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TRANSPORTATION RESEARCH PROJECT NO. 66

DEVELOPMENT OF A RATIONAL MIX DESIGN METHOD FOR ASPHALT BASES AND CHARACTERISTICS OF ARKANSAS ASPHALT MIXTURES

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University of Arkansas

Conducted for
The Arkansas State Highway and Transportation Department
In Cooperation With
The U.S. Department of Transportation
Federal Highway Administration

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16. Abstract

The relationships between asphalt pavement performance parameters and physical characteristics of pavement cores and Marshall laboratory mixtures were investigated. Thirty-six pavement sites from all areas of Arkansas were cored and evaluated. Eight pavement sites had conventional granular bases, 17 sites had asphalt bases and 11 sites had portland cement concrete bases. All of the pavements were high-type asphalt concrete with 12 foot lanes, sealed shoulders and with good drainage. Their ages ranged from 0.5 to 23 years.

Pavement field tests include: Dynaflect deflection, rut depth, crack classification, condition rating, and Mays Rideability rating. The cores were measured for layer thickness and sawed into layers. The core layers were tested for: resilient modulus, bulk specific gravity, maximum mixture specific gravity, Marshall stability and flow, asphalt content and aggregate gradation. Laboratory specimens having aggregate characteristics similiar to the pavement cores were molded with an AC-30 asphalt and likewise tested.

An excellent relationship was obtained between air voids, voids in the mineral aggregate and asphalt content for different gradations. Equations were developed that relate pavement performance parameters with physical properties of the pavements for different levels of traffic. The ACHM resilient modulus was used to develop equations that related the Marshall mix design values to pavement performance. The estimated change in pavement resilient modulus with time was determined. A rational method for mix design of Arkansas asphalt bases was developed.

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ABSTRACT

The relationship between asphalt pavement performance parameters and physical characteristics of pavement cores was investigated. Thirty-eight pavement sample sites from all areas of Arkansas were cored in 1982. Nine cores at each site were obtained. Cores from two sites were not satisfactory for testing. Eight pavement sites had conventional granular bases, 17 sites had asphalt bases, with 11 sites having portland cement concrete bases. All of the pavements investigated were high-type asphalt concrete with 12 foot lanes, sealed shoulders, and good drainage. Their ages ranged from 0.5 to 23 years. The types of mineral aggregate in the ACHM mixtures included: limestone, sandstone, gravel, syenite, and novaculite.

Pavement field tests included: Dynaflect deflection, rut depth, crack classification, pavement condition rating, skid number, and Mays Rideability rating. The Dynaflect test was performed at six points along the wheel paths and between the wheel paths at each site. Rut depths were measured in the wheel paths. Dynaflect and rut depth measurements were taken when the sites were cored in 1982 and were repeated in 1983.

The cores were measured for layer thickness and sawed into layers. The core layers were tested for: resilient modulus (77 F), bulk specific gravity, maximum mixture specific gravity, Marshall stability and flow, asphalt content and aggregate gradation. Laboratory molded specimens having aggregate characteristics simillar to the pavement cores were likewise tested.

An excellent relationship was obtained between air voids and voids filled with asphalt for the surface layer of the asphalt pavements. The excellent interrelationships obtained between air voids, voids in the mineral aggregate and asphalt content for different mixture gradations are presented. Equations were developed by regression analysis that relate pavement performance parameters with physical properties of the pavements for different levels of traffic. The common thread of ACHM resilient modulus was used to develop equations that relate the Marshall mix design values to pavement performance. The estimated change in pavement resilient modulus with time is presented.

The equations and analysis of data presented will enable the design engineer to analyze pavement mixtures designed by the Marshall method and to predict pavement performance. A rational method for mix design of Arkansas asphalt bases is presented.

GAINS, FINDINGS, AND CONCLUSIONS

The results of this investigation will enable the design engineer to analyze asphalt pavement mixtures designed by the Marshall method and predict their pavement performance for varying levels of traffic. A rational method of mix design of Arkansas asphalt bases was developed based upon the Marshall mix design procedure and equipment.

There is an identifiable relationship between the resilient modulus and other physical characteristics of the asphalt pavement and the Marshall laboratory mixtures. The pavement resilient modulus was estimated to increase with age at the rate of 30,000 psi per year.

The physical characteristics of the asphalt mixtures that best related to pavement performance were: stability, flow, air voids and voids filled with asphalt. Voids in the mineral aggregate, air voids and asphalt content are dependent upon one another; they may be analyzed by use of equations presented in this report.

Pavement cracking increases as air voids increase, whereas pavement rutting increases as air voids decrease. Pavements with high stability and between 2.5 and 5 percent air voids indicated moderate rutting and cracking.

Asphalt pavements with air void contents from 2 to 5 percent, stabilities of 1500 pounds or more and with 75 to 85 percent voids filled with asphalt indicated superior performance.

IMPLEMENTATION STATEMENT

The results of this study may be used to analyze Marshall designed asphalt mixtures for their potential pavement performance with varying levels of traffic. This analysis will permit the optimization of the mixture characteristics to provide increased pavement performance and longer service life.

The immediate practical application of the study findings would be to modify the Marshall laboratory job mix design criteria to insure that all proposed mixtures have the proper amount of asphalt cement and air voids that are in harmony with the VMA and voids filled with asphalt criteria as reported herein.

The base mix design procedure presented will provide excellent asphalt bases. This proposed mix design procedure should be evaluated by testing additional types and gradations of aggregate with suitable asphalt cements prior to implementation to verify the proposed mix design criteria.

The establishment of levels of criteria for mix designs for surface, binder and base courses with respect to low, medium and high traffic volumes may be implemented from the data of this investigation. The analysis of the data reported on the basis of type of construction will indicate if different controlling factors in the mix design should be used for each design type.

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CHAPTER I

INTRODUCTION

The performance of asphalt pavements has been studied for many years by various highway investigators, research agencies and highway engineers. Much has been learned about the behaviour of asphalt concrete pavements in regard to the effects of soil conditons, pavement structure, traffic and environment. However, some pavements are still failing to perform up to the expectations of the designers and the problem of designing and constructing long-lasting, smooth and safe pavements has not yet been completely resolved.

The objectives of the study are to evaluate Arkansas asphalt mixtures' physical characteristics, to correlate them with pavement performance under various traffic conditions and to develop a rational mix design for asphalt bases. This rational design will reflect the existing Arkansas State Highway and Transportation Department (AHTD) aggregate grading specifications with recommended design criteria.

The AHTD has used asphalt concrete hot mix for new construction and for reconstruction of old roads for many years. There has been some variation in the level of performance obtained from these asphalt pavements. The variations are considered to be the result of a number of variables, such as asphalt and aggregate characteristics, mixture stability, void content, traffic and environmental conditions. These variations of pavement performance have shown a need to relate laboratory and field tests of asphalt concrete and mineral aggregate physical properties and their mixture characteristics to field measurements that relate to pavement performance.

