SN_{40} for Arkansas pavements and traffic conditions should be determined and the results of this investigation implemented by specifying the asphalt mixture components to attain the required skid resistance.

4. It is also recommended that all types and gradations of asphalt mixtures used or to be used in Arkansas pavements be evaluated by the Arkansas Accelerated Polishing Device test procedure. The implementation of this recommendation will ensure that the asphalt mixtures used will be satisfactory for the purpose for which they were intended.

REFERENCES

1) Gillespie, William M. A Manual of the Principles and Practice of Road-Making, Comprising the Location, Construction, and Improvement of Roads, and Rail-Roads, 6th Ed., A. S. Barnes & Co., New York, 1853.

2) Goodwin, W.A. "Skid Resistance - A Historical Perspective," Proceedings, The Association of Asphalt Paving Technologists, Volume 42, 1973.

3) Agg, T.R. The Construction of Roads and Pavements, McGraw-Hill Book Co., Inc., New York, 1920.

4) Kummer, H.W. and Meyer, W.E. "Tentative Skid-Resistance Requirements for Main Rural Highways," Highway Research Board NCHRP Report No. 37, 1967.

5) Dillard, J.H. "Supplemental Pavement Skidding Resistance Tests in Virginia," Highway Research Board Bulletin 186, 1958.

6) Brown, M.G. and Finney, E.A. "Michigan's Skid Testing Program," Highway Research Board Bulletin 186, 1958.

7) Mullen, W.G. Skid Resistance and Wear Properties of Aggregates for Paving Mixtures, Project ERD-110-69-1, Final Report, Highway Research Program, North Carolina State Highway Commission, 1972.

8) Csathy, T.I., Burnett, W.C. and Armstrong, M.D. "State of the Art of Skid Resistance Research," Highway Research Board Special Report 95, 1968.

9) Beaton, J.S. "Providing Skid Resistant Pavements," Transportation Research Board Record 622, 1977.

10) Burnett, W.C., Gibson, J.L., and Kearney, E.J. "Skid Resistance of Bituminous Surfaces," Highway Research Board Record 236, 1967.

11) Sherwood, W.C. and Mahone, D.C. "Predetermining the Polish Resistance of Limestone Aggregates," Highway Research Board Record 341, 1970.

12) Grambling, W.L. "Effect of Aggregate Mineralogy on Polishing Rate and Skid Resistance in Pennsylvania," Highway Research Board Record 341, 1970.

13) Underwood, J.P., Hankins, D.D., and Garana, E. "Aggregate Polishing Characteristics: The British Wheel Test and the Insoluble Residue Tests," Research Report 126-2, Highway Design Division Research Section, Texas Highway Department, 1971.

14) Mullen, W.G., Dahir, S.H.M., and El Madani, N.F. "Laboratory Evaluation of Aggregates, Aggregate Blends, and Bituminous Mixes for Polish Resistance," Transportation Research Board Record 523, 1974.

15) Dahir, S.H. and Meyer, W.E. "Effects of Abrasive Size, Polishing Effort and Other Variables on Aggregate Polishing," Technical Paper presented to the Annual Meeting of the Transportation Research Board, 1976.

16) "Skid Resistance," National Cooperative Highway Research Program Synthesis of Highway Practice No. 14, Highway Research Board, 1972.

17) Nakkel, Egil. "Corrective Measures for Slipperiness - European View," Proceedings, The Association of Asphalt Paving Technologists, Volume 42, 1973.

18) Giles, C.G., Sabey, B.E., and Cardew, K.H.F. "Development and Performance of the Portable Skid-Resistance Tester," STP 326, American Society for Testing and Materials, 1962.

19) Kessinger, B.G. and Nielson, W.D. Skid Resistance Qualities as Related to the Chemical Composition of Limestone Aggregates in Arkansas, Arkansas State Highway Department, Division of Planning and Research, 1970.

20) Grambling, W.L. "Development Implementation of a Program to Reduce Skid Accidents," Transportation Research Board Record 622, 1977.

21) Martinez, J.E., Lewis, J.M., and Stocker, A.J. "A Study of Variables Associated with Wheel Spin-Down and Hydroplaning," Highway Research Board Record 396, 1972.

22) Elsenaar, P.M., Reichert, J., and Sauterey, R. "Pavement Characteristics and Skid Resistance," Transportation Research Board Record 622, 1977.

23) Lees, G. Katekhda, Izz ed Din, Bond, R., and Williams, A.R. "The Design and Performance of High Friction Dense Asphalts," Transportation Research Board Record 624, 1977.

24) Salt, G.F. "Research on Skid-Resistance at the Transport and Road Research Laboratory (1927-1977)," Transportation Research Board Record 622, 1977.

25) Rose, J.G., Galloway, B.M. "Water Depth Influence on Pavement Friction," Transportation Engineering Journal of ASCE, Americal Society of Civil Engineers, Volume 103, No. TE4, 1977.

26) Henry, J.J. and Hegman, R.R. "Pavement Texture Measurement and Evaluation," ASTM STP 583, American Society for Testing and Materials, 1975.

27) A Report on the CHLOE Profilometer, Arkansas State Highway Department, Division of Planning and Research, 1965.

28) Gee, S., Kin , W.L., and Hegman, R.R., "Pavement Texture Measurement by Laser: A Feasibility Study," ASTM STP 583, American Society for Testing and Materials, 1974.

29) Schonfeld, R. "Photo-Interpretation of Pavement Skid Resistance in Practice," Skid Resistance, Transportation Research Board Record 523, 1974.

30) Veres, R.E., Henry, J.J., and Lawther, J.M. "Use of Tire Noise as a Measure of Pavement Macrotexture," ASTM STP 583, American Society for Testing and Materials, 1975.

31) Doty, R.N. "Study of the Sand Patch and Outflow Meter Methods of Pavement Surface Texture Measurement," ASTM STP 583, American Society for Testing and Materials, 1975.

32) Standard Specifications for Highway Construction, Arkansas State Highway Commission, 1972.

33) Sandvig, L.D. and Kofalt, J.A. "Penetration and Viscosity Function in Bituminous Concrete," Proceedings, The Association of Asphalt Paving Technologists, Volume 37, 1968.

34) Kandhal, P.S., Brunner, R.J., and Nichols, T.H. "Design, Construction and Performance of Asphalt Friction Courses in Pennsylvania," Paper Presented to the Annual Meeting of the Transportation Research Board, January, 1977.

35) Adam, V. and Shah, S.C. "Evaluation of Open Graded Plant Mix Seal Surfaces for Correction of Slippery Pavement," Transportation Research Board Record 523, 1974.

36) Smith, R.W., Rice, J.M., and Spelman, S.R. "Design of Open-Graded Asphalt Friction Courses," Federal Highway Administration Report No. FHWA-RD-74-2, January, 1974.

37) Lamb, D.R. "Some U.K. Developments in Skid-Resistant Road Surfacings," Transportation Research Board Record 624, 1976.

38) Galloway, Bob M. and Hargett, E.R. "Blending Lightweight Aggregates with Natural Aggregates for the Production of Bituminous Concrete," Highway Research Board Record 273, 1968.

39) Jacks, Jr., J.W., "Investigation of Skid Resistance and Durability of Seven Asphalt Mixtures," Unpublished Master's Thesis, University of Arkansas, 1976.

40) Shook, J.F., Painter, L.J., and Lepp, T.Y. "Use of Loadometer Data in Designing Pavements for Mixed Traffic," Highway Research Board Record 42, 1963.

41) 1973 Annual Book of ASTM Standards, Part 11, American Society for Testing and Materials, 1973.

42) Dillard, J.H. and Mahone, D.C. "Measuring Road Surface Slipperiness," ASTM STP No. 366, American Society for Testing and Materials, 1964.

43) Draft Report No. TC-6, Arkansas State Highway Department, Arkansas No. 1, Contract No. DOT-FH-11-7949, FTC Center, TTI, Texas A&M University, College Station, Texas, June, 1973.

44) Hankins, K.D., "A Program for Reducing Skidding Accidents During Wet Weather," Transportation Research Board Record 622, 1977.

45) Lees, G. and Katekhda, I.e.D. "Prediction of Medium and High Speed Skid Resistance Values by Means of a Newly Developed Outflow Meter," Proceedings, The Association of Asphalt Paving Technologists, Volume 43, 1974.

46) A Policy on Geometric Design of Rural Highways, American Association of State Highway Officials, Washington, D.C., 1965.

47) Smith, H.A. "Pavement Contributions to Wet-Weather Skidding Accident Reduction," Transportation Research Board Record 622, 1977.

48) Goodwin, W.A. "Pre-Evaluation of Pavement Materials for Skid Resistance - A Review of U.S. Techniques," Highway Research Board Special Report 101, 1969.

49) McCasland, Willard. Study of the Wearability and Polish Resistance of Oklahoma's Coarse Aggregates, Research and Development Division, Oklahoma Department of Highways, 1972.

50) Stephens, J.E. and Goetz, W.H. "Designing Fine Bituminous Mixtures for High Skid Resistance," Proceedings, Volume 39, 1960.

51) Mullen, W.G. and Dahir, S.H.M. Skid Resistance and Wear Properties of Aggregates for Paving Mixtures, Project ERD-110-69-1, Interim Report, Highway Research Program, North Carolina State University, June, 1970.

52) Dahir, S.H., Meyer, W.E., and Hegman, R.R. "Laboratory and Field Investigation of Bituminous Pavement and Aggregate Polishing," Paper Presented to the 54th Annual Meeting of the Transportation Research Board, January, 1975.

53) Mullen, W.G. "Prediction of Pavement Skid Resistance from Laboratory Tests," Transportation Research Board Record 523, 1974.

54) Furbush, M.A. and Styers, K.E. The Relationship of Skid Resistance to Petrolography of Aggregates, Final Report, Pennsylvania Department of Transportation, July, 1972.

55) Hveem, F.N. "Types and Causes of Failure in Highway Pavements," Highway Research Board Bulletin 187, 1958.

56) Hubbard, P. "Adhesion of Asphalt to Aggregates in the Presence of Water," Proceedings, Highway Research Board, Volume 18, Part 1, 1938.

57) Mathews, D.H. "Adhesion in Bituminous Road Materials: A Survey of Present Knowledge," Journal of the Institute of Petroleum, Volume 44, No. 420 (Dec.), 1958.

58) Majidzadeh, K. and Brovold, F.N. "State of the Art: Effect of Water on Bitumen-Aggregate Mixtures," Highway Research Board Special Report 98, 1968.

59) Nicholson, V. "Adhesion Tension in Asphalt Pavements, Its Significance and Methods Applicable in Its Determination," Proceedings, The Association of Asphalt Paving Technologists, Volume 3, 1932.

60) Thelen, E. "Surface Energy and Adhesion Properties in Asphalt-Aggregate Systems," Highway Research Board Bulletin 192, 1958.

61) Riedel, W. "Mechanism of Adhesion of Bitumen to Stone," Asphalt und Teer, Volume 34, 1934.

62) Winterkorn, H.F., Eckert, G.W., and Shipley, E.B. "Testing the Adhesion Between Bitumen and Mineral Surfaces with Alkaline Solutions," Proceedings, The Association of Asphalt Paving Technologists, Volume 9, 1937.

63) Hughes, R.I., Lamb, D.R. and Pordes, D. "Adhesion in Bitumen Macadam," Journal of Applied Chemistry, Volume 10, No. 11, 1960.

64) Lee, A.R. "Adhesion in Relation to Bituminous Road Materials," Journal of the Society of Chemical Industry, February, 1936.

65) Tyler, O.R. "Adhesion of Bituminous Films to Aggregates," Purdue University Research Bulletin, No. 62, Volume 22, September, 1938.

66) Stross, F.H. and Anderson, A.P. "Stripping vs. Coating of Mineral Aggregate," Proceedings, The Association of Asphalt Paving Technologists, Volume 13, 1942.

67) Rice, J.M. "Relationship of Aggregate Characteristics to the Effect of Water on Bituminous Paving Mixtures," ASTM STP 240, American Society for Testing and Materials, 1958.

68) McLeod, N.W. "Applications of Surface Chemistry and Physics to Bituminous Mixtures," Proceedings, The Association of Asphalt Paving Technologists, Volume 9, 1937.

69) Mack, C. "Physico-Chemical Aspects of Asphalt Pavements: Energy Relations at Interface Between Asphalt and Mineral Aggregate and Their Measurements," Industrial and Engineering Chemistry, December, 1935.

70) Stevens, D.E. "Ravelling," Proceedings, The Association of Asphalt Paving Technologists, Volume 28, 1959.

71) Fromm, H.J. "The Mechanisms of Asphalt Stripping from Aggregate Surface," Proceedings, The Association of Asphalt Paving Technologists, Volume 43, 1974.

72) Dow, A.W. "The Wash Test in Differentiating Between Aggregates or Colprovia Paving Mixtures," Proceedings, The Association of Asphalt Paving Technologists, Volume 7, 1936.

73) Lang, F.C. and Thomas, T.W. "Laboratory Studies of Asphalt Cements," University of Minnesota, Engineering Experiment Station Bulletin No. 15, 1939.

74) Critz, P.F. and Goode, J.F." Additives for Bituminous Materials," Public Roads, Volume 24, No. 5, July-August-September, 1945.

75) Sanderson, F.C. "Methylchlorosilanes as Antistripping Agents," Proceedings, Highway Research Board, Volume 31, 1952.

76) "Method of Test for Film Stripping," Test Method No. Calif. 302-C, State of California,Department of Public Works, Division of Highways, July, 1963.

77) Stanton, T.E. and Hveem, F.N. "Role of the Laboratory in the Preliminary Investigation and Control of Materials for Low Cost Bituminous Pavements," Proceedings, Highway Research Board, Volume 14, Part II, 1934.

78) Holmes, A. "Evaluating the Adhesive Properties of Asphalt," Proceedings, American Society for Testing and Materials, Volume 39, 1939.

79) Gzemski, F.C. "Factors Affecting Adhesion of Asphalt to Stone," Proceedings, The Association of Asphalt Paving Technologists, Volume 17, 1948.

80) Carroll, D. Rock Weathering, Plenum Press, New York, 1970.

81) Roediger, J.C. "Stripping Test for Bitumen-Aggregate Mixtures," Proceedings, American Society for Testing and Materials, Volumes 52, 1952.

82) "Standard Method of Test for Coating and Stripping of Bitumen-Aggregate Mixtures," Designation: D1664, 1972 Annual Book of ASTM STandards, Part II.

83) Karius, H. and Dalton, J.L. "Detachment of Stone from Binder Under Influence of Water in Road Surface Dressings," Journal of Institute of Petroleum, Volume 50, No. 481, January 1964.

84) Riedell, W. and Weber, H. "On the Adhesiveness of Bituminous Binders of Aggregates," Asphalt und Teer, Volume 33, 1933.

85) Ford, M.C., Manke, P.G., and O'Bannon, C.E. "Quantitative Evaluation of Stripping by the Surface Reaction Test," Transportation Research Board Record 515, 1974.

86) Krchma, L.C. and Loomis, R.J. "Bituminous-Aggregate Water Resistance Studies," Proceedings, The Association of Asphalt Paving Technologists, Volume 15, 1943.

87) Pauls, J.T. and Rex, H.M. "A Test for Determining the Effect of Water on Bituminous Mixtures," Public Roads, Volume 24, No. 5, July-August-September, 1945.

88) Pauls, J.T. and Goode, J.F. "Application and Present Status of the Immersion-Compression Test," Proceedings, The Association of Asphalt Paving Technologists, Volume 16, 1947.

89) Pauls, J.T. and Goode, J.F. "Further Developments and Application of the Immersion-Compression Test," Public Roads, Volume 25, No. 6, December, 1948.

90) "Standard Method of Test for Effect of Water on Cohesion of Compacted Bituminous Mixtures," Designation: D1075, 1972 Annual Book of ASTM Standards, Part 11.

91) "Flexible Airfield Pavements," U.S. Army Corps of Engineers, EM 1110-45-302, August 1958.

92) Swanberg, J.H. and Hindermann, W.L. "The Use of an Abrasion Test as a Measure of Durability of Bituminous Mixtures," ASTM STP 94, American Society for Testing and Materials, 1949.

93) Mathews, D.H., Colwill, D.M. and Yuce, R. "Adhesion Tests for Bituminous Materials," Journal of Applied Chemistry, Volume 15, No. 9, 1965.

94) Olsen, R.E. "Stability of Bituminous Pavements Related to Aggregate Characteristics," Public Roads, Volume 33, No. 3, August, 1964.

95) Eager, W.L. "Effect of Moisture on Bituminous Pavements in Rock Mountain Areas," Public Roads, Volume 33, No. 3, August, 1964.

96) Goldbeck, A.T. "Immersion-Compression Tests Compared with Laboratory Traffic Tests on Bituminous Concrete," ASTM STP 94, American Society for Testing and Materials, 1949.

97) Andersland, O.B. and Goetz, W.H. "Sonic Test for Evaluation of Stripping Resistance in Compacted Bituminous Mixtures," Proceedings, The Association of Asphalt Paving Technologists, Volume 25, 1956.

98) Goode, J.F. "Use of the Immersion-Compression Test in Evaluating and Designing Bituminous Paving Mixtures," ASTM STP 252, American Society for Testing and Materials, 1959.

99) Goldbeck, A.T. "Tests for the Traffic Durability of Bituminous Pavements," Proceedings , The Association of Asphalt Paving Technologists, Volume 7, 1936.

100) Klinger, E.W. and Roediger, J.C. "Laboratory and Field Studies of Asphalt Paving Mixtures," ASTM STP 240, American Society for Testing and Materials, 1958.

101) Mathews, D.H. and Colwill, D.M. "The Immersion Wheel-Tracking Test," Journal of Applied Chemistry, Volume 12, No. 11, 1962.

102) Mathews, D.H. "Surface-Active Agents in Bituminous Road Materials," Journal of Applied Chemistry, Volume 12, No. 2, 1962.

103) Skog, J. and Zube, E. "New Test Methods for Studying the Effect of Water Action on Bituminous Mixtures," Proceedings, The Association of Asphalt Paving Technologists, Volume 32, 1963.

104) Schmidt, R.J. and Graf, P.E. "The Effect of Water on the Resilient Modulus of Asphalt-Treated Mixes," Proceedings, The Association of Asphalt Paving Technologists, Volume 41, 1972.