For many years the AHTD has been partially successful in using an asphalt stabilized base course. This mixture, having large aggregate (minus 1.5 in.) was not designed by the usual Marshall method of mix design, but contained an estimated asphalt content of between 3 and 4 percent. This low asphalt content held the base material together and provided a relatively high density material at a reasonable cost. Since there were no design procedures, the actual characteristics of this asphalt stabilized material have varied depending on the aggregate type, gradation of the mineral aggregate and asphalt content.

The increasing cost of asphalt cement and increasing traffic, both in frequency and magnitude of loads, warrants this study on development of a rational mix design method for asphalt bases and characteristics of asphalt mixtures. Data collected under this study will provide the background to establish levels of criteria for mix designs of surface, binder and base courses with respect to low, medium and high levels of traffic volume for Arkansas materials.

Research investigators have developed means of measuring fatigue and resiliency characteristics of asphalt mixtures, both from in-service and laboratory samples. Schmidt (1) reported a practical method for measuring the resilient modulus (Mr) of asphalt-treated mixes in 1972. This method of testing for modulus of resiliency is non-destructive and may be used to evaluate environmental and aging factors affecting asphalt mixtures.

A laboratory test system for predicting moisture induced damage to asphalt mixtures using the resilient modulus test has been reported by Lottman (12). An interim report by Lottman on a five-year field evaluation phase of this work indicates that pavements that were predicted (from laboratory tests) to have moderate to severe moisture damage are beginning to show an increasing trend toward loss of strength. The change in the resilient modulus of an asphalt mixture in service appears to correlate with pavement performance that is affected by the stripping effects of water.

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 pavement, base or subgrade material, are reflected in the

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(1) The number in parentheses corresponds to the listing of the literature cited in the Reference section.

pavement surface.

In regard to characteristics of asphalt concrete mixtures, it is concluded that the mineral aggregate and asphalt cement change their properties during construction and after being subjected to the effects of traffic and the environment. Therefore the measurement of the asphalt mixtures' properties, rather than the individual properties of the asphalt and aggregate, appears to be a rational approach toward the solution of the problem. The evaluation of the resilient modulus of the laboratory mixtures and pavement mixtures and their correlation with pavement performance would assist the highway engineer in designing and constructing more durable asphalt pavements.

This investigation of Arkansas asphalt pavements was designed to evaluate the in situ asphalt pavement mixtures' characteristics and relate them to the performance of the pavement. Thirty-eight sites were selected for investigation. The locations of these study sites were selected to provide pavements of varying ages, mineral aggregate compositions, traffic levels and types of design. The different types of design include: asphalt concrete over a granular base, asphalt concrete over portland cement concrete and asphalt concrete over black base. Several pavement sites were chosen because they exhibited distress such as rutting, flushing or cracking.

Pavement performance was evaluated by both quantitative and qualitative methods. Dynaflect measurements were taken to relate the structural adequacy of the existing pavement. Pavement condition evaluations were based on rut depths, degree of cracking and surface roughness. Laboratory tests performed on the pavement cores include: resilient modulus, Marshall stability and flow, bulk specific gravity, maximum specific gravity, asphalt content and extracted aggregate gradation. Laboratory molded Marshall specimens were made to simulate the condition of the initial asphalt mixtures for each generic type of aggregate. These aggregate types include: limestone, sandstone, gravel and syenite.

Relationships between pavement performance parameters and asphalt mixtures characteristics are included. A rational mix design method for asphalt bases is presented.

CHAPTER II

REVIEW OF LITERATURE

A combination of mineral aggregate and asphalt cement is used to make asphalt mixtures. Asphalt concrete pavement is an asphalt mixture consisting of the proper proportions of aggregate and asphalt cement that is heated, mixed, placed, and compacted while hot. The acronym for this mixture is ACHM, asphalt concrete hot mix. When ACHM is properly designed, manufactured, and placed on a well constructed road bed it provides an excellent pavement to serve the traveling public. The performance of ACHM pavement is dependent upon the many possible combinations of aggregates, asphalt cements, construction practices, road beds, traffic densities and environmental conditions. The characteristics of asphalt mixtures that relate to pavement performance include its stability, durability, and skid resistance.

Pavement Performance

The ability of a pavement to serve traffic is its performance. The methods of evaluating performance are quantitative, qualitative, or both, depending upon the view of the evaluator. Present methods of quantifying pavement performance are based upon the procedures devised for the AASHO Road Test (3) at Ottawa, Illinois. The performance measurements that may be taken include pavement deflection, rut depth, amount of cracking and patching, roughness, and skid resistance. The results of these field measurements may be combined to give a pavement condition rating (PCR) or a present serviceability index (PSI) to a pavement section. The weight to be given to each measured item in calculating PCR or PSI varies with each evaluator.

Qualitative measurement of pavement performance is in the eye and seat of the evaluator. Thus the qualitative pavement performance rating (PPR) results from the rideability and appearance of the pavement surface as traveled by the evaluator. The combination of PCR and PPR ratings in the proper proportions gives the best indication of pavement performance.

Pavement Durability Evaluation

The magnitudes of the limiting pavement deflection, rutting, cracking and roughness that indicate pavement failure are still being established. The durability of asphalt pavement surface has been defined as its resistance to change during service.

Vallerga (4) summarizes pavement deficiencies as related to asphalt cement durability. A classification system of types and causes of asphalt pavement failures was given. The three types of failures were disintegration, instability, and fracture or cracking. Hveem (5) reported the types and causes of failure of highway pavements in 1958. This classic work divided the types of distress into six groups: disintegration, cracking, instability, slippage cracks, deep grooves, and complete breakthrough. The cause of each distress along with excellent photographs illustrating the distress were included in this report.

Earlier, in 1948, Hveem and Carmany (6) reported the factors underlying the rational design of pavements. This report to the Highway Research Board presented a method of pavement design utilizing the Hveem Stabilometer, Cohesiometer, and Kneading Foot Compactor. Hveem (7) reported the importance of fatigue failures due to pavement deflection in 1955. This report presents graphical data relating axle loads to pavement deflection for seven different types of pavement structures. Hveem further reported that some comparisons were made between the Shell vibrator (an early Dynaflect device) and the Benkelman Beam. He also presented graphical relationships between pavement condition and deflections.