105) Dictionary of Geological Terms, American Geological Institute, Anchor Books, New York, 1974.

106) Pirsson, L.V. Rocks and Rock Minerals, John Wiley and Sons, Inc., New York, 1908.

107) McBride, E.F. "A Classification of Common Sandstones," Journal of Sedimentary Petrology, Volume 33, No. 3, 1963.

108) Report No. TC-50, Arkansas State Highway Department, Arkansas No. 2, Contract No. DOT-FH-8889, Central/Western FTC, TTI, Texas A&M University, Colelge Station, Texas, January, 1977.

109) Manual of Field Sampling and Testing Procedures, Division of Materials and Tests, Arkansas State Highway Department, Undated (Author's copy received January 7, 1974.)

110) Asphalt Surface Treatments and Asphalt Penetration Macadam, Manual Series No. 13, 2nd Edition, The Asphalt Institute, 1969.

111) Wallace, Hugh A. and Martin, J.R. Asphalt Pavement Engineering, McGraw-Hill Book Company, New York, 1967.

112) "Accelerated Polish Test for Coarse Aggregate," Test Method Tex-438-A, Materials and Tests Division, Texas Highway Department, January 1, 1974.

113) Gunn, P.E. "Laboratory Investigation of the Polishing Characteristics of Aggregates Used in Arkansas Asphalt Pavements," Unpublished Master's Thesis, University of Arkansas, 1977.

114) Parker, Herbert J., Jr. "An Economical Analysis to Determine the Feasibility of Blending Aggregates for Improving Pavement Skid Resistance," Unpublished Report for C.E. 5633, University of Arkansas, 1977.

APPENDIX A

Method of Test for Total Insoluble Residue in Coarse Aggregate

METHOD OF TEST FOR TOTAL INSOLUBLE RESIDUE IN COARSE AGGREGATE

OHD-L-25 (6-29-70)

Scope: 1. This method of test is intended for the determination of acid insoluble material in aggregates used in asphaltic concrete.

Apparatus: 2. The apparatus for this test will consist of the following:

(a) Half-gallon jars

(b) Hydrochloric Acid Technical Grade

(c) Evaporating Dishes (Vycor 350 ml)

Procedure: 3. (a) Crush sample so that all material is less than one-half inch.

(b) Split the sample to approximately 200 grams, weigh accurately, and place in clean, labeled half-gallon jar.

(c) Add 400 ml of water and slight excess of concentrated hydrochloric acid (approximately one ml per gram of rock) over amount needed to react with available carbonate. Stir mixture over a period of days until all reaction ceases.

(d) Wash the insolubles free of excess ions by filling jar with tap water, allowing all of the material to settle (about 48 hours) and pour off the clear solution. Procedure is repeated three times.

(e) After the third wash cycle, wash the insolubles into an evaporating dish, dry at 100-105 C and weigh.

Report: 4. Report insoluble residues as retained on the #200 sieve as percent of total sample used.

APPENDIX B

Details of Arkansas Accelerated Polishing Device

Figures B1 - B3

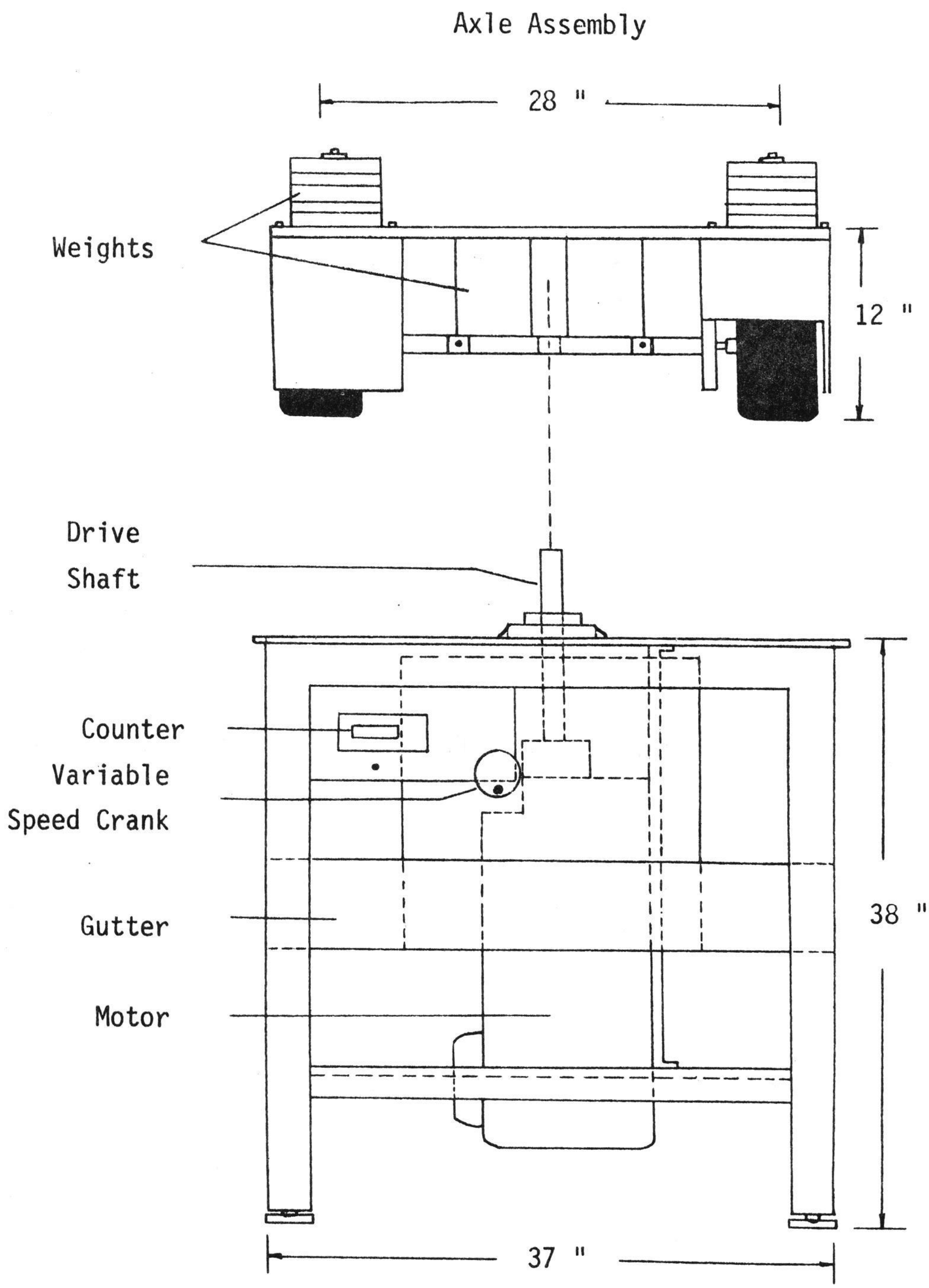


Figure B1. Arkansas Accelerated Polishing Device

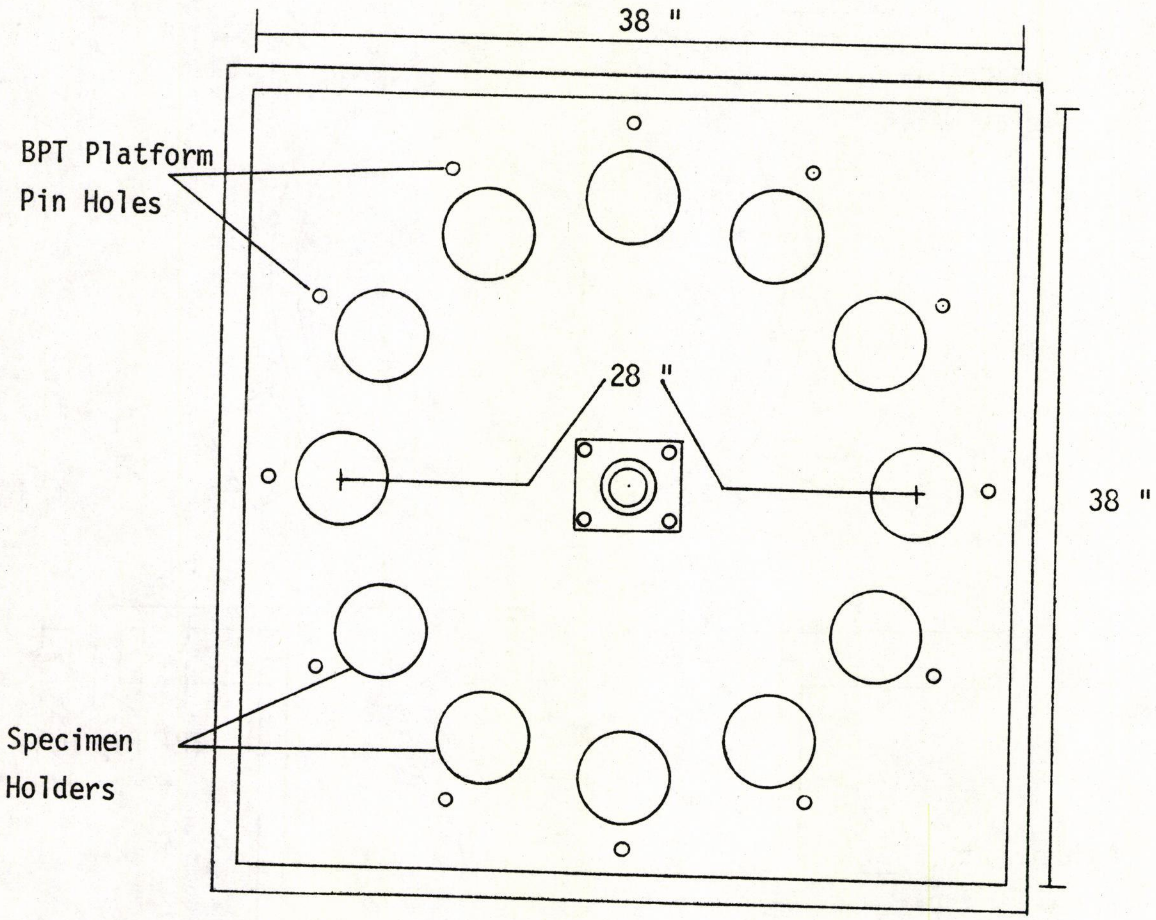


Figure B2. Arkansas Accelerated Polishing Device Plan View

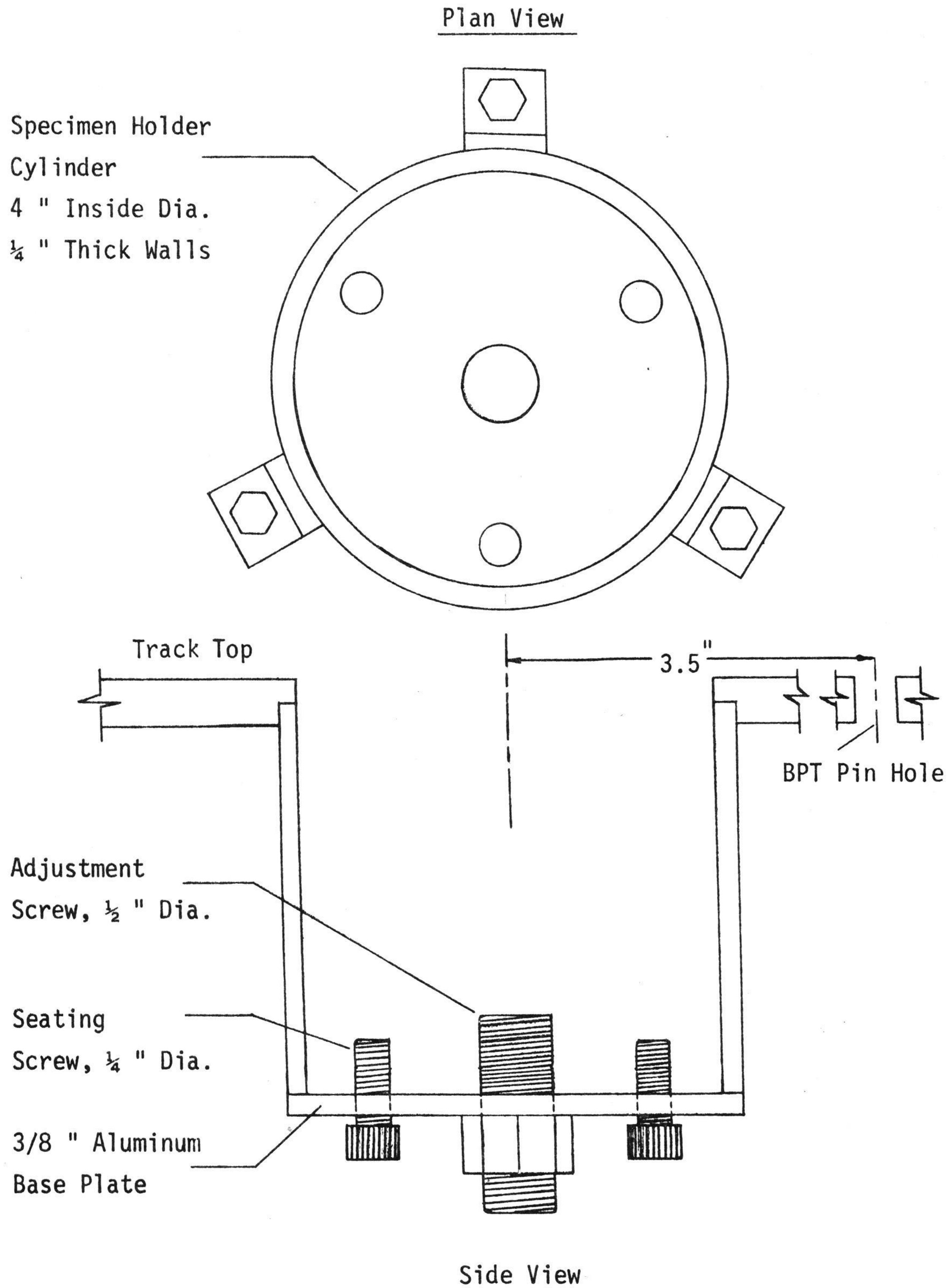


Figure B3. Arkansas Accelerated Polishing Device Specimen Holder

APPENDIX C

Procedure for Casting Polyester Specimens for Accelerated Polishing

Figures C1 - C4

PROCEDURE FOR CASTING POLYESTER SPECIMENS FOR ACCELERATED POLISHING

I. Placing Aggregate in Mold

A. To prepare mold for casting, clean thoroughly with Acetone to remove any polyester which might be on the mold from previous castings. Next wax the mold with two or three coats of wax. This will help prevent polyester from sticking to the sides of the mold and will aid in removing the sample when it has hardened.

B. After cleaning and waxing, place the mold together, being sure that the ring slides easily onto the base. If the ring and base are in a bind, separation will be difficult after casting. Measure out approximately 100 grams of the aggregate sample to be encased in polyester. It is recommended that a sample splitter be used to obtain a representative sample.

C. Placement of the aggregate in the mold is a slow and tedious operation. Spread the aggregate over a smooth surface to clearly identify each stone individually. Placement of the aggregate in the mold is done much the same way as one would put a jigsaw puzzle together. Begin by placing individual stones in the mold, being sure to place the flattest surface of that stone face down on the steel base plate of the mold. The order in which the stones go into the mold is not as important as fitting the stones together as compactly as possible. One suggested method for placing the aggregate is to start around the exterior of the mold with the larger stones, then work inward using progressively smaller stones.

D. Once the aggregate is in the mold, it is necessary to fill the openings between the stones to prevent the polyester from pushing through, thus affecting the skid test. Ottawa sand placed carefully between the stones works very effectively for this purpose. However, if too much sand is used, proper bonding between the polyester and the aggregate will be prevented.

II. Placing Polyester in Mold

A. Measure out approximately 100 grams of polyester for each sample to be molded. Do this in a paper cup as disposal is much easier than cleaning. Also, disposable wooden stir sticks are advisable. (Note: The following mixing and casting should be done in a vented laboratory hood and rubber gloves should be worn at all times.) Hardener is added to the polyester. The proper proportion of hardener to polyester is 17 drops of hardener per 100 grams of polyester. It is advisable to add the drops in stages with vigorous stirring between additions of drops. The hardener must be well mixed with the polyester resin to ensure a good specimen.

B. Placement of the polyester in the mold on top of the aggregate requires patience. Again, beginning around the outside, using the wooden stir stick place a small amount of polyester in the mold next to the edge. Then carefully draw the stick away using the edge of the mold to pull off the polyester. This procedure is illustrated in Figure C1. As the stick is pulled out, the polyester should be gently pushed down to prevent the polyester from pulling the aggregate out of position. This will also aid in pushing the polyester down between the stones to provide better contact.

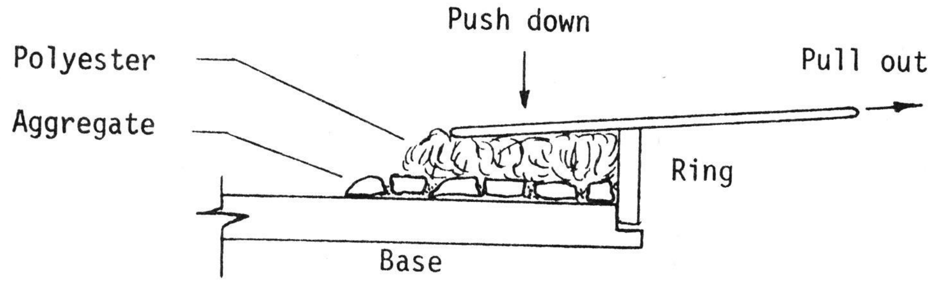


Figure C1.

Continue to work around the inside ring edge until the outer portion of the specimen is covered with polyester. Then fill in the center as shown in Figure C2.

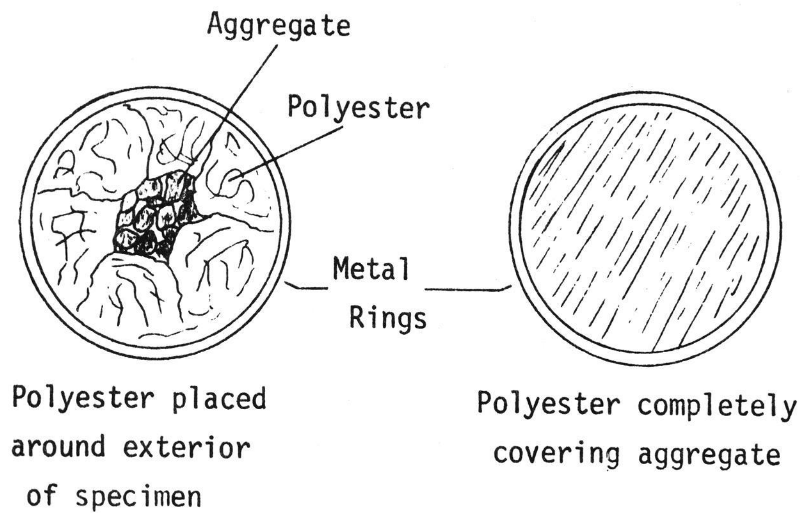


Figure C2

Once the aggregate is completely covered, draw a straight edge across the top of the ring to smooth the top surface and remove any excess polyester.

III. <u>Preparing Specimen for Hardening of the Polyester Resin</u>

A. The specimen should now set for at least two days before being removed from the mold. However, if the specimen is not weighted properly it will shrink and curl while hardening. Curved, or bowed, specimens should be rejected.

B. With the remainder of the polyester build up a small mound around the now smooth surface approximately one-half inch inside the ring's inner edge as shown in Figure C3.

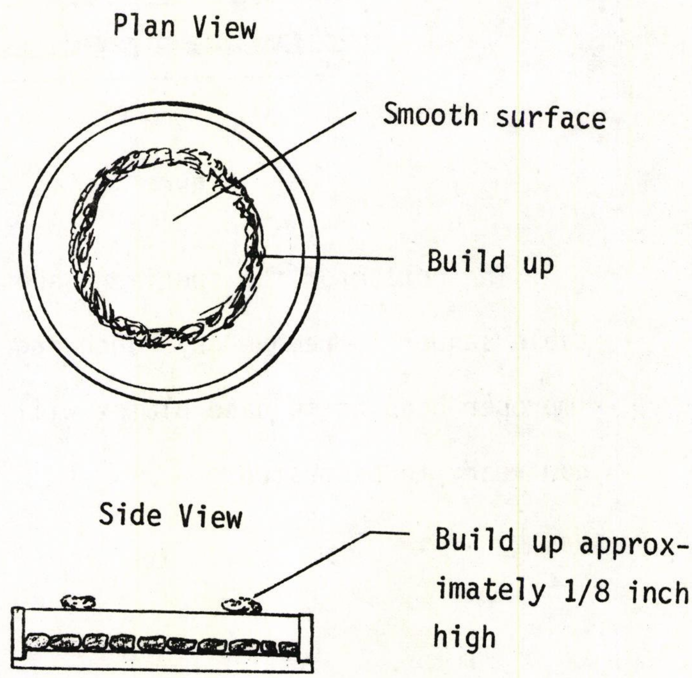


Figure C3

C. Now place an oiled flat board or piece of metal on the specimen and place approximately 5 to 10 lbs. on top of the board. This will keep the specimen flat while hardening and is illustrated in Figure C4.

D. The specimen should stay in the mold for at least two days. At the end of this two day curing period, carefully separate the specimen from the mold and try to avoid breaking the specimen or pulling any aggregate out. The specimen should then cure for another two days outside the mold under a weight of 2 to 5 lbs.

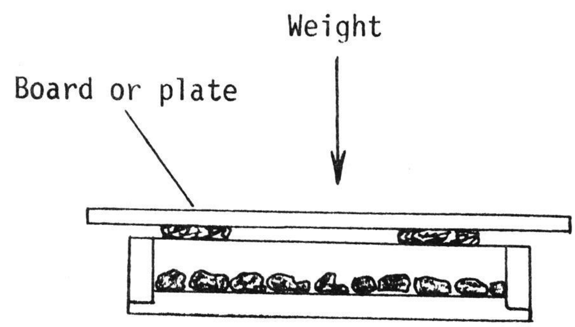


Figure C4

The bottom of the specimens should be sanded smooth with a table sander to remove any rough edges. If this is not done, improper bonding to base plates will result. The specimens are now ready to be tested.

APPENDIX D

Surface Reaction Test Procedure

Figures D1 and D2

SURFACE REACTION TEST PROCEDURE

Sample Preparation

The sample to be evaluated was prepared and coated with asphalt cement and then subjected to the effect of water by the SIS or DIS procedure previously described. Thus the aggregate samples used in this test had previously been coated with asphalt cement and partially stripped in the SRT and DIS tests. An uncoated duplicate sample of the aggregate was immersed in distilled water at the same time as the SIS and DIS specimen was immersed. At the end of the SIS and DIS test, the partially stripped and uncoated samples were dried by blotting with paper towels and spread in pans to air dry. The samples were air dried approximately 24 hours before the surface reaction test.

It is desirable to perform this test on oven-dry materials; however, when the partially stripped aggregate samples were oven dried at 212 F, the remaining asphalt cement diffused and completely recoated the stripped aggregate surfaces. Because of this recoating tendency, oven drying was eliminated and the samples were simply air dried before testing.

Reagent Preparation

Each test required 200 ml of acid solution. Because duplicate samples i.e., two uncoated and four partially coated samples, of each of the various aggregates were tested, a liter of acid solution was prepared for each series of aggregate samples. The acid test solutions were prepared with reagent grade acids and deaired distilled water. All proportions of these acid solutions were calculated on a weight basis.

The 200 ml of acid solution to be used in each test was measured with a graduated cylinder and placed in a 250 ml nalgene jar. The weight of acid solution in each jar was determined and its density, in g/ml, was calculated. This density and weight determination was used as a check for obtaining equal strengths of solution for each test. The balance of the liter of solution originally prepared was retained for titrations. The actual normality of the acid was determined by titration against a known weight of sodium carbonate using methyl orange as an indicator.

Calibration of Equipment

Prior to beginning a series of tests, the strip chart recorder was checked to ascertain proper operation. The pressure was checked with a King Manometer Pressure Regulator, while the thermistor and Tele-Thermometer temperatures were checked with a precise thermometer.

Pressure Vessel

The pressure vessel was to be operated as a closed system. Therefore, it was necessary to provide a means of adding the acid to the aggregate without changing the ambient pressure. A 250 ml stainless steel beaker was attached using a threaded joint, to a rod extending through the body of the pressure vessel. The rod opening was sealed by using neoprene o-rings both inside and outside the wall of the pressure vessel. A handle was attached to the exterior end of the shaft. To inundate the test specimen, the beaker was positioned upright and filled with 200 ml of the acid solution, then the handle was turned until the contents of the beaker were poured into the plastic container holding the aggregate sample. Details of the pressure container device are shown in Figure D1.

The pressure container was connected to the recorder and Tele-Thermometer by about 15 feet of electrical wiring. Fume hood temperature was maintained about 68 F. Before initiating a test, samples to be tested, a graduated cylinder and placed in a 250 ml nalgene jar. The weight of acid solution in each jar was determined and its density, in g/ml, was calculated. This density and weight determination was used as a check for obtaining equal strengths of solution for each test. The balance of the liter of solution originally prepared was retained for titrations. The actual normality of the acid was determined by titration against a known weight of sodium carbonate with methyl orange as an indicator.

Calibration of Equipment

Prior to beginning a series of tests, the strip chart recorder was checked to ascertain proper operation. The pressure was checked with a King Manometer Pressure Regulator, and the thermistor and Tele-Thermometer temperatures were checked with a precise thermometer.

Pressure Vessel

The pressure vessel was to be operated as a closed system. Therefore, it was necessary to provide a means of adding the acid to the aggregate without changing the ambient pressure. A 250 ml stainless steel beaker was attached by means of a threaded joint to a rod extending through the body of the pressure vessel. The rod opening was sealed by using neoprene o-rings both inside and outside the wall of the pressure vessel. A handle was attached to the exterior end of the shaft. To inundate the test specimen, the beaker was positioned upright and filled with 200 ml of the acid solution, then the handle was turned until the contents of the beaker were poured into the plastic container holding the aggregate sample. Details of the pressure container device are shown in Figure D1.

The pressure container was connected to the recorder and Tele-Thermometer by about 15 feet of electrical wiring. Fume hood temperature was maintained about 68 F. Before a test, samples to be tested, acid

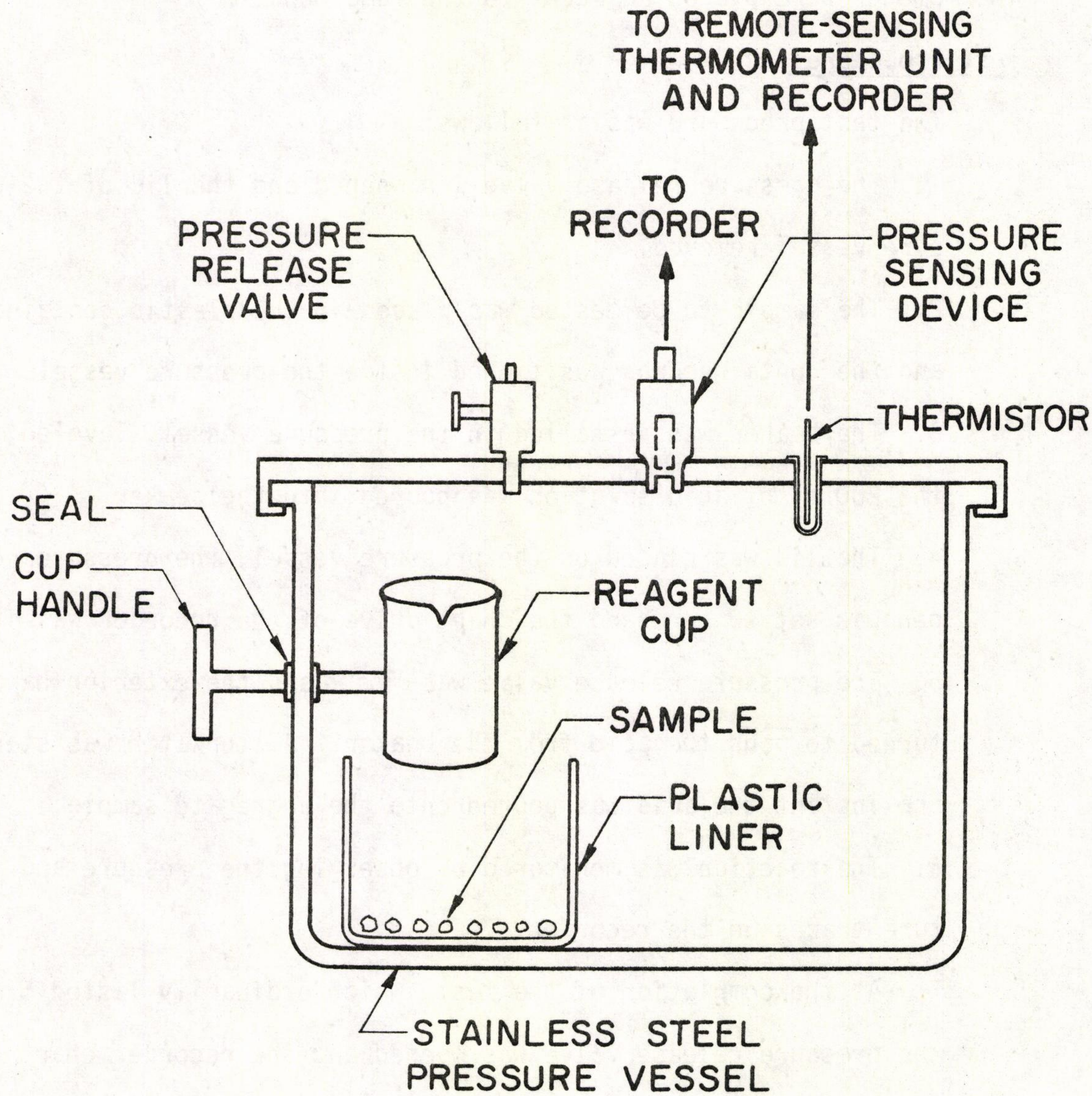


Figure D1. Details of Pressure Container Device
U.S. Patent No. 3,915,636 of October 28, 1975

solutions, and the pressure container were all placed in the fume hood and brought to a constant temperature. The recorder and Tele-Thermometer were placed adjacent to the fume hood.

Test Procedure

The test procedure was as follows:

1. The pressure release valve was opened and the lid of the pressure vessel removed.

2. The sample to be tested was placed in the plastic container and the container was positioned inside the pressure vessel.

3. The beaker was installed in the pressure vessel, leveled, and the 200 ml of acid solution was poured into the beaker

4. The lid was placed on the pressure vessel, the pressure recording pen was set to zero,and the chart drive of the recorder was started.

5. The pressure release valve was closed and the exterior handle turned to pour the acid from the beaker. A stopwatch was started at the instant the acid was poured onto the aggregate sample.

6. The reaction was monitored by observing the pressure and temperature traces on the recorder.

7. At the completion of the test, which ordinarily lasted 5 minutes, the pressure release valve was opened and the recorder chart drive stopped.

8. After the pressure was released, the top of the pressure vessel was removed and the acid beaker taken out. The sample was removed and the reaction of the acid solution and sample was terminated.

   a. For samples tested with HCL this was accomplished by flooding the mixture with tap water.

   b. For samples tested with HF the reaction was stopped by slowly adding a sufficient amount of calcium oxide slurry to deplete the HF in the mixture. Methyl orange indicator was used to determine when the solution was neutralized.

9. The acid beaker and plastic sample container were then washed and dried before another test was begun.

Stripping Calculation

The pressure-temperature curves plotted on the recorder chart were then analyzed. A horizontal line was drawn on the chart paper for each 15 seconds of elapsed reaction time. The pressure and temperature readings were scaled from the chart paper and tabulated. Pressures were adjusted to 68 F for comparative stripping calculations. This adjustment of pressures was necessary because of the slightly different operating temperatures and the higher temperatures created by some of the reactions.

The surface area exposed was considered proportional to the change in pressure over a certain time interval. For limestone aggregates and aggregates of mixed composition, the change in pressure from 0.25 to 1.5 minutes of reaction time was used. The reaction between the siliceous aggregates and the HF solution was slower or less violent and required a longer reaction time (0.25 to 5.0 minutes) to obtain a significant pressure difference. The effect of inertia on the pressure transducer operation and the recorder chart pen response was the primary reason for using the initial gas pressure value at 0.25 minutes of reaction time. The reaction time used for the final pressure value was the time required to obtain a measurable pressure without deep etching of the exposed aggregate surface.

A drawing typical of the strip-chart recorder trace of the pressures obtained for an uncoated limestone aggregate sample and a partially coated limestone aggregate sample is shown in Figure D2. The initial pressure reading, taken at 15 seconds of reaction time, is shown as P_1, and the final pressure reading, taken at 90 seconds of reaction time, is shown as P_2. The retained coating of asphalt was calculated as:

$$RC = 100 - \left(\frac{\Delta P_s}{\Delta P_u}\right) 100$$

where:

RC = percent retained coating of stripped sample

$\Delta P_s = P_2 - P_{1s}$ = change in pressure for stripped sample.

$\Delta P_u = P_{2u} - P_{1u}$ = change in pressure for uncoated sample.

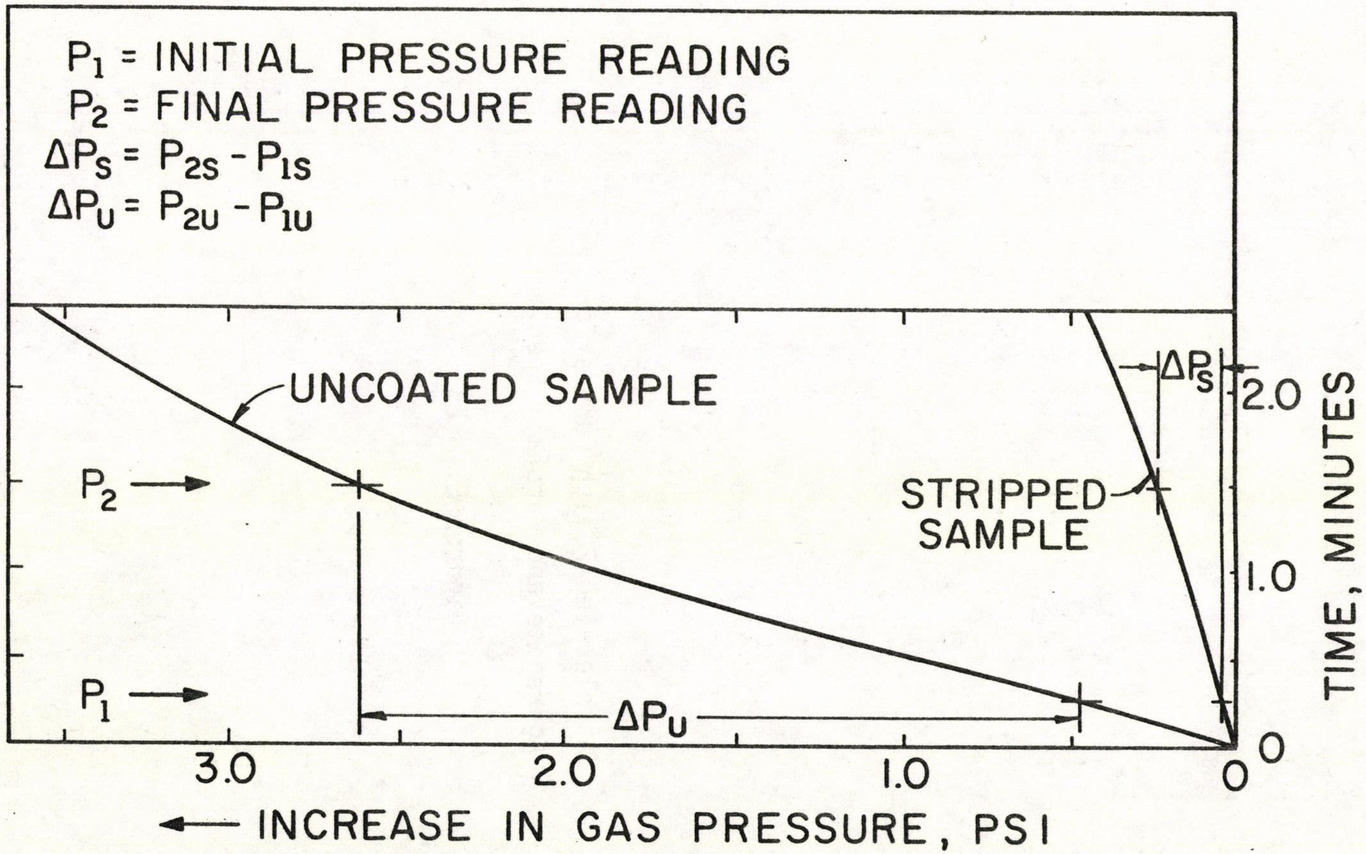


Figure D2. Typical Surface Reaction Test Pressure-Time Curves for Limestone Aggregate, from Strip-Chart Recorder