A 1962 report on the effect of resilience-deflection relationship on the structural design of asphaltic pavements was presented by Hveem et al (8) at the International Conference on the Structural Design of Asphalt Pavements. In this paper, a tabulation of tentative maximum pavement deflection for various thicknesses of pavement was presented. Beaton et al (9) reported a field application of the resilience design procedure of the California Highway Department that was developed by Hveem (6,7,8). Table I gives the tentative maximum tolerable pavement deflections that have been

TABLE I MAXIMUM TOLERABLE PAVEMENT DEFLECTIONS (9)

Pavement Thickness (inches)	Pavement Type	Tolerable Deflection (inches)
8	Portland Cement Concrete	0.012
6	Cement Treated Base (with ACHM surface)	0.012
4	AC (plant mixed) on aggregate base	0.017
3	same design	0.020
2	same design	0.025
1	AC (road mixed) on aggregate base	0.036
0.5	surface treatment	0.050

applied as a guide criteria for planning the reconstruction of existing roadways in California. This paper by Beaton et al also presents the variation in tolerable deflection of different types of pavements related to equivalent 5000 pound wheel loads. The relationships are linear when plotted on a log-log scale and are based on asphalt concrete fatigue tests. Of interest is the fact that the maximum pavement deflections shown in Table I are unchanged from Hveem's initial report of 1955.

Finn (10) reports the factors involved in the design of asphaltic pavement surfaces. He repeats Table I in his report and indicates the safe maximum deflection values are tentative. Finn reviews laboratory and field studies to evaluate fatigue properties and indicates that asphaltic concrete responds to repetitive loading in a manner similar to that found in elastic materials. He further observes that the stiffness modulus of the asphaltic concrete surfacing ranged from 450,000 to 1,300,000 psi on the AASHO Road Test and 670,000 to 1,220,000 psi on the California Designs (based on Hveem's deflection criteria). These values were calculated for a temperature of 40 to 50 F.

Williams and Lee (11) reported a load-deflection study of selected high-type flexible pavement in Maryland in 1958. In the discussion to this paper, W. H. Campen concluded that flexible pavement can tolerate about 0.05 inch total deflection without cracking and failing, which is contrary to the data of Hveem shown in Table I. In

Williams and Lee's closure, it was noted that Campen's conclusion could not be drawn from their data because of its limited nature.

Ford and Bissett (12) reported on flexible pavement performance studies in Arkansas in 1962. Benkelman beam deflection tests were performed using an 18,000 pound axle load on several types of pavement. The average pavement deflection for high-type pavements was 0.03 inch, with a range from 0.016 to 0.041 inch. The performance of the pavements investigated was found to correlate with the deflection ratio rather than with the absolute value of maximum pavement deflection. The deflection ratio was the maximum pavement deflection divided by the radius of influence of the wheel load.

A recent report by Way et al (13) indicates that the Spreadibility Index (SI) is used in the structural overlay design method for Arizona. The SI is determined from the pavement deflection as measured by the five Dynaflect sensors located at 1 foot intervals from the dynamic 1000 pound applied load. The SI is the average deflection of a pavement structure expressed as a percentage of the maximum deflection occurring under the wheel load.

The use of elastic layer theory and fatigue tests to predict pavement resistance to cracking and subsequent failure is well documented in the literature. The development and improvement of test equipment to measure the elastic characteristics of asphalt mixtures, such as the resilient modulus equipment reported by Schmidt (1), has facilitated this area. Meadors (14) has prepared a comprehensive review of elastic layer theory, resilience, fatigue, and deflection relationships in his thesis.

Asphalt Mixture Characteristics

The proper combination of different types and gradations of aggregate with varying quantities of asphalt cement to yield a satisfactory asphalt pavement is known as mix design. In general, current mix design methods can be divided into two groups, those using the Hveem method and those using the Marshall method (15). Either method yields a satisfactory job mix design; the Hveem method requires more equipment and testing time than the Marshall method. This discussion of asphalt mixture characteristics will be based upon the Marshall method, as it is the method used in Arkansas.

Marshall mix design parameters usually include aggregate gradation limits, stability, flow, air voids, voids in the mineral aggregate, and water susceptability criteria. The level of traffic determines the design criteria to be followed. For example, a heavily traveled highway may dictate a mixture with a minimum stability of 1700 pounds whereas a low traffic rural highway may only need a mixture with a stability of 1000 pounds.

The Marshall method of mix design is attributed to Mr. Bruce Marshall (16). The initial criteria for a satisfactory mix included the requirements for minimum stability, flow, and density (air voids). The air voids were calculated on the basis of apparent specific gravity (Ga) of the mineral aggregate. The importance of the voids in the mineral aggregate (VMA) was presented in the Marshall test manual (16). There has been a continuing evolution in the Marshall mix design criteria with each agency using their experience to obtain an asphalt mixture yielding good pavement performance.

Goode and Lufsey (17) reported the results of a study that included the relationship between air voids, film thickness and asphalt hardening. Marshall specimens were utilized in this work. The film thickness of asphalt coating the aggregate was calculated using the effective asphalt content of the mix and the aggregate surface area. They presented detailed procedures for calculating the surface area and film thickness. A definite trend for asphalt hardening to increase as the film thickness decreased and the air voids increased was noted. An asphalt mixture having film thickness of 6 microns and air voids of 4 to 5 percent showed good resistance to hardening. Earlier work by Campen et al (18) indicated a film thickness of from 6 to 8 microns produced the most desirable pavement mixture. In a later report, Campen et al (19) indicated that the proper asphalt content for open graded mixtures should be determined by considering film thickness and surface area. A film thickness of 10 microns for their open graded mixture was recommended.

One area of controversy that has not been resolved is the percentage of air voids (AV) needed or permitted in an asphalt pavement and how these air voids are to be determined. The maximum theoretical specific gravity (Gmt) of asphalt mixture may be calculated based on four different aggregate specific gravities: apparent; bulk (Gb); bulk saturated surface dry (Gbssd); and bulk impregnated (Gbi). McLeod (20,21) gives a very complete discussion of AV and VMA determination and their requirements for asphalt mixtures used for paving. These requirements are similar to those shown in the Asphalt Institute MS-2 (15). Since McLeod's work, the determination of Gmt has been standardized by ASTM as Method D 2041 (22). An initial description of this test, attributed to James M. Rice, was contained in a fundamentals for design of bituminous paving mixtures report (23). The effective aggregate specific gravity (Ge) may be calculated from the Gmt of ASTM Method D 2041 if the amount and specific gravity of the asphalt cement is known. Therefore, there are 5 different aggregate specific gravities that may be used to analyze the voids in an asphalt mixture. Of interest perhaps is that the initial voids recommended by Marshall (16), based on Ga, are very close to the present-day voids requirement published by the Asphalt Institute (15) that are based on Ge.

The area of water susceptibility of Arkansas asphalt mixtures has been evaluated and reported by Ford (24).

This study reported the film stripping and immersion compression relationship of 18 Arkansas aggregates and asphalt mixtures. It was shown that compacted asphalt mixtures with air voids greater than 5 percent tended to have a retained strength of 75 percent or less. Schmidt and Graf (25) reported the use of the resilient modulus test to evaluate the effect of water on asphalt mixtures. Unlike the immersion compression test, the resilient modulus test is non-destructive and specimens may be evaluated periodically to determine the long-term effects of water on compacted asphalt mixtures.