APPENDIX E

Accelerated Polish Curves for Aggregate and Marshall Specimens

Figures E1 - E8

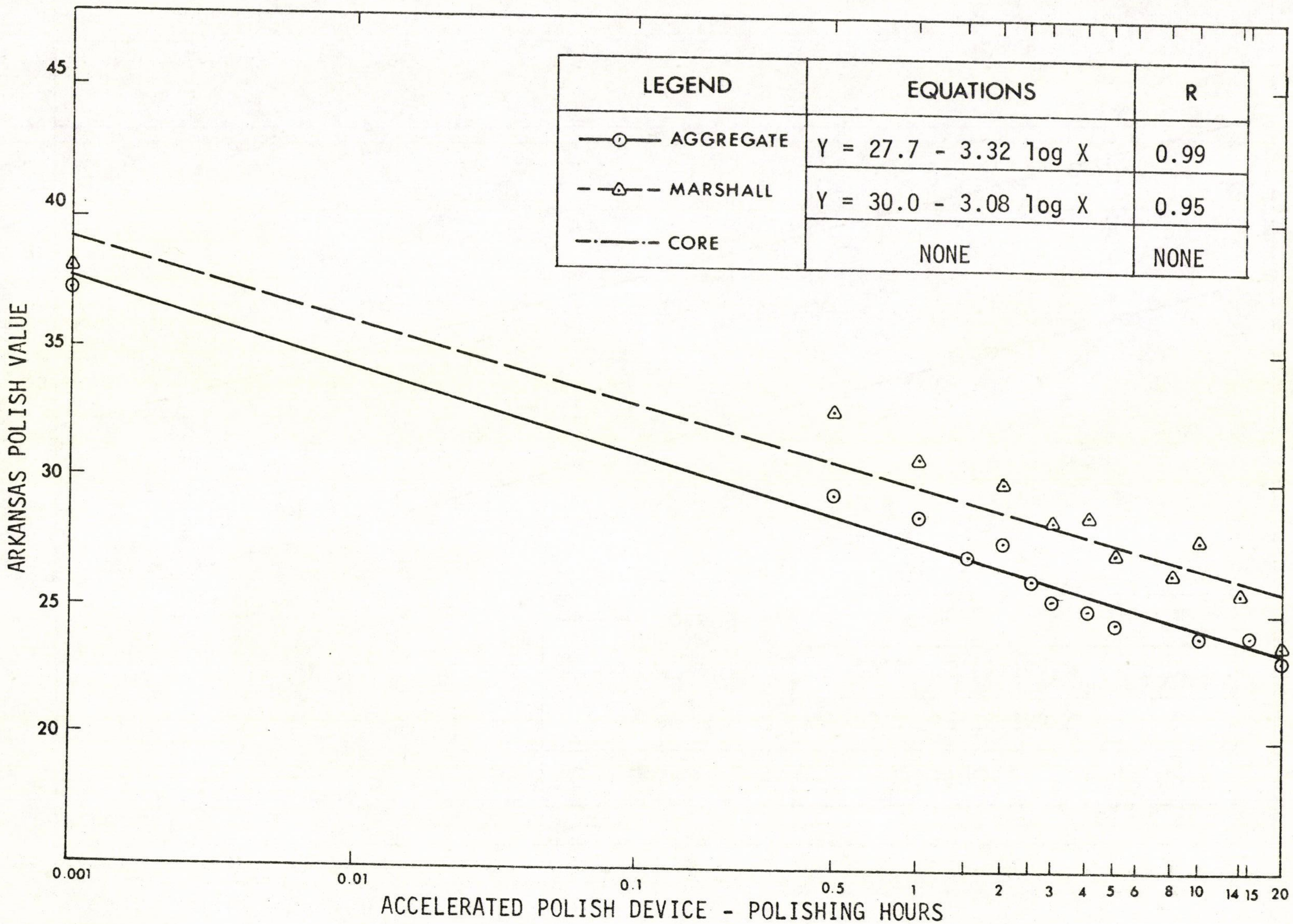


Figure E1. Arkansas Polish Value vs. Hours of Polish - Aggregate and Marshall - Twin Lakes Limestone

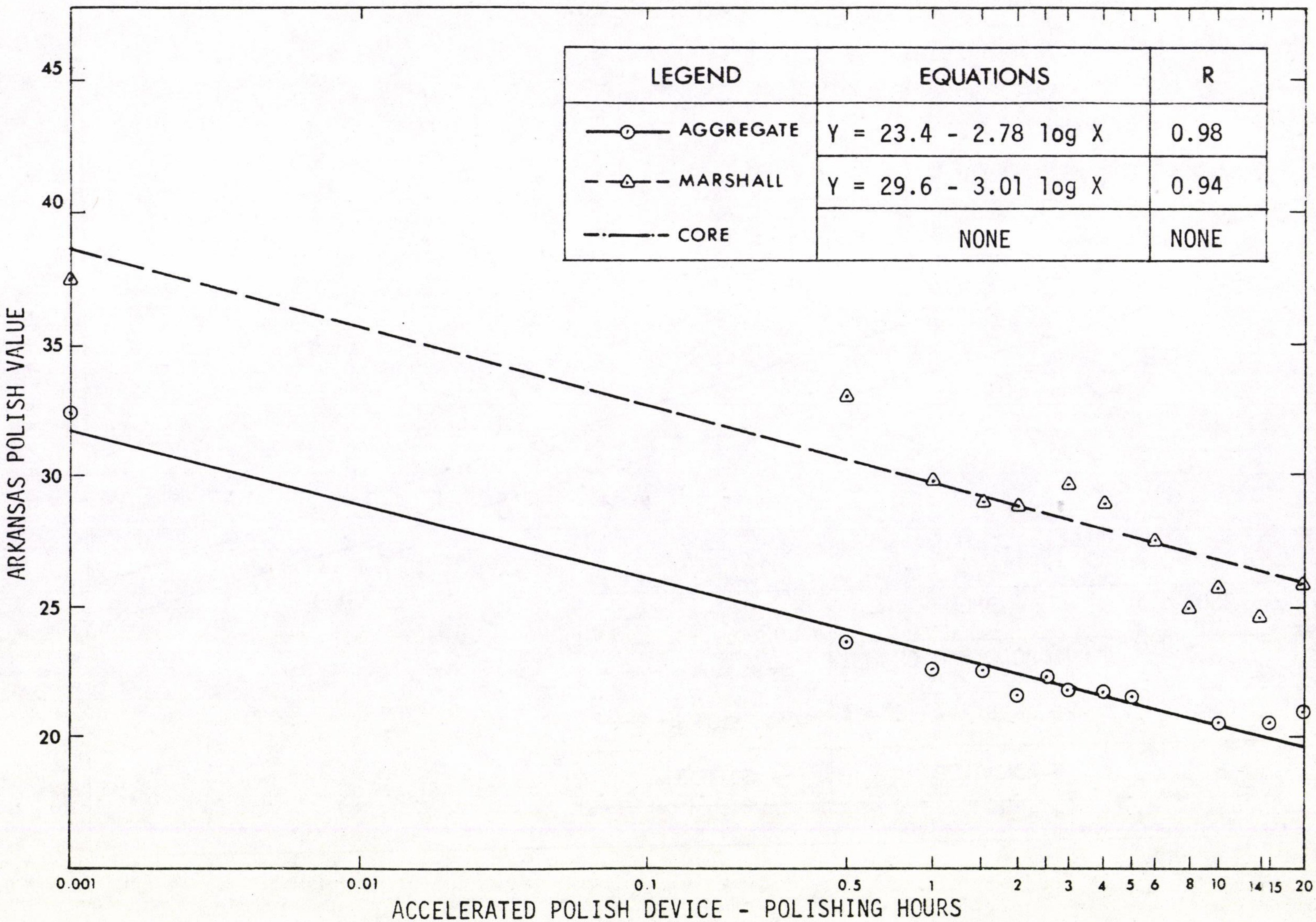


Figure E2. Arkansas Polish Value vs. Hours of Polish - Aggregate and Marshall - Kentucky Limestone

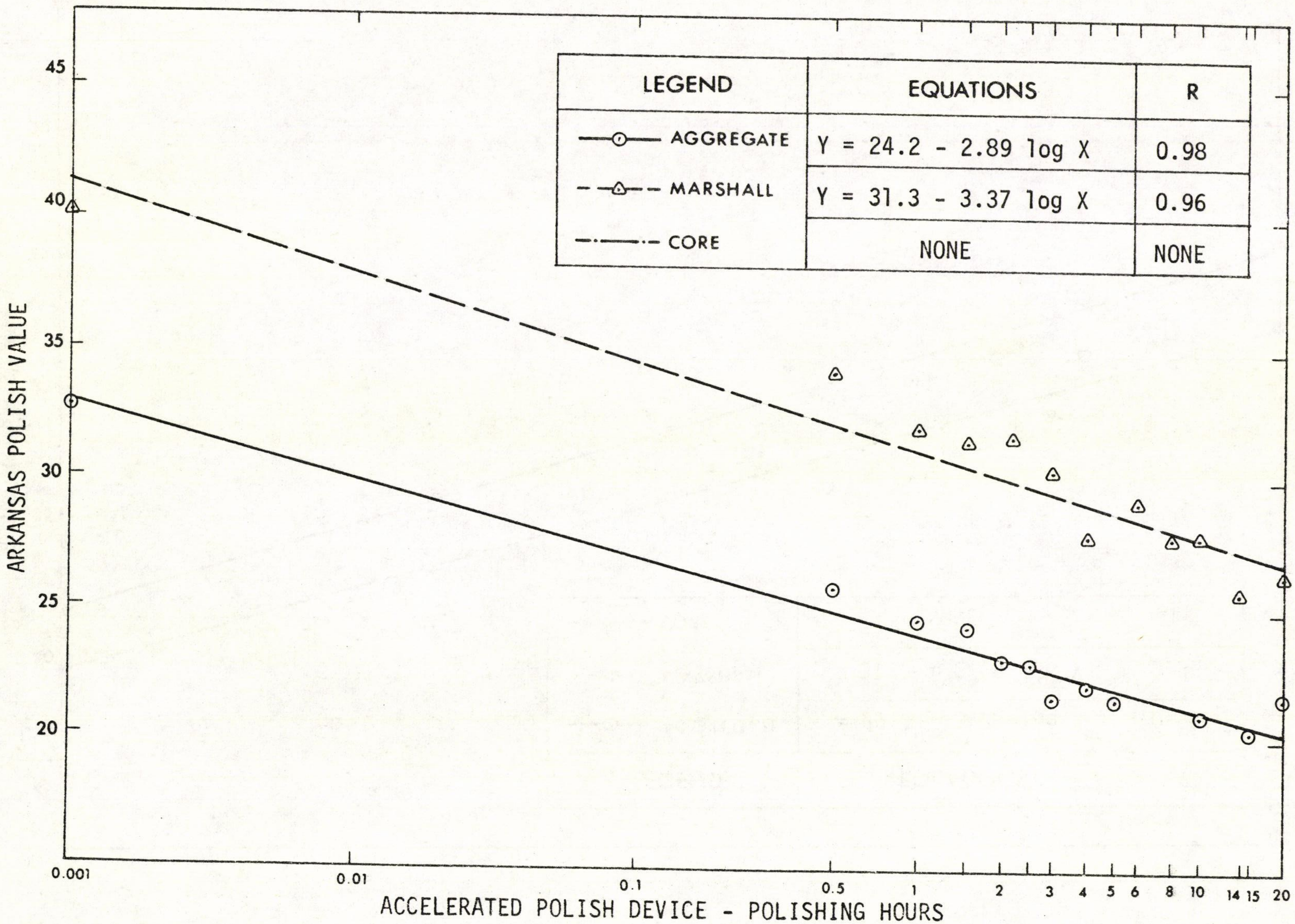


Figure E3. Arkansas Polish Value vs. Hours of Polish - Aggregate and Marshall - Rocky Point Limestone

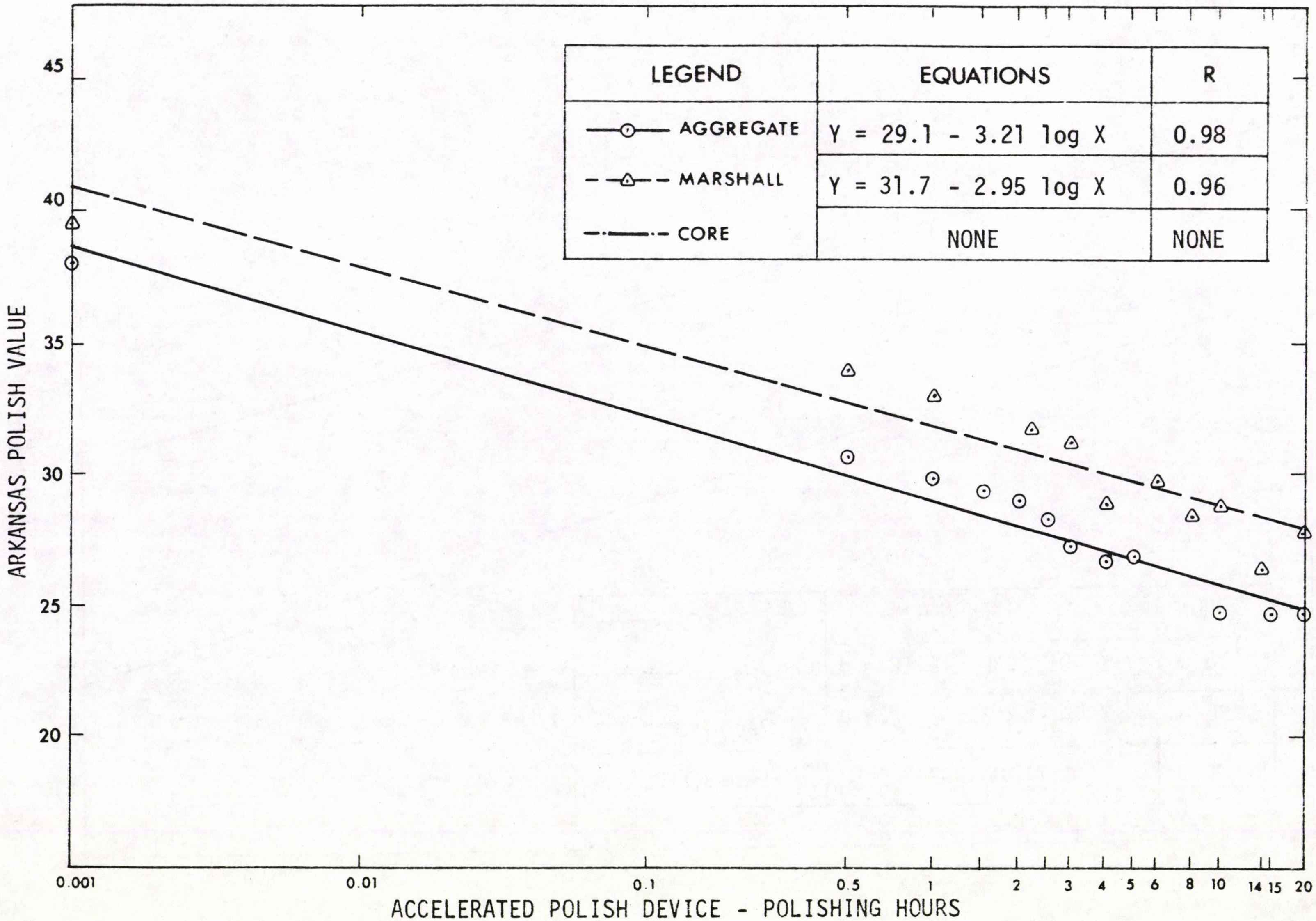


Figure E4. Arkansas Polish Value vs. Hours of Polish - Aggregate and Marshall - Black Rock Limestone

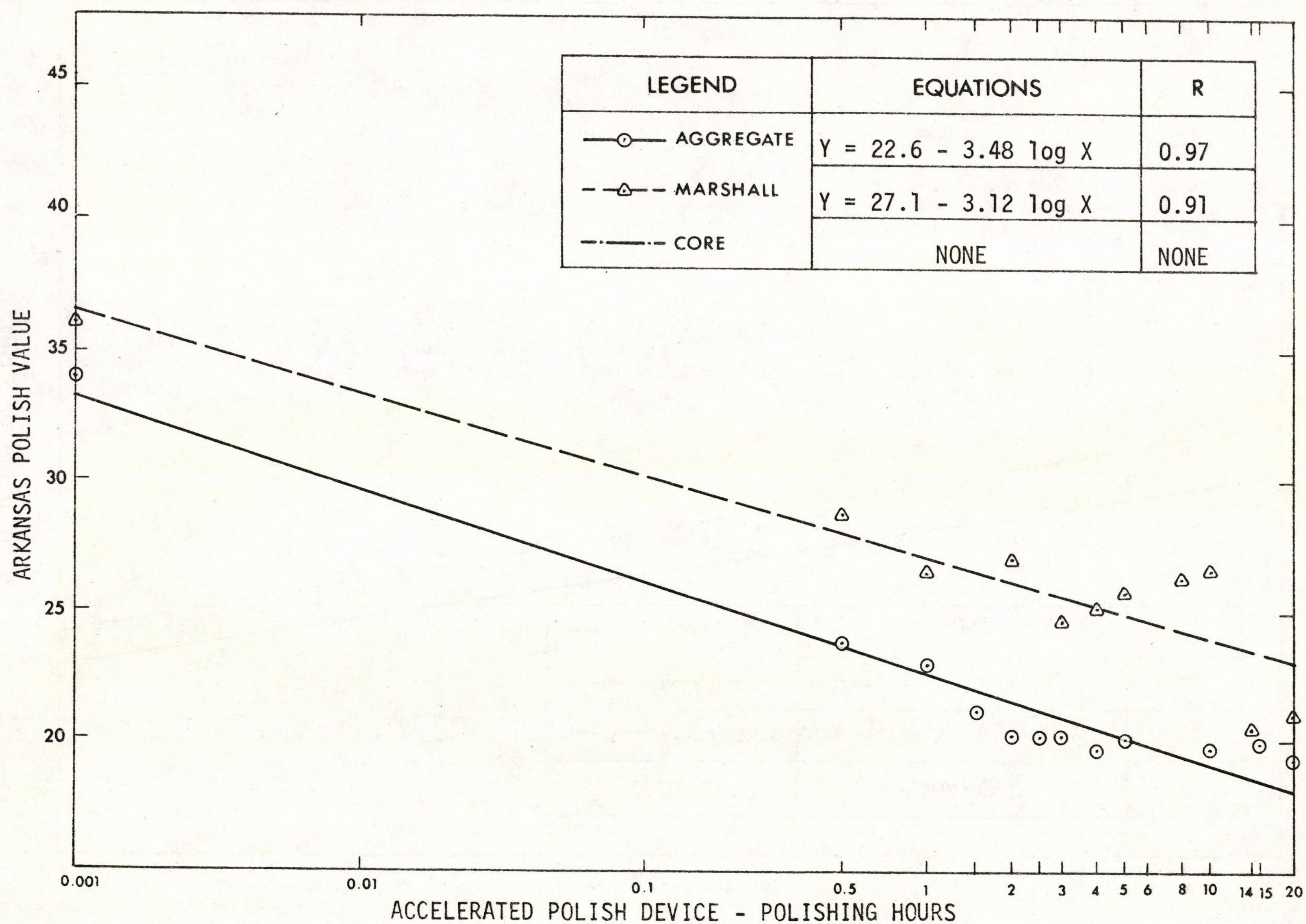


Figure E5. Arkansas Polish Value vs. Hours of Polish - Aggregate and Marshall - West Fork Limestone

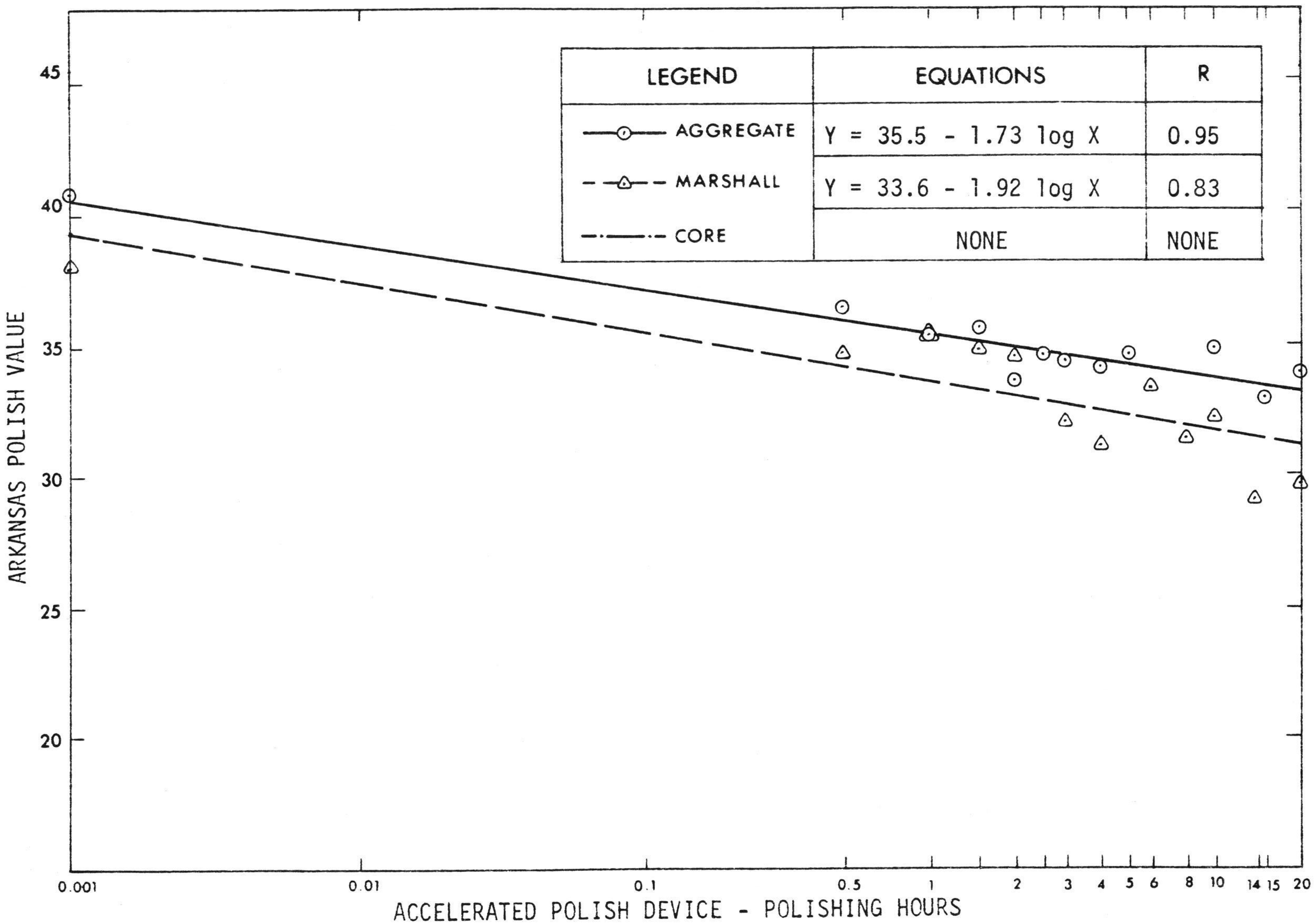


Figure E6. Arkansas Polish Value vs. Hours of Polish - Aggregate and Marshall - Russellville Sandstone

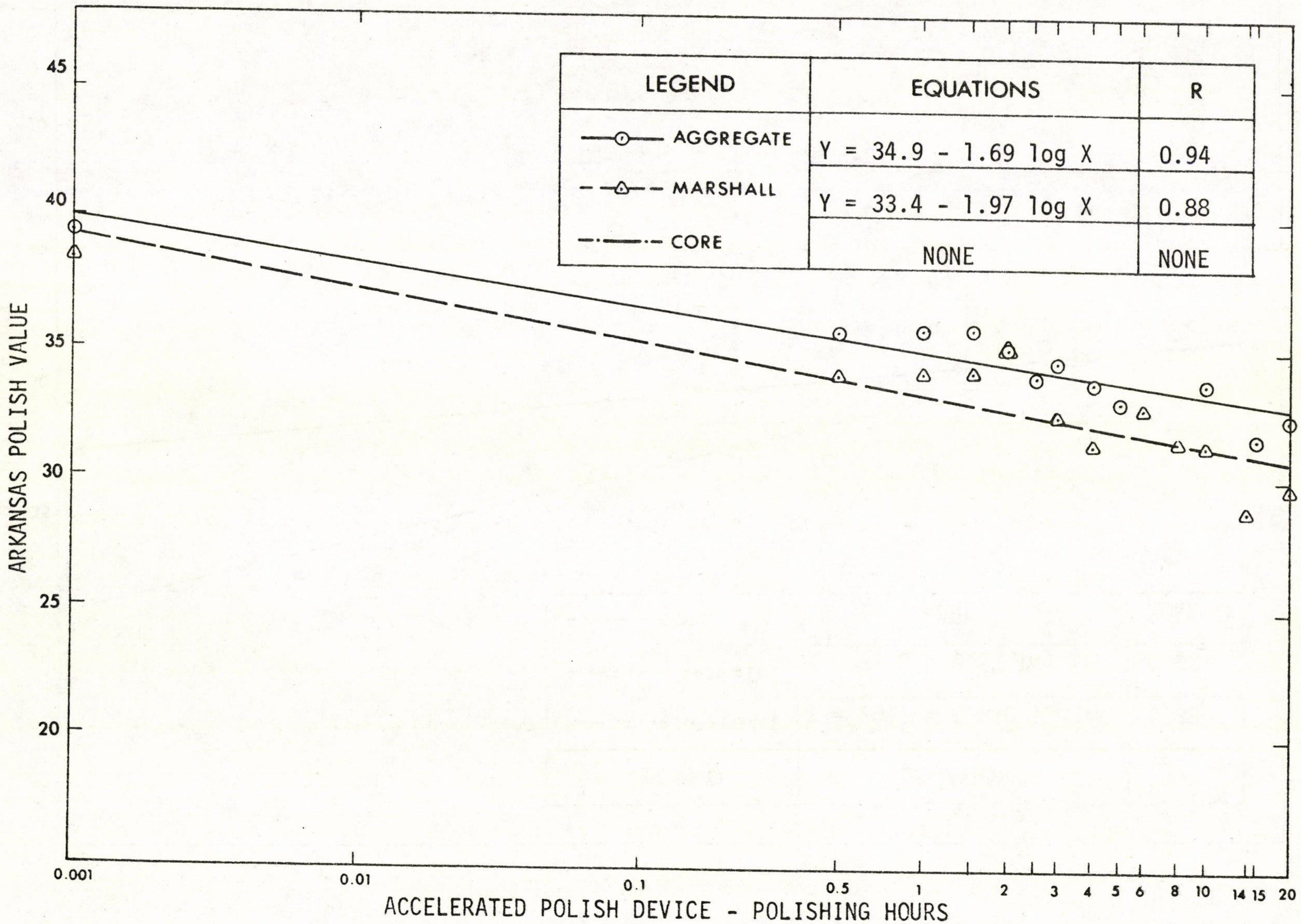


Figure E7. Arkansas Polish Value vs. Hours of Polish - Aggregate and Marshall - Bald Knob Sandstone

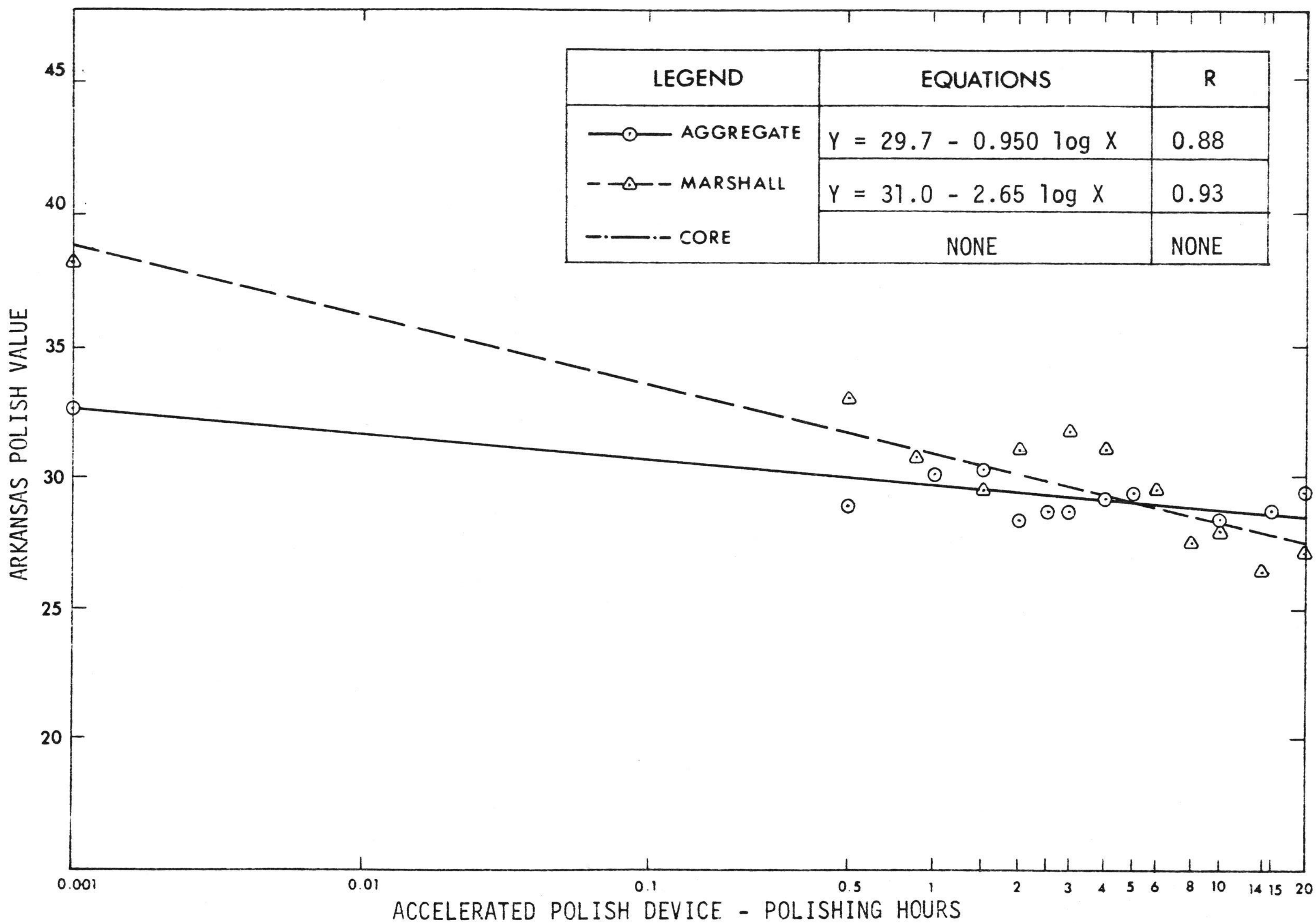


Figure E8. Arkansas Polish Value vs. Hours of Polish - Aggregate and Marshall - Hampton Gravel

APPENDIX F

Accelerated Polish Curves for
Blends - Marshall Specimens

Figures F1 - F15

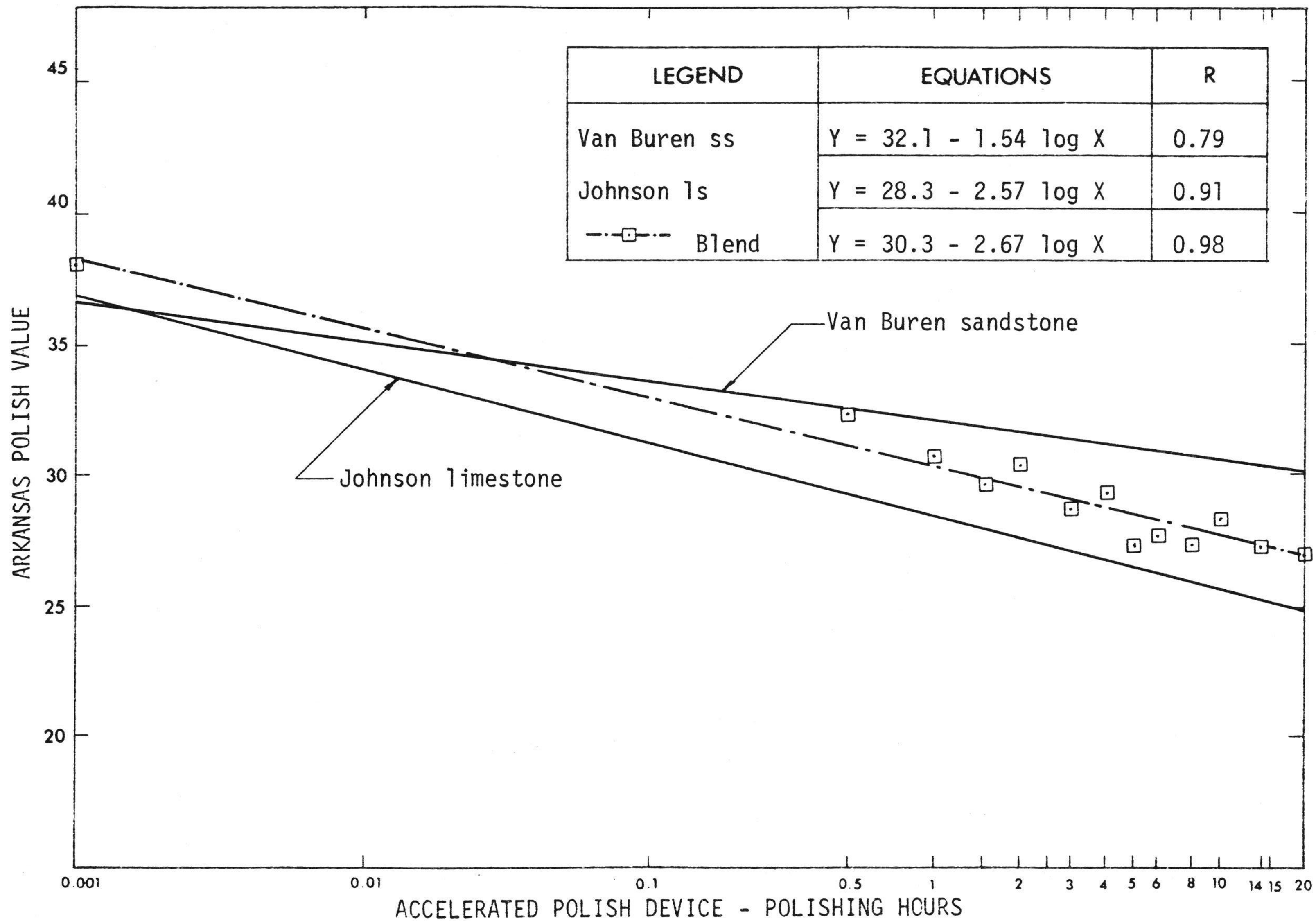


Figure F1. Arkansas Polish Value vs. Hours of Polish - Equal Blend - Johnson ls and Van Buren ss - [illegible]hall Specimen