Base Course Design

A standard method for the mix design of an asphalt base course (ABC) is not found in the review of literature. A problem in this respect has been the development of a laboratory compaction device to provide compacted asphalt mixtures of a size suitable for testing that will have the same structure as that developed in an asphalt pavement under rolling and traffic. The Marshall method of mix design uses a 4 in. diameter by 2.5 in. thick specimen for stability and flow tests on surface and binder mixes. However, an ABC mixture contains aggregate up to 1.5 in. in diameter, and therefore a 6.0 in. specimen diameter is needed, since the specimen diameter should be equal to 1.5 times the maximum particle size in order to meet standard ASTM test criteria.

In 1961 Ellison (26) reported on the construction and performance of asphalt bases using aggregate up to 2.0 in. in diameter built in Virginia about 1950. The asphalt content was determined by putting in as much asphalt as the mix would hold. Ellison noted that stability tests were not conducted on the ABC mixes because the mix had such large aggregate that they could not duplicate stability test results. Warden and Hudson (27) reported on an ABC construction using a sand-gravel material with a maximum size of 1.5 in. The Marshall mix design method was used to determine the physical properties of the ABC. They reported an average stability of 1120 lb. with a flow of 8 at an asphalt content of 5.2 percent.

An investigation to determine the feasibility of using larger-sized aggregate for ABC mixtures was reported by Khalifa and Herrin (28). They prepared and tested mixtures using aggregate up to 2.5 in. in diameter. Their test specimens were cored out of large slabs of asphaltic concrete and tested in triaxial compression using a Texas triaxial cell. It was concluded that increasing the aggregate size caused a reduction in the percentage AV and VMA, and a more economical asphalt mixture because of a reduction in the required amount of asphalt.

McDowell and Smith (29) presented a comprehensive procedure for design and control of ABC mixtures in Texas. Their test specimens were molded in a gyratory device and were 6 in. in diameter and 8 in. high. The specimens were tested in unconfined compression and the criteria for selection of a mixture were based on the air voids to asphalt ratio along with the anticipated traffic.

Barksdale (30) reported the results of the application of fatigue and rutting tests on ABC mixes. The aggregate used in the ABC mixtures were a maximum of 1.5 in. The mix designs investigated were prepared by the Georgia DOT. An optimum asphalt content of 4.8 percent resulted in a Marshall stability of 2150 lbs. with a flow of 13. This mixture had air voids of 4.5 percent with a VMA of 14.9 percent. Barksdale noted that rutting was directly related to asphalt content and that a 6/32 to 8/32 in. rut depth would occur with these ABC mixtures. The actual fatigue and rutting beams of ABC mixtures tested by Barksdale were prepared using a kneading type compactor.

The ABC mixtures used as part of the AASHO Road Test (31) experiment were designed by the Marshall method. The aggregate used was minus 1.0 in. in size. These ABC mixtures had a Marshall stability of 1650 lbs. with a flow of 10 and air voids of 6.2 percent at an asphalt content of 4.8 percent. The AASHO ABC meets the requirement for an Arkansas Type 2 Binder Course.

<u>ACHM Design Developments and Criteria</u>

The criteria for asphalt pavement design continues to change as more information on pavement mix characteristics and performance becomes available. The highway design and test engineer now has available precise testing equipment and computer methods to facilitate the evaluation and analysis of asphalt pavement materials and their performance.

For example, the pavement structure may now be evaluated with the Dynaflect device much faster than with the Benkelman beam. The operation of the Dynaflect device was reported by Scrivner et al (32) where it was described as a new tool for measuring pavement deflection. Tenison (33) reports the use of a device similar to the Dynaflect called a Road Rater to measure pavement deflections under a dynamic 1800 lb. load in New Mexico. The results of these deflection measurements were used to predict the remaining life of the existing pavement and the required overlay thickness for a given design period.

Way (13) used the deflection results obtained with a Dynaflect device and the pavement roughness measured with a Mays Ridemeter to devise a structural overlay design method for Arizona. The loss of serviceability in flexible pavements due to rutting and cracking was evaluated by Rauhut (34). He reported the use of a calibrated mechanistic model (VESYS III-B) computer program to predict pavement damage functions for rutting and fatigue cracking. The computer program was calibrated using test data that included a condition survey, materials characterization from core samples, resilient modulus of pavement and subgrade, and axle loads.

AASHTO (35) gives guidelines for design of pavement structures. This manual also presents typical criteria for design of asphalt mixtures. The desired properties of the asphalt mixtures are based upon the level of traffic for a 20 year traffic analysis period. The three levels of traffic, based on an equivalent daily 18 kip axle load, are: 1 to 50, 50 to 500, and 500 to 3000. The compactive effort used in the Marshall method of design for these levels of traffic are 35 blow, 50 blow, and 75 blow to each end of the test specimen. The AASHTO manual recommends design values of Marshall stability and flow, total voids and voids filled for surface, binder and base mixtures. Of interest is that no criteria is given for VMA in these mixtures.

Current mix design procedures were assessed by Finn et al (36). This report presented two case studies where pavements designed in accordance with the Marshall procedure had experienced premature failure by rutting and cracking. Finn et al investigated the failures and performed Hveem stability tests and a creep test to modify the mix designs to obtain a more durable pavement. Their creep test was performed on 4 in. diameter by 8 in. high specimens with an MTS device to estimate permanent pavement deformation. The creep test results yielded a creep modulus, which was used to predict an acceptable asphalt content for the asphalt mixtures. N. W. McLeod, in his discussion to this report, indicated that, in his experience, most cases where rutting has occurred it has been caused by a combination of very low percent air voids and a high Marshall flow index. McLeod also said that "the Marshall flow index has for a very long time been a very effective creep test."

Summary

Much has been learned about the behaviour of asphalt pavement under traffic from the many intensive research investigations conducted over the last 50 years. The continuing effort of most research work is to establish the true relationship between laboratory test results of asphalt mixtures and their performance in pavements under field conditions. However, the laboratory investigations and resulting job mix design to predict the performance of a pavement to serve traffic for 10 to 30 years has not been perfected.

CHAPTER III

PAVEMENT TEST SITES AND TEST METHODS

The pavement test sites were selected to represent the various types of asphalt concrete hot mix (ACHM) pavement that have been constructed during the past 25 years in Arkansas. The various types of pavement are divided into three groups, based upon the type of base material involved: ACHM placed over a granular base (type X), ACHM placed over an asphalt base (type Y), and ACHM placed over portland cement concrete (type Z). Other criteria followed in choosing the test sites included: traffic, type of mineral aggregate, service age, and pavement condition. Six sites were also selected to coincide with the long term monitoring program undertaken by the Pavement Management Section of the Arkansas State Highway and Transportation Department.