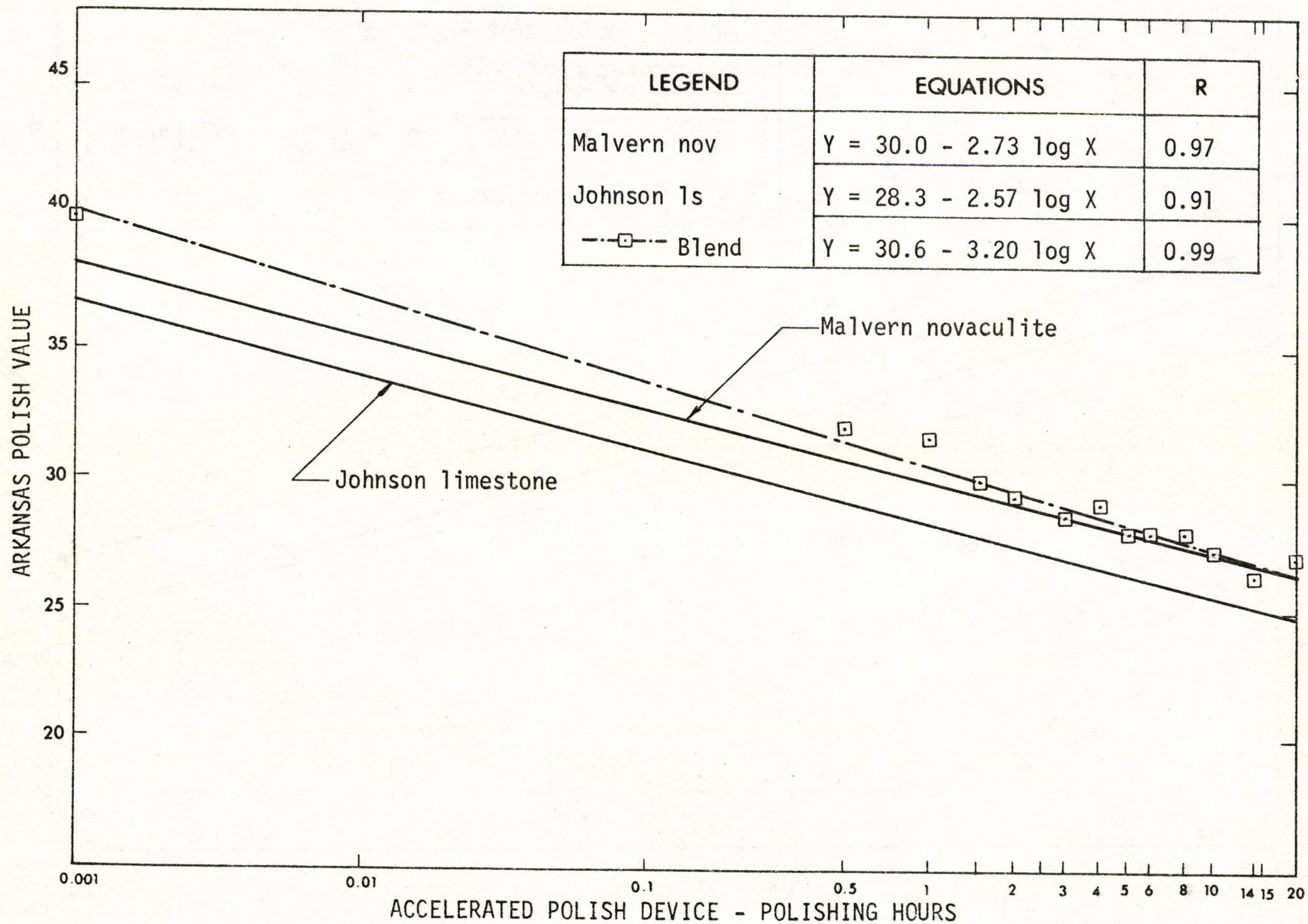


Figure F2. Arkansas Polish Value vs. Hours of Polish - Equal Blend - Johnson ls and Malvern nov - Marshall Specimen

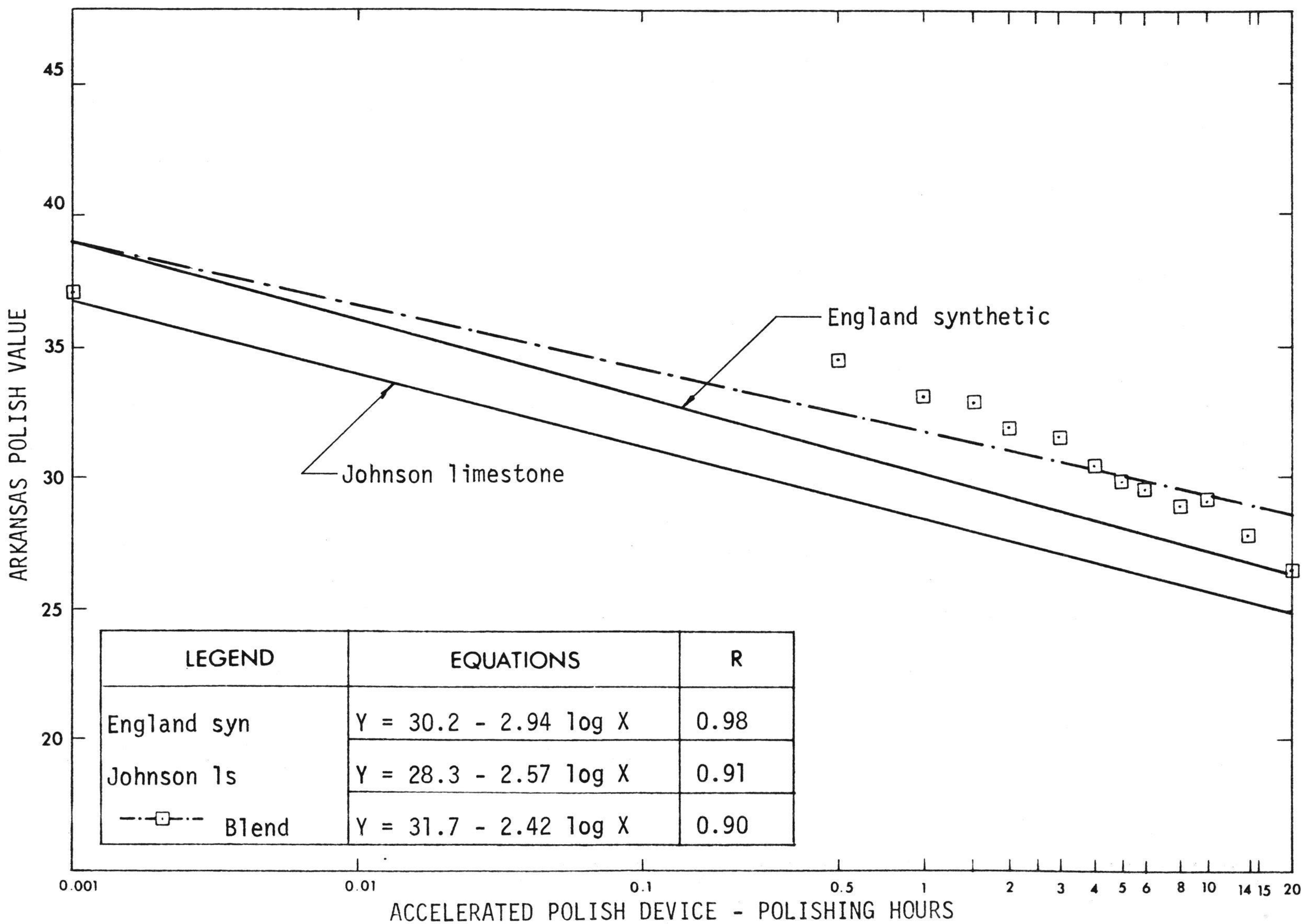


Figure F3. Arkansas Polish Value vs. Hours of Polish - Equal Blend (By Volume) - Johnson ls and England syn - M all Specimen

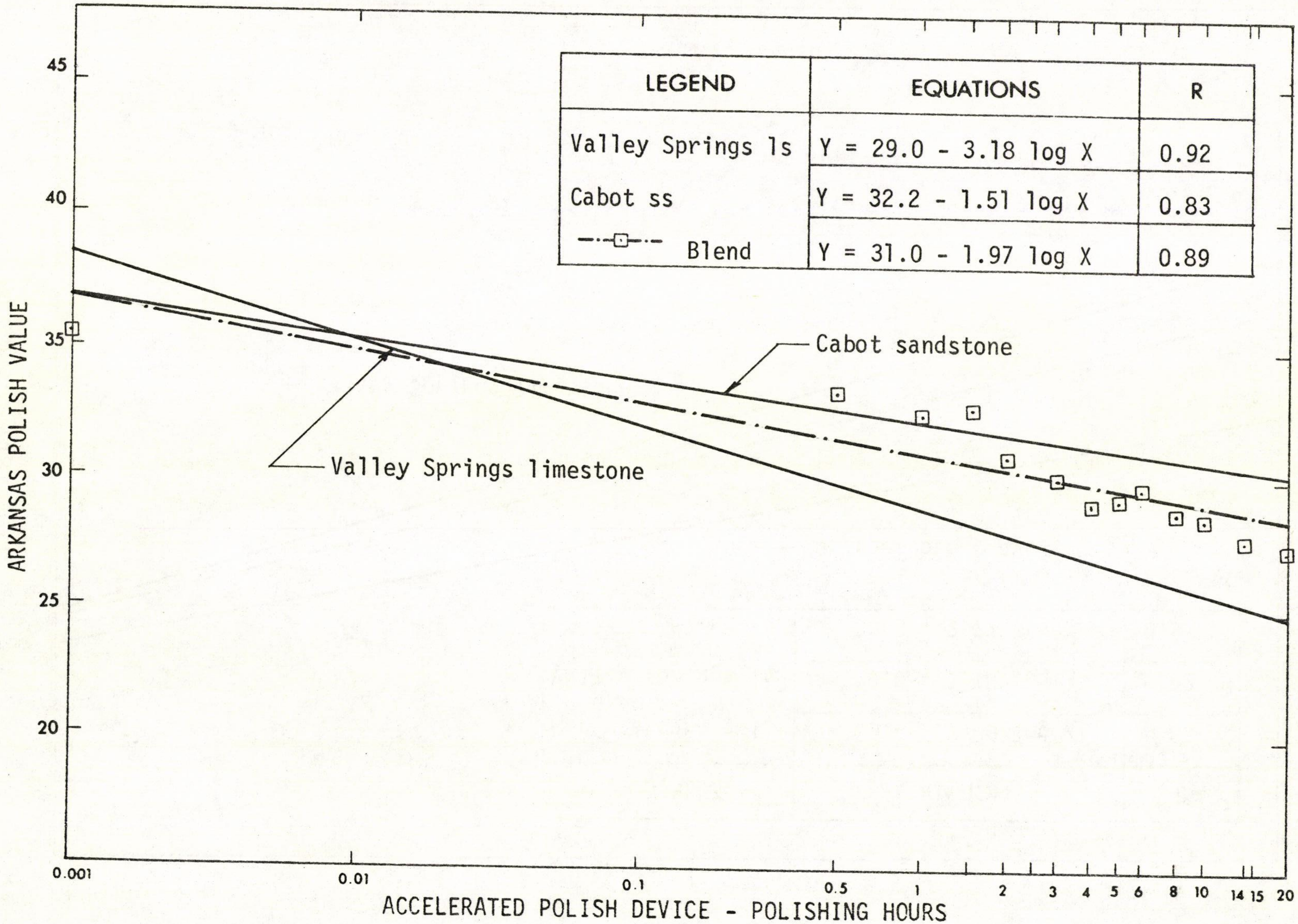


Figure F4. Arkansas Polish Value vs. Hours of Polish - Equal Blend - Valley Springs ls and Cabot ss - Marshall Specimen

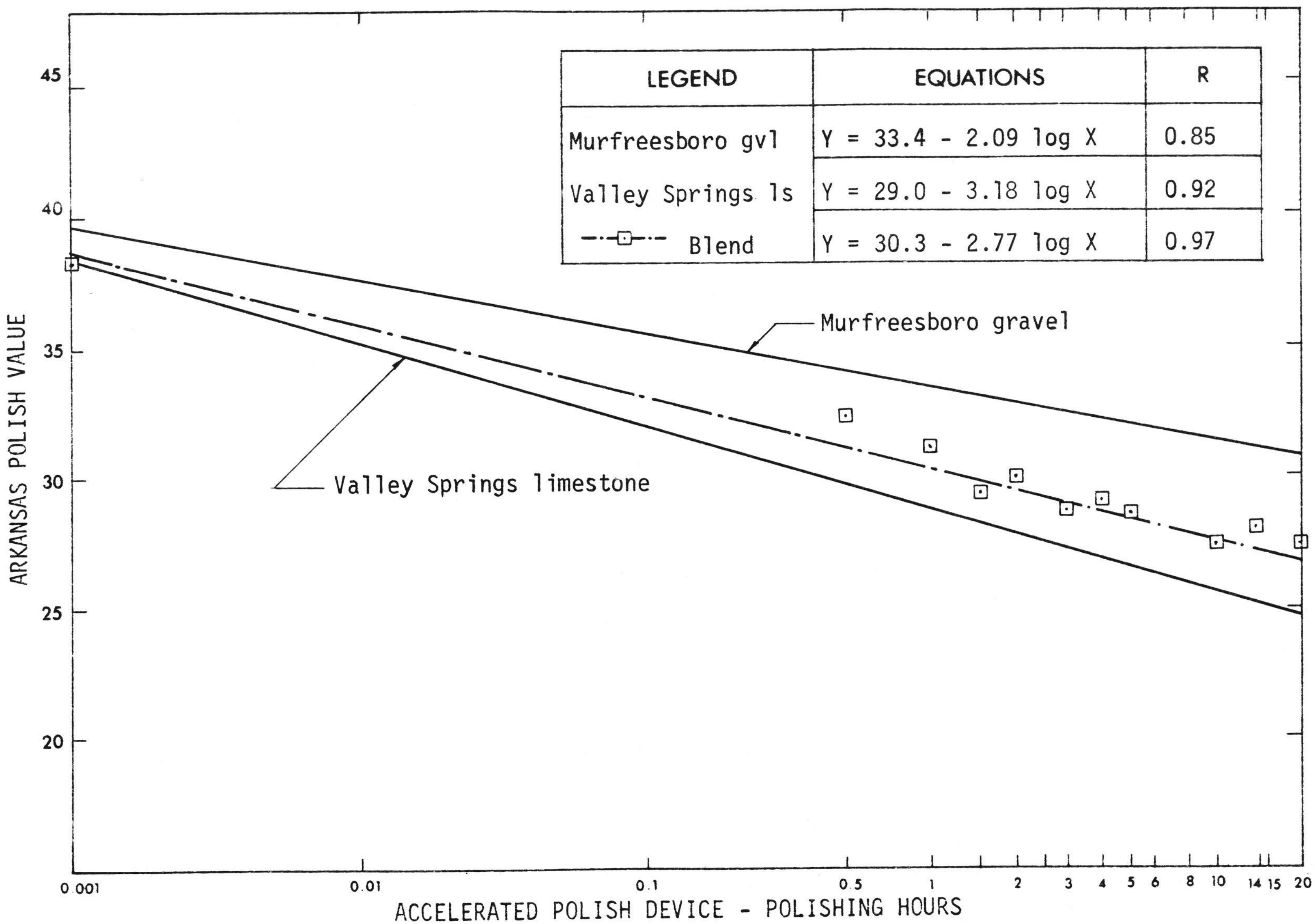


Figure F5. Arkansas Polish Value vs. Hours of Polish - Equal Blend - Valley Springs ls and Murfrees o gvl - Marshall Specimen

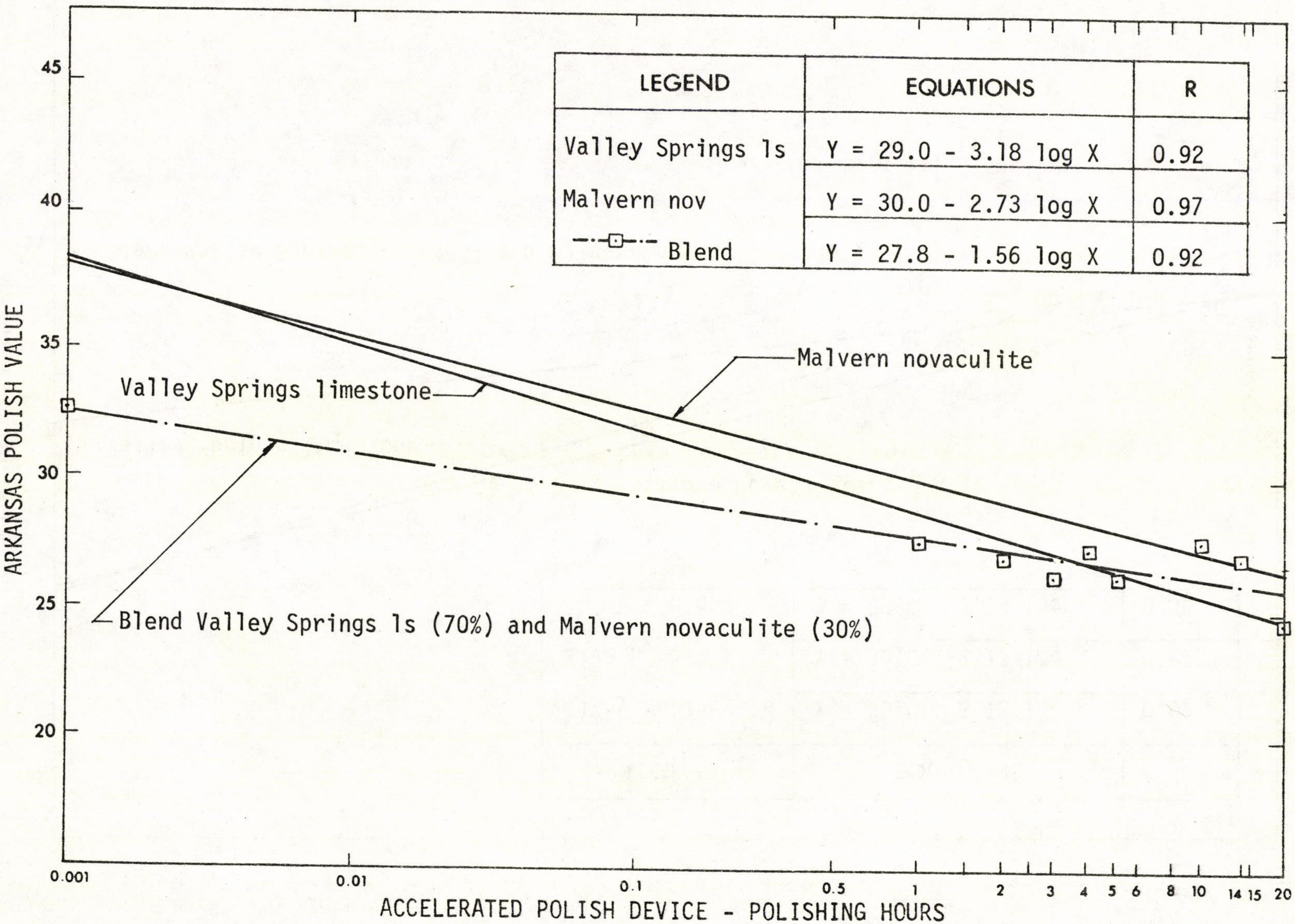


Figure F6. Arkansas Polish Value vs. Hours of Polish - Blend - 70 Percent Valley Springs ls and 30 Percent Malvern nov - Marshall Specimen