Thirty-eight pavement sites were selected for evaluation. In general, the pavement lanes were 12 feet wide with sealed shoulders and good drainage. The sites were usually on tangents with level grades and good sight distance to permit safe field operations. The types of mineral aggregate and their number in the surface and binder layers, respectively, were: limestone (LS),10; sandstone (SS), 7; gravel (GVL), 5; syenite (NS), 11; and novaculite (NOV), 5. The types of mineral aggregate and their number in the asphalt base layer, respectively, were: LS (5), SS (4), GVL (7), NS (5) and NOV (0).

The pavement surface age at the time of coring ranged from 0.5 years to 22.7 years. The daily traffic ranged from 1940 vpd to 26,200 vpd, with truck traffic from 5 percent to 37 percent. The number of total accumulated 18 kip single axle equivalent (EAL) loads ranged from 130,000 to 3,100,000 passes.

Field evaluation of the pavement test site included coring, dynaflect measurements, rut depth measurement, and visual estimation of pavement conditions. In addition, the pavement roughness and skid number were determined in the vicinity of each test site.

Laboratory tests of pavement cores included layer thickness, bulk density, resilient modulus, maximum mixture specific gravity, Marshall stability and flow, asphalt content and gradation. A description of the specific test methods employed along with the identification of the pavement test sites are given below.

Pavement Sites and Field Tests

The location of each test site by route-section, county, direction, lane, log mile, job number and date of construction is given in Table II. A two lane road is indicated when no lane is given. Twenty-two sites were on two lane roads, and sixteen test sites were on four lane roads.

Test sites were chosen on both the inner and outer

TABLE II TEST SITE IDENTIFICATION

Site No.	Route-Section	County	Direction	Lane	Log Mile	Job No.	Date Const.
1	65-15	Jefferson	North	IL	4.6	2798	8-78
2	65-15	Jefferson	North	OL	4.1	2798	8-78
3*	71-17	Washington	North	OL	4.5	9563	11-71
4	71-17	Washington	South	OL	4.5	4827	9-81
5	71-16B	Washington	North	OL	3.2	4698	5-77
6	71-16B	Washington	North	IL	3.2	4698	5-77
7&	71-16	Washington	South	-	5.2	9432	12-66
8#	71-19	Benton	South	OL	8.4	9579	11-78
9	271-1	Sebastian	North	OL	1.7	4491	11-69
10*	71-13	Sebastian	North	-	8.1	4547	5-71
11*	71-14	Sebastian	North	-	4.4	4-687	4-73
12&	22-3	Logan	East	-	14.7	4473	10-64
13	65-12A	Saline	South	OL	0.3	6779	9-63
14	65-12A	Saline	South	IL	0.3	6779	9-63
15	167-12	Saline	South	-	4.0	6782	5-64
16*	70-9	Garland	East	-	10.0	6-604	7-71
17#	30-22	Saline	South	OL	120.5	6997	4-78
18#	30-12	Hempstead	South	OL	26.9	3854	4-81
19*	71-1	Miller	North	-	6.5	3655	10-75
20*	71-2	Miller	North	-	2.3	3623	8-72
21*	82-2	Lafayette	West	-	3.8	7819	7-79
22	7-2	Union	North	OL	7.9	7710	6-77
23	167-2	Union	South	-	5.7	7845	4-80

TABLE II TEST SITE IDENTIFICATION (Concluded)

Site No.	Route-Section	County	Direction	Lane	Log Mile	Job No.	Date Const.
24	167-3	Calhoun	South	-	7.5	7716	10-75
25&	81-6	Lincoln	North	-	10.3	2-663	6-74
26	65-20	Chicot	North	-	10.0	2659	10-69
27	65-18	Drew	North	-	2.0	2824	6-78
28	82-11	Chicot	West	-	2.9	2534	10-59
29#	55-11	Crittenden	North	OL	9.1	11829	10-78
30*	70-8	Garland	East	-	1.9	6-677	3-78
31@	1-8	Phillips	North	-	1.5	11955	10-79
32@	1-9	Lee	North	-	4.8	11-730	11-78
33	49-9	Monroe	North	-	17.5	11519	9-65
34	64-13	Woodruff	East	-	4.0	11956	6-79
35	64-14	Woodruff	East	-	6.5	11956	6-79
36	49-3	Craighead	South	-	3.9	10677	10-65
37	79-6	Dallas	North	OL	3.0	7753	9-76
38	79-6	Dallas	South	OL	3.0	7753	9-76

Symbols @, &, *, # Indicate Road Investigated By:

@ HRP 4, & HRP 17, * HRP 38, # LTM

lanes of three roads: sites 1 and 2 on Route 65-15 (in Pine Bluff); sites 5 and 6 on Route 71-16B (in Fayetteville); and sites 13 and 14 on Route 65-12A (in Saline County just north of the junction with Route 167-12). These six sites showed visual signs of distress as follows: slick surface and rutting (sites 1 and 2); excessive rutting, shoving and distortion (sites 5 and 6); and multiple cracks and rough surface on site 14 (inner lane) with much less cracking on site 13 (outer lane).

Pavement test sites that are also part of the long term monitoring (LTM) program include sites 8, 17, 18, 29, and 37. These sites are on 4-lane divided highways and are in the outside lane. The other 4- lane highways included in this investigation are: sites 3 and 4 (Fayetteville Bypass); site 9 (Route 271-1 in Ft. Smith); site 22 (Smackover Bypass); and site 38 (Fordyce Bypass).

The approximate locations of the pavement test sites are shown in Figure 1. The sites range from the Missouri line in Northwest Arkansas to the Louisiana line in Southwest Arkansas to the Mississippi river in Southeast Arkansas to just north of Jonesboro in Northeast Arkansas.

Some of the test sites are located in the vicinity of previous highway research projects. HRP No. 4 reported pavement performance and Benkelman Beam deflection tests for sites 31 and 32. HRP No. 17 reported on asphalt cement tests for sites 12 and 25. HRP No. 38 reported asphalt surface durability and skid resistance for sites 3, 10, 11,

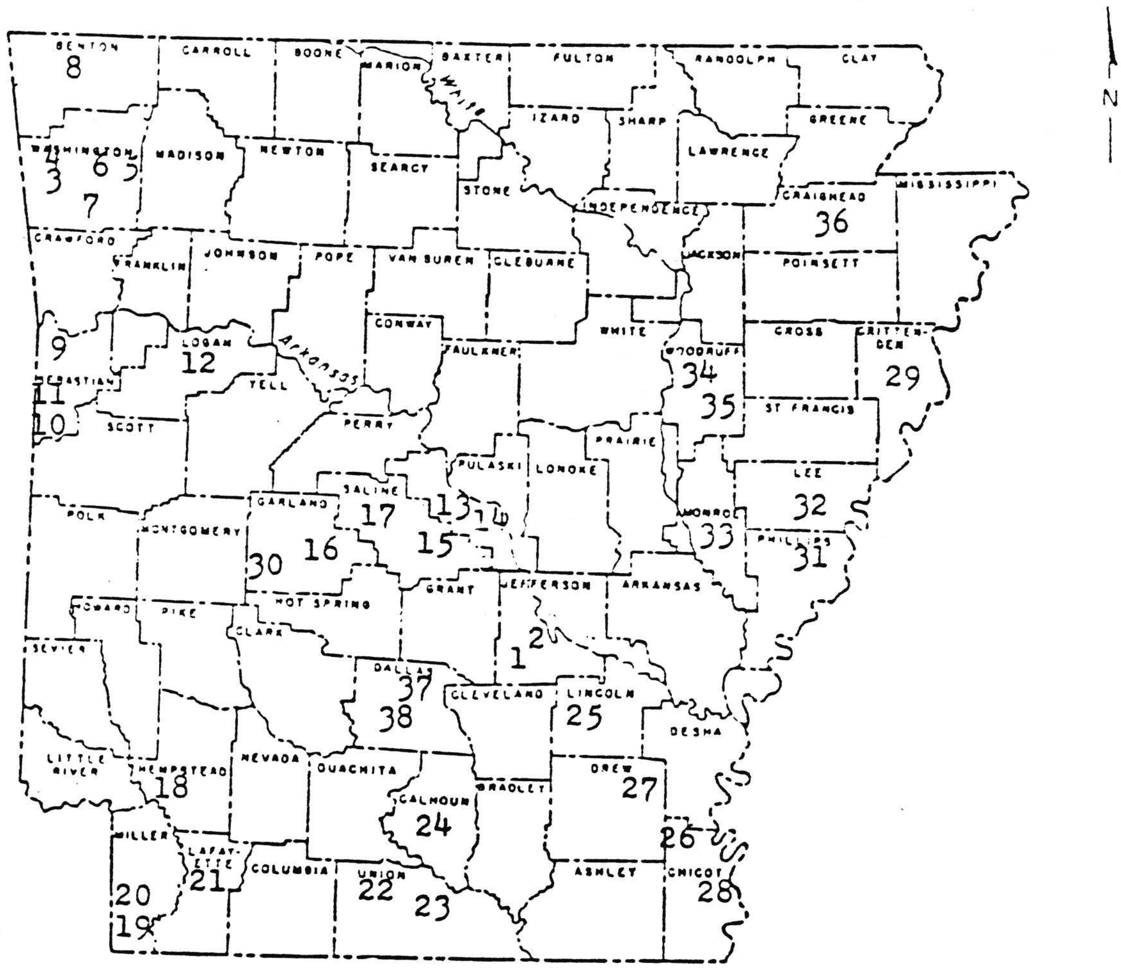


Figure 1. Location of Pavement Test Sites

16, 19, 20, 21 and 30.

A sample of the total asphalt mixture at each site was obtained by the AHTD Materials and Research Division using a 4-inch round, diamond studded core barrel attached to a vertical-shaft, water cooled coring machine. Nine cores were secured at each test site. The core drilling and numbering pattern is shown in Figure 2. Three cores were taken in the outer wheel path (OWP), three cores were taken in the inner wheel path (IWP), and three cores were taken between the wheel path (BWP). The core locations were staggered as shown to prevent the tires of a vehicle from hitting two core holes at the same time. Each core was labeled as to site number, core number and date cored, and wrapped in heavy manila paper for transporting to the laboratory.

The date of core removal at each site is shown in Table III. The age of the cored pavement surface shown in Table III was estimated from the job completion date reported in Table II. The first cores were secured at sites 1 and 2 on April 19, 1982; the last core sample was taken at site 38 on November 14, 1982. The cores taken at site 16 were not intact and could not be tested for density and stability. The cores appeared to be partially "stripped." The cores taken at site 18 were only partially intact and could not be tested for density and stability. The construction at this site was an ACHM overlay of portland cement concrete (PCC) pavement with a binder layer