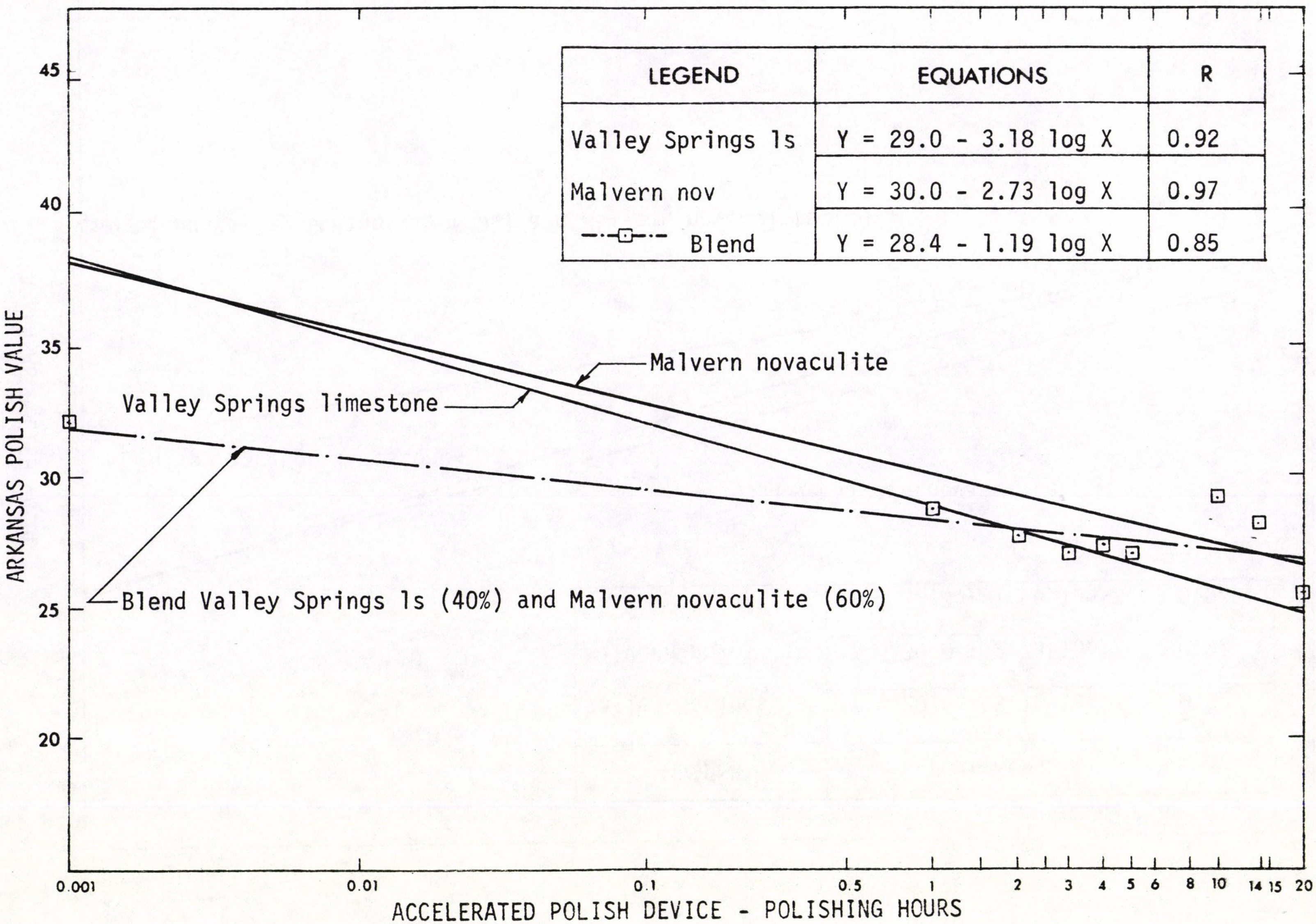


Figure F7. Arkansas Polish Value vs. Hours of Polish - Blend - 40 Percent Valley Springs ls and 60 Perce lalvern nov - Marshall Specimen

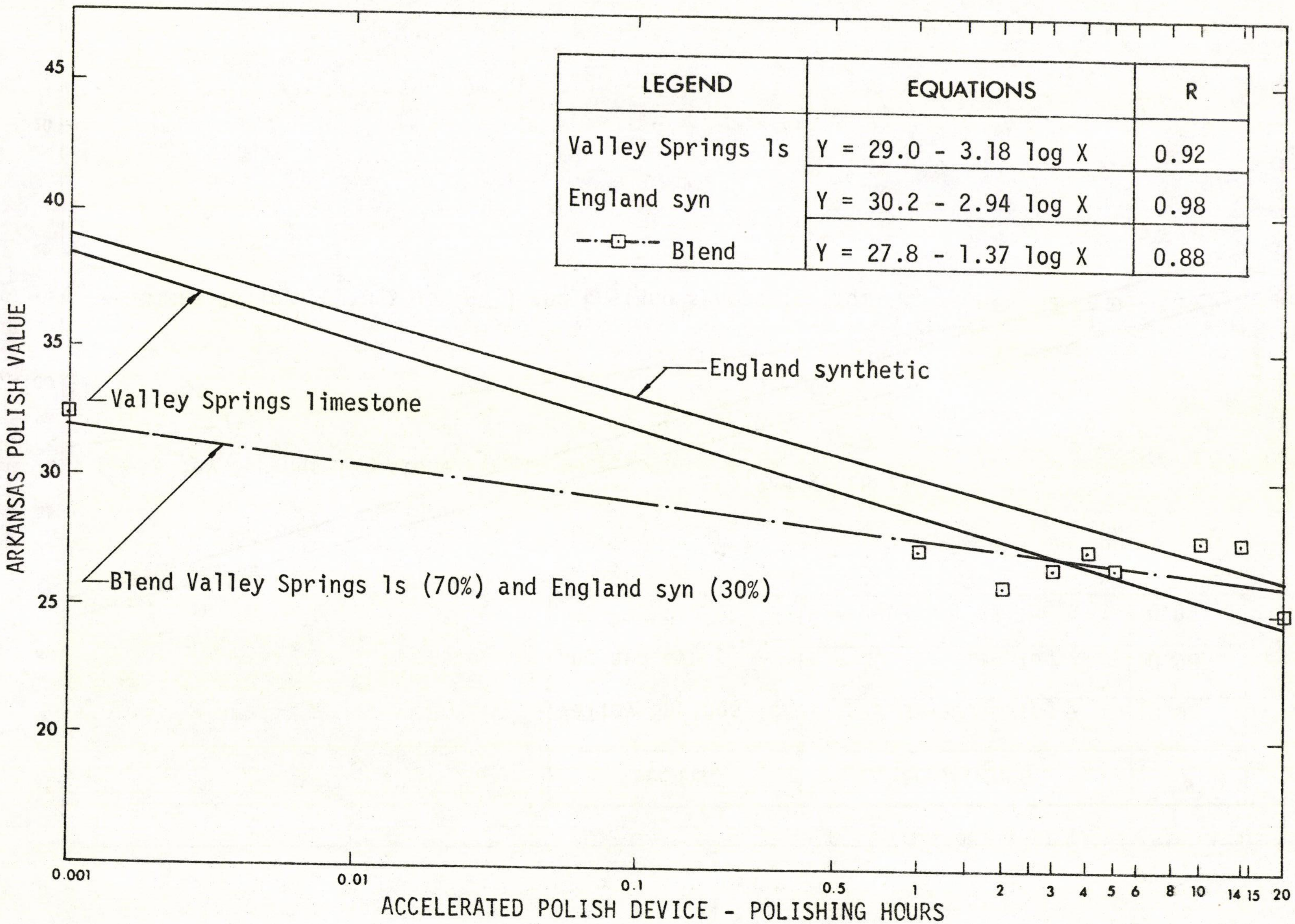


Figure F8. Arkansas Polish Value vs. Hours of Polish - Blend (By Volume) - 70 Percent Valley Springs ls and 30 Percent England syn - Marshall Specimen

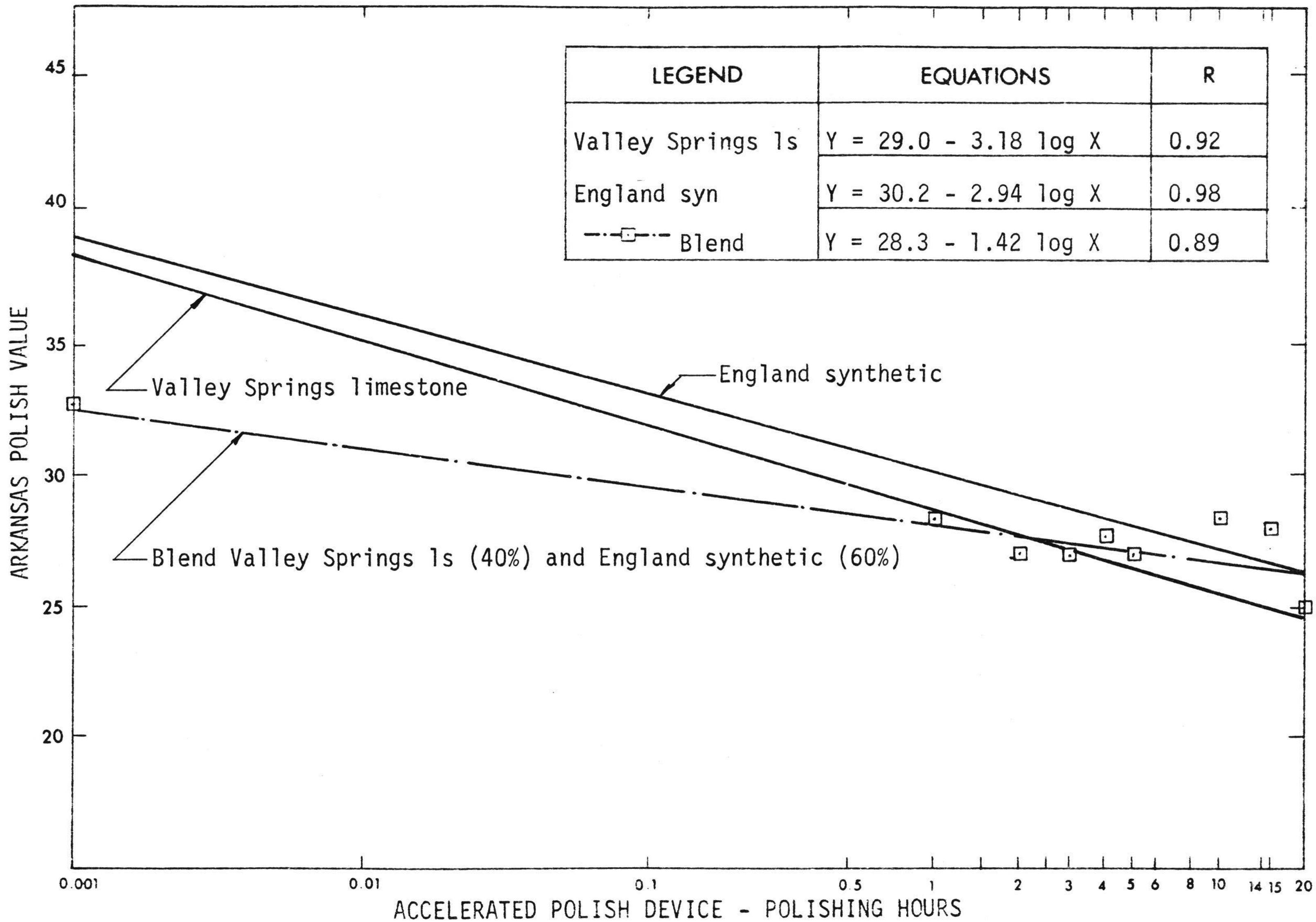


Figure F9. Arkansas Polish Value vs. Hours ^f Polish - Blend (By Volume) - 40 Percent Valley Springs ls and 60 Perce ngland syn - Marshall Specimen

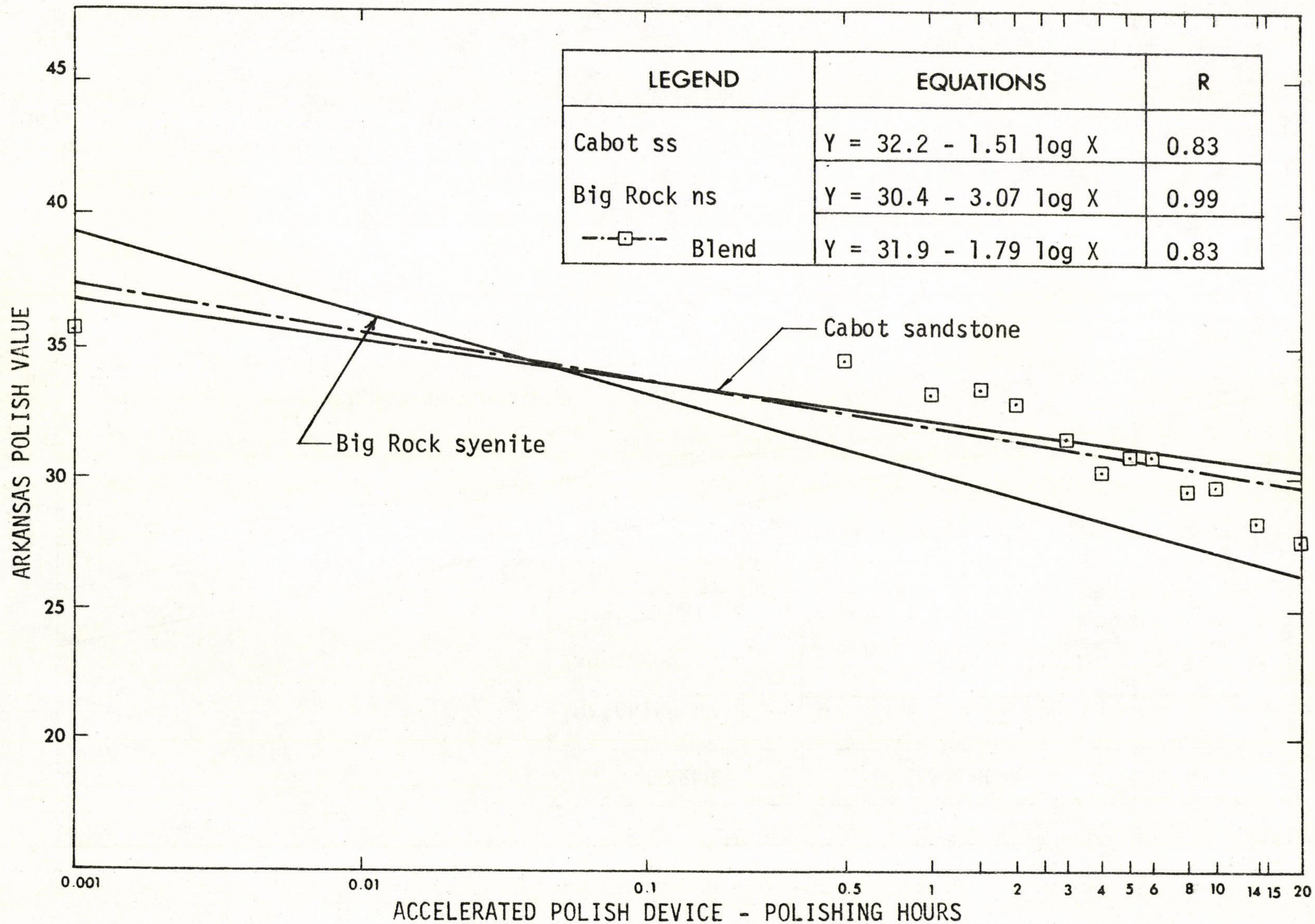


Figure F10. Arkansas Polish Values vs. Hours of Polish - Equal Blend - Cabot ss and Big Rock ns - Marshall Specimen