Dynaflect Deflection Locations

And

4 inch Core Drilling Pattern and Numbering System

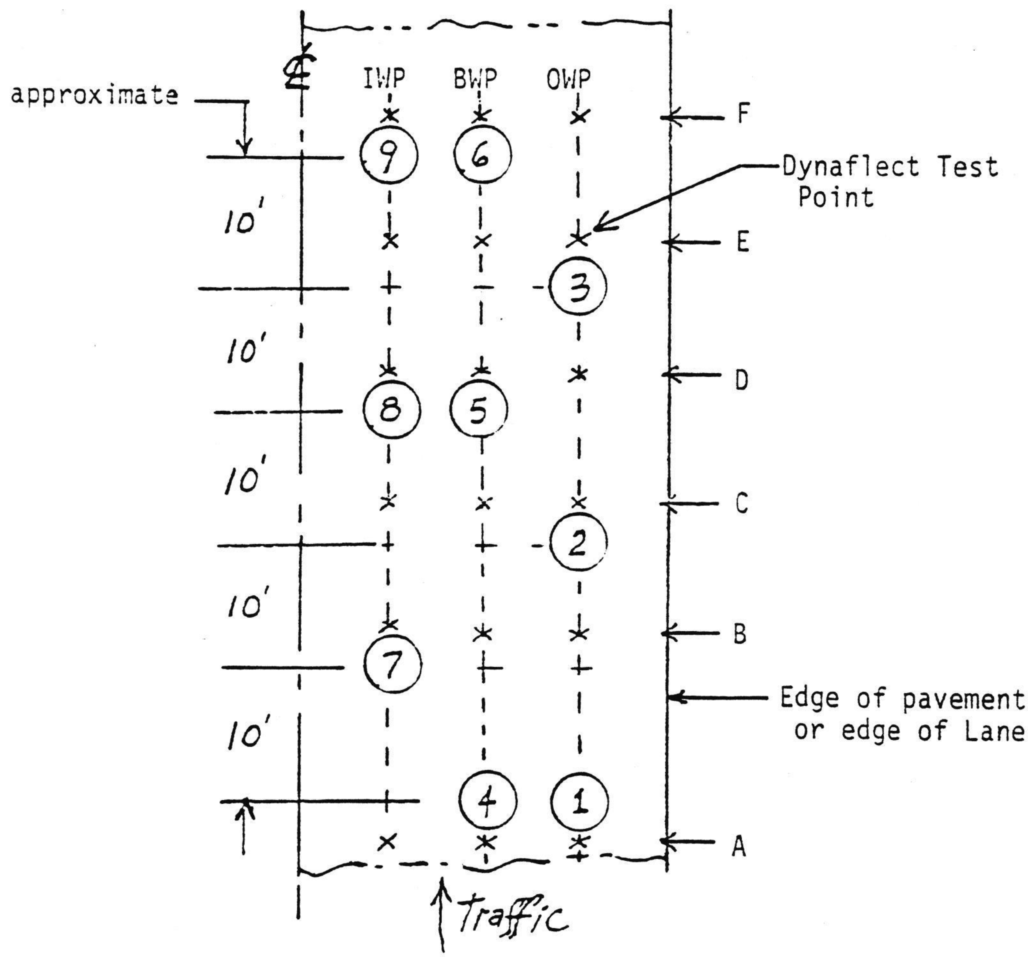


Figure 2. Core Drill Pattern and Dynaflect Test Points

TABLE III TRAFFIC AND SAMPLE DATE

Site No.	R-S	Date Cored	Age Years	Total EAL	1982 Traffic ADT	%T	EAL*	EAL Lanes
1	65-15	4-19-82	3.7	0.25	7080	16	55	2
2	65-15	4-19-82	3.6	1.01	7080	16	218	2
3	71-17	4-27-82	10.4	3.06	5800	18	290	1
4	71-17	4-27-82	0.5	0.22	7000	18	280	2
5	71-16B	4-27-82	5.1	0.52	17500	5	74	2
6	71-16B	4-27-82	5.0	0.13	17500	5	19	2
7	71-16	4-27-82	15.3	1.44	7640	16	170	1
8	71-19	4-28-82	3.4	0.36	4800	20	106	2
9	271-1	5-11-82	12.5	0.59	6230	9	54	2
10	71-13	5-10-82	11.0	0.94	5400	14	105	1
11	71-14	5-10-82	9.0	1.82	10800	15	225	1
12	22-3	5-10-82	17.6	0.68	4250	9	53	1
13	65-12A	6-3-82	18.8	1.58	9800	17	120	2
14	65-12A	6-3-82	18.8	1.46	9800	17	111	2
15	167-12	6-3-82	18.0	1.47	3930	20	109	1
16	70-9	6-10-82	11.1	0.96	5700	13	103	1
17	30-22	7-2-82	4.2	2.37	26200	22	619	2
18	30-12	5-17-82	1.0	0.99	16000	37	650	2
19	71-1	5-18-82	6.6	0.55	2700	23	87	1
20	71-2	5-18-82	9.9	1.19	4950	21	146	1
21	82-2	3-29-83	3.0	0.48	4150	20	115	1
22	7-2	6-1-82	5.0	0.30	3150	17	80	1
23	167-2	6-1-82	2.2	0.25	4100	17	135	1

TABLE III TRAFFIC AND SAMPLE DATE (Concluded)

Site No.	R-S	Date Cored	Age Years	Total EAL	1982 Traffic ADT	1982 Traffic %T	1982 Traffic EAL*	EAL Lanes
24	167-3	6-2-82	6.7	0.89	7080	17	242	1
25	81-6	6-17-82	8.0	0.43	2050	19	47	1
26	65-20	6-16-82	13.7	1.29	3720	22	98	1
27	65-18	6-17-82	4.0	0.47	4000	19	92	1
28	82-11	6-16-82	22.7	1.92	4950	18	107	1
29	55-11	10-13-82	4.0	2.60	24200	21	606	2
30	70-8	6-10-82	4.6	0.11	4250	5	30	1
31	1-8	10-4-82	3.0	0.24	3900	19	103	1
32	1-9	10-4-82	3.9	0.32	4300	13	78	1
33	49-9	10-4-82	17.1	0.38	1940	14	38	1
34	64-13	10-5-82	3.3	0.36	4150	21	121	1
35	64-14	10-5-82	3.3	0.30	2350	23	75	1
36	49-3	10-12-82	17.0	1.01	5600	13	101	1
37	79-6	6-2-82	5.8	0.87	2920	20	174	2
38	79-6	10-14-82	6.0	0.84	2580	20	154	2

Note: Total EAL In Millions; 1982 EAL In Thousands
* EAL Distribution: 80% Outside Lane, 20% Inside Lane, Except Site 13 and 14 Split 52:48

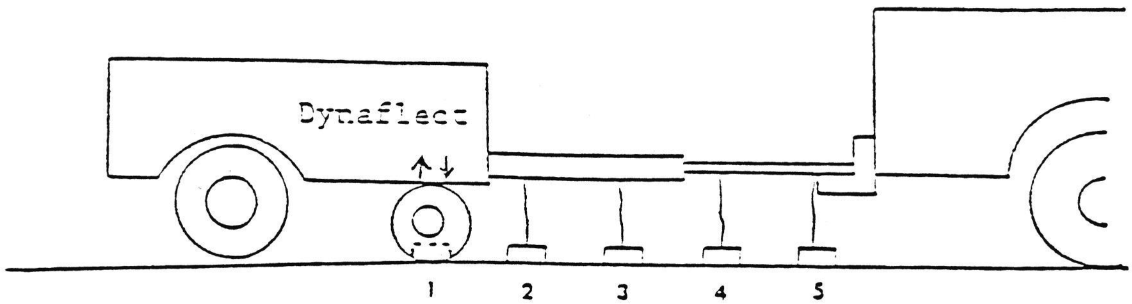
placed on top of the PCC. A "SAMI" layer of rubberized asphalt chip seal had been placed on top of the binder layer, and, finally, an ACHM wearing surface had been placed on top of the SAMI layer. The core broke in two at the interface of the SAMI and binder layer.

After the coring operation was complete, a series of Dynaflect tests was performed using the AHTD Materials and Research Division's device. A sketch of the Dynaflect device is shown in Figure 3. The dynaflect tests were run at 6 points along the OWP, at 6 points along the BWP and then at 6 points along the IWP. These test points were at the same cross-section in each path. Since each dynaflect test measures the pavement deflection at 5 points 4 feet apart in the longitudinal direction, a total of 90 deflection readings were recorded at each pavement test site. The location of dynaflect tests in relation to the core hole location is shown in Figure 2. The test site included about 70 to 80 feet of the test lane.

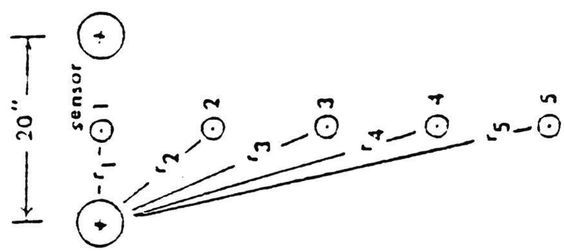
The amount of rutting in the wheel path was measured at four locations in each wheel path. An eight foot aluminum channel beam and steel scale was used to measure the maximum rut depth to the nearest 1/32 inch.