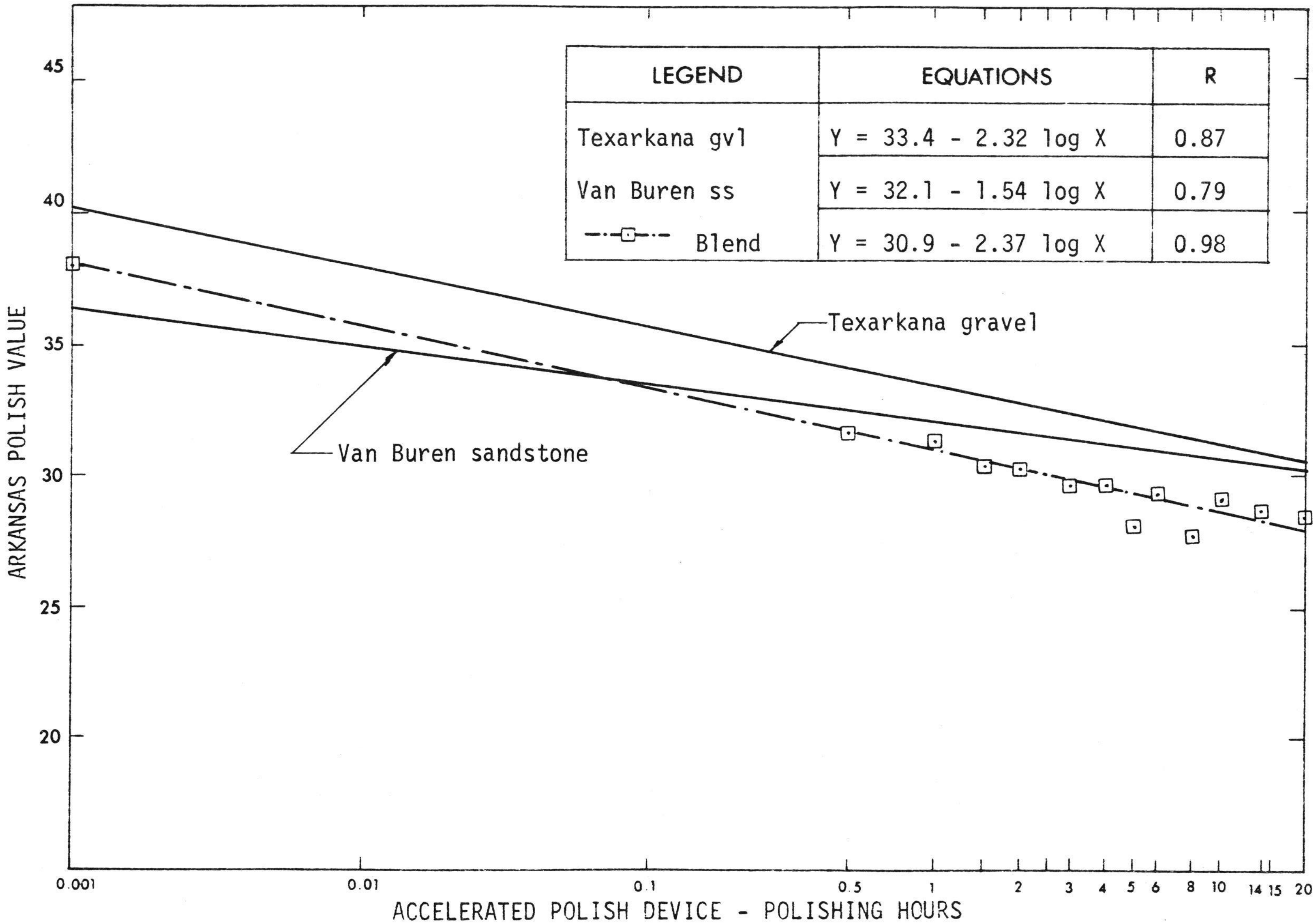


Figure F11. Arkansas Polish Value vs. Hours of Polish - Equal Blend - Van Buren ss and Texarkana gv Marshall Specimen

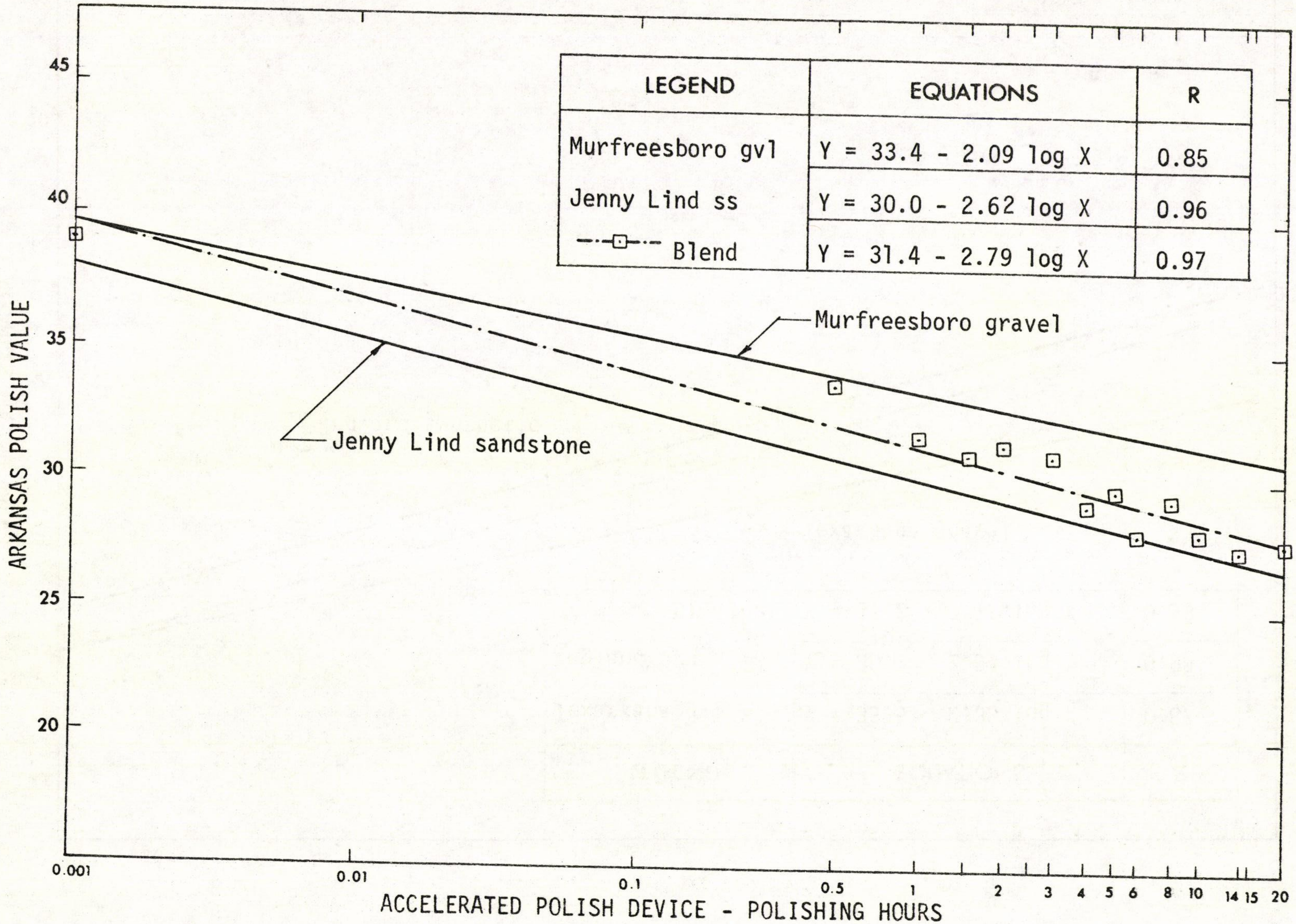


Figure F12. Arkansas Polish Value vs. Hours of Polish - Equal Blend - Jenny Lind ss and Murfreesboro gvl - Marshall Specimen

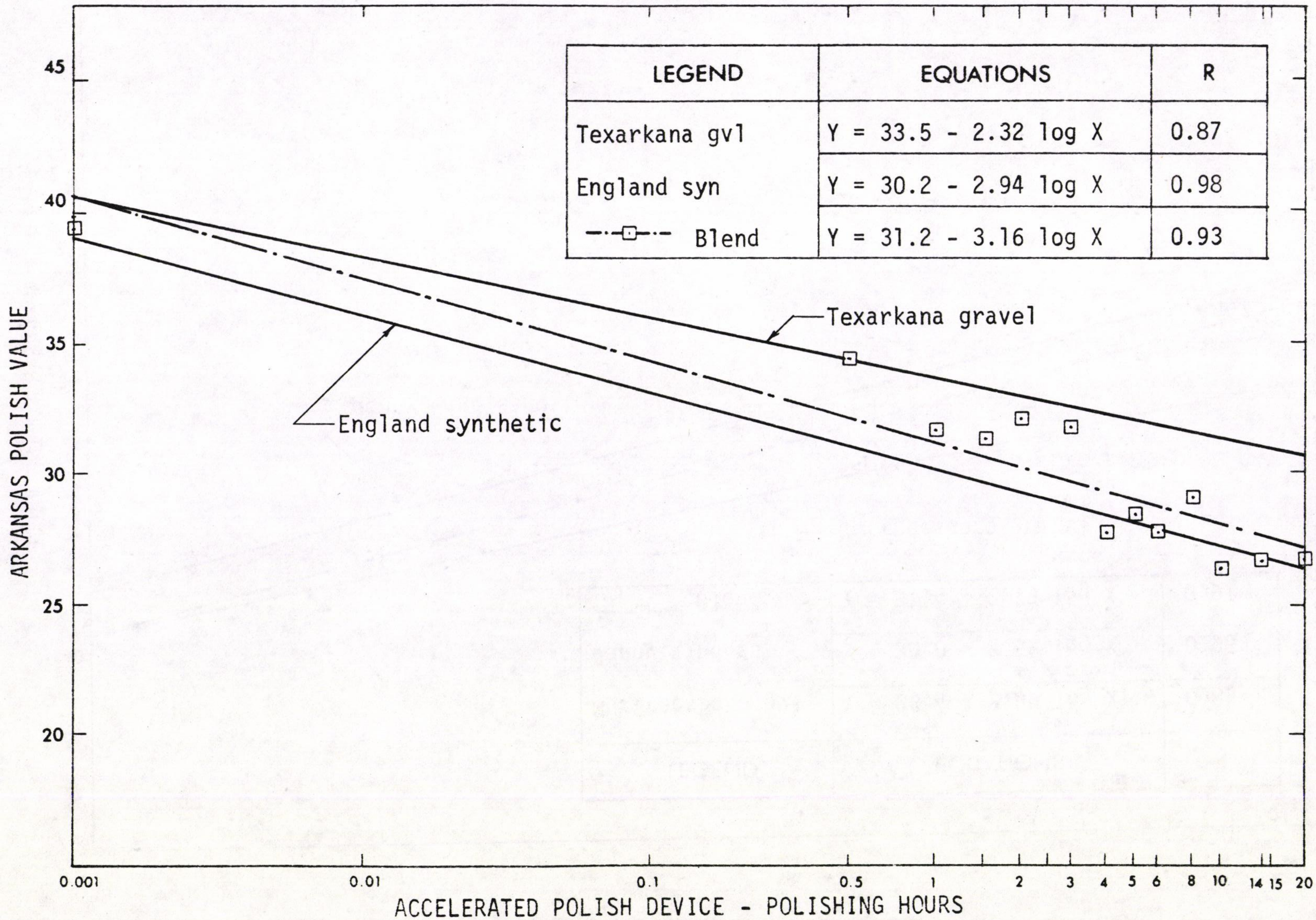


Figure F13. Arkansas Polish Value vs. Hours of Polish - Equal Blend (By Volume) - Texarkana gvl and England syn Marshall Specimen

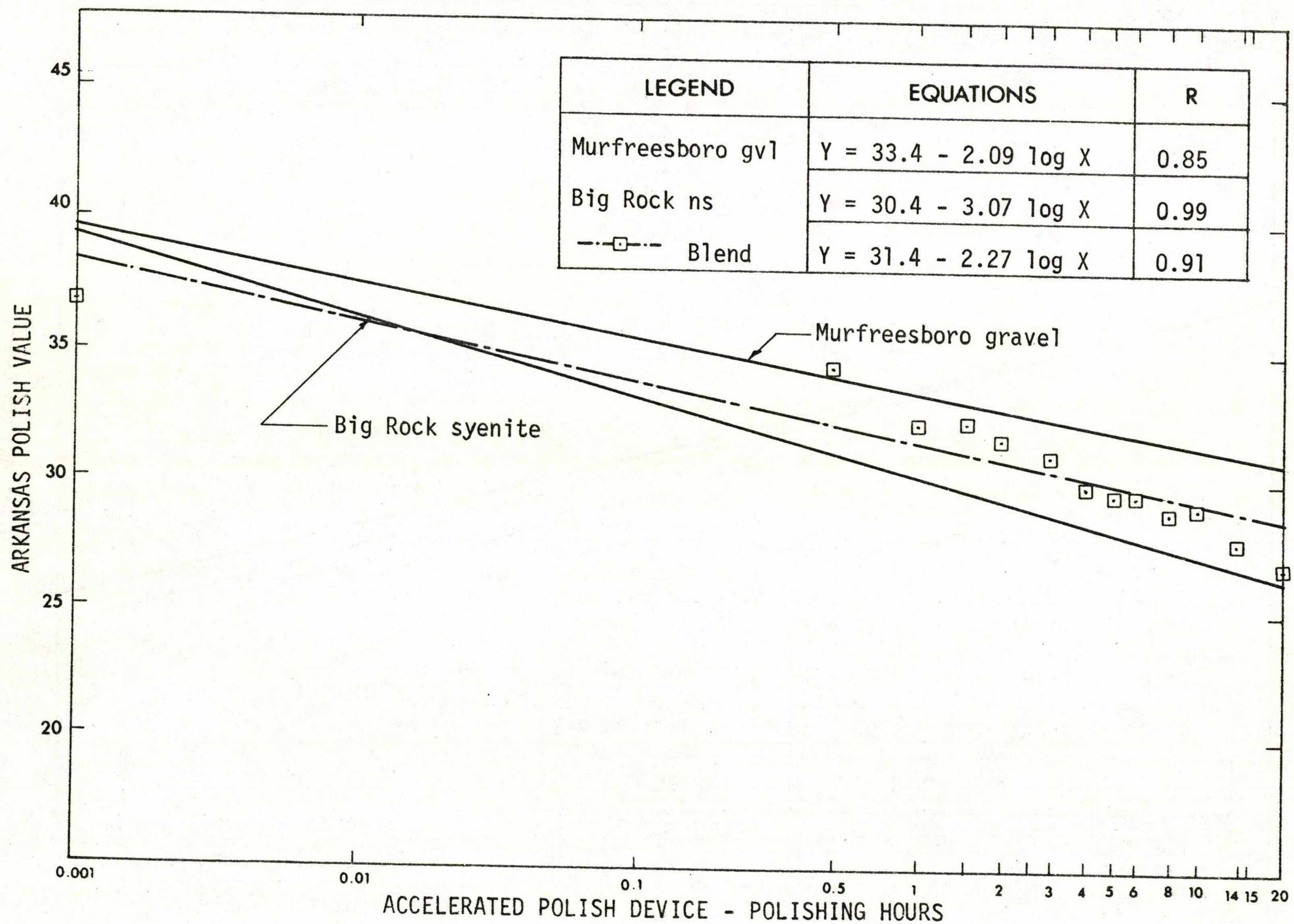


Figure F14. Arkansas Polish Value vs. Hours of Polish - Equal Blend - Murfreesboro gvl and Big Rock ns - Marshall Specimen

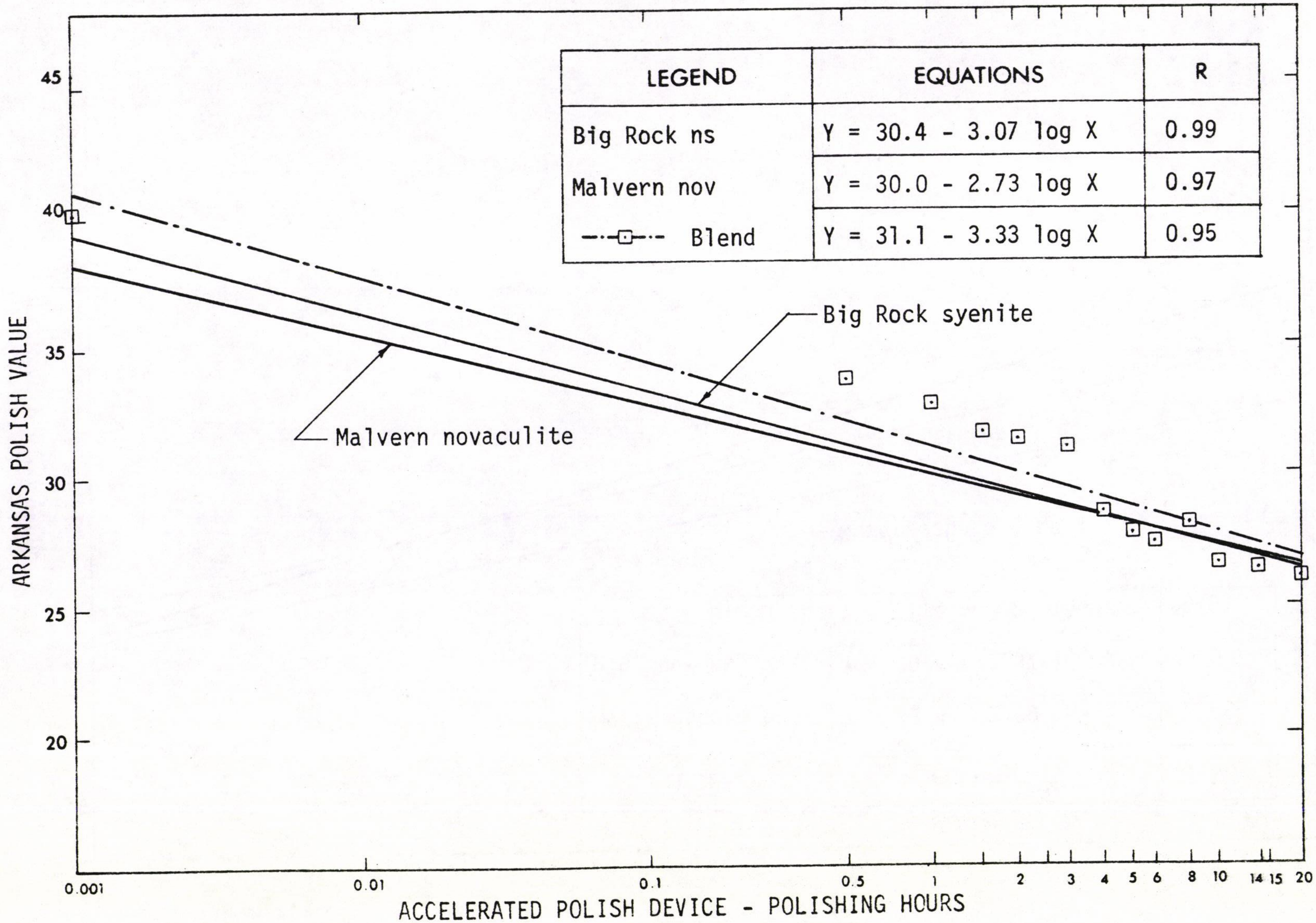


Figure F15. Arkansas Polish Value vs. Hours of Polish - Equal Blend - Big Rock ns and Malvern nov - [illegible]shall Specimen