The pavement surface condition was noted and a sketch of surface cracking or distress was recorded. Cracking was mapped in three classes in accordance with the AASHO (3) definition. These classes of cracking are defined as follows: class 1, fine random cracks having no definite

DYNAFLECT



Elevation View



Plan View

r_1 = 10.0"

r_2 = 15.6"

r_3 = 26.0"

r_4 = 37.4"

r_5 = 49.0"

* 8 Hertz Oscillating Load
* 1000 pound peak to peak force
* Five geophones arrayed as shown between two force wheels

Figure 3. Dynaflect Geometry (after Way, 13)

pattern; class 2, a progression of class 1 cracking into a definite pattern with widening of the cracks and slight spalling along the crack edges; class 3, a progression of class 2 cracking with pronounced widening of the cracks and separation of the individual segments into loose pieces. The units of measurement for cracks used in the AASHO Road Test were sq. ft. per 1000 sq. ft. of surface area. Photographs of the pavement were taken for comparison purposes. Each site was given a condition rating based upon The Asphalt Institute IS-169 Method (37).

Later, the roughness of the pavement in the area of each test site was measured by the AHTD using the Mays Meter. Likewise, the skid resistance of the pavement in the area of each site was measured by the AHTD using a Skid Trailer.

The amount of traffic in 1982 over each test site is also reported in Table III. This information was provided by the Planning Division of the AHTD. Further, the total EALs over each test site from date of construction to date of coring are given in Table III.

<u>Laboratory Tests</u>

The cores were stored in the laboratory by laying them on their sides on a shelf until ready for evaluation. The cores were unpackaged and marked with lumber keel as to site and core number. The layer thickness was measured, along with the overall core height. The core to be tested was then sawed into layers using a Target Masonary Saw with a water cooled 14-inch diamond studded blade. The sawed core layers were then air dried until a constant weight was obtained. The height and diameter of each core was measured using a 0.001 inch dial gage device.

Next, the resilient modulus (MR) of each core layer was measured using the Retsina Mark V device. The resilient modulus test consisted of applying a light pulsating load across the vertical diameter of a specimen causing a corresponding elastic deformation across its horizontal diameter. This deformation was measured with linear variable differential transducers (LVDT). The dynamic load applied to the specimen consisted of a load duration of 0.1 second and a rest period of three seconds.

The deformation and dynamic load were recorded from a digital readout in the MR device. Using this data the MR was calculated by the following equation:

$$MR = P(v + 0.2734)/(h \; \#)$$

where:

MR = resilient modulus

P = peak load

v = Poisson's ratio

h = specimen thickness

\# = deformation across specimen

Schmidt (1) stated that a range of values for Poisson's ratio could be used without excessive error in the calculated MR. He suggested that the assumption of 0.35 for asphalt concrete gave a reasonable agreement among MR values calculated by direct tension, compression, or flexural methods.

The Retsina MR device was made to handle 4 inch diameter samples that are between 1.5 and 3 inches thick. The samples were tested at a temperature of 77 F (plus or minus 2 F). In general, the load applied to the sample was 75 pounds (plus or minus 10 pounds). Since there is no ASTM or AASHTO test method, these limits were set after experience showed that temperature variation and large differences in loads resulted in inconsistant resilient modulus values.

Four readings were taken on each sample with the MR device. Two were taken across the same points. The sample was rotated 90 degrees and two more readings were taken across its diameter. After the MR values were calculated, the standard deviation of the values were determined. If the standard deviation was greater than 10 percent of the largest MR value, the procedure was repeated until agreement was found. If no agreement could be made after three trials, an average of the twelve values was used.

The bulk specific gravity of the surface and binder layers was measured in accordance with ASTM Method D 2726 (22). The weight of the sample in air, water (at 77 F) and saturated surface dry was obtained using a Mettler digital readout automatic balance. The bulk specific gravity of the more open graded base samples was obtained using paraffin in accordance with ASTM Method D 1188 (22).

The Marshall stability (lbs.) and flow (0.01 inch) of each core layer was determined in accordance with ASTM Method D 1559 (22). The maximum stress in pounds per square inch (psi) was then calculated. This value was taken to be equal to the Marshall stability divided by the cross-sectioned area of the specimen. The Marshall modulus (Em) was calculated by dividing the stress by the strain at maximum load. It is noted that the flow was taken to be at the point of maximum load as determined from the strip chart recorder printout from the Marshall test apparatus.

Next, the core specimens were heated to 250 F until soft enough to break apart with a trowel. The loose asphalt mixture was then tested for its maximum specific gravity in accordance with ASTM Method D 2041 (22) except as noted below. The ASTM procedure was modified by using a wetting agent, Aersol OT, in the deaired distilled water. The asphalt mixture was covered with water in a one-half gallon pycnometer and deaired for 15 minutes using a water aspirator. This device pulled a vacuum of about 26 inches of mercury. Care was exercised in removing all of the air bubbles from inside the glass pycnometer prior to taking the final weight of the asphalt mixture in water. A water temperature of 77 F was maintained during the maximum specific gravity test.

The asphalt mixture was then placed in a pan and the excess water removed. The mixture was dried to a constant weight at 212 F before starting the extraction test. The amount of asphalt in each core specimen was determined by extraction in accordance with ASTM Method D 2172 (22). Both Method A (rotarex) and Method B (reflux) were used. The solvent used was 1,1,1 Trichloroethane, technical grade. The amount of ash in the effluent was determined by centrifuging.

The mechanical analysis of the extracted aggregate was performed in accordance with AASHTO Method T30 (38). The aggregate was soaked overnight in water with 3 percent calgon added prior to washing over the number 200 sieve. The material was oven dried and weighed to determine the amount of minus No. 200 material. The extracted aggregate was then sieved over the following sieves: 1.5", 1.12", 1.0", 0.75", 0.50", 0.38", #4, #10, #20, #40, #80, and #200. The total percent material passing each sieve was calculated.

A voids analysis for each core layer tested was performed. The amount of air voids, voids in the mineral aggregate, and voids filled with asphalt (Vf) was calculated on the basis of aggregate effective specific gravity. Otherwise the procedure of The Asphalt Institute MS-2 (15) was followed. The Vf percentage was calculated by taking the difference between the VMA and the AV, multiplying by 100, and dividing by the VMA.

Marshall laboratory mix designs were prepared using these types of generic aggregate: limestone, sandstone, gravel, and syenite. The aggregate gradations used were similar to the cored job mix designs. A Tosco AC-30 paving grade asphalt was used in all of the laboratory mixtures. The laboratory samples were tested in the same manner as the pavement core samples. These data will be used to relate the laboratory test results (of the original mix designs) and to estimate the initial resilient modulus of the pavement mixture.

CHAPTER IV

TEST RESULTS AND DISCUSSION

The pavement core samples were tested in the laboratory to obtain their physical characteristics. These characteristics include: layer thickness, total height of asphalt bound materials, resilient modulus, bulk specific gravity, Marshall stability and flow, maximum mixture specific gravity, asphalt content and aggregate gradation. For most of the sites, triplicate samples were tested, one sample being taken from each wheel path and the third sample from between the wheel paths. The layer thickness for all nine cores at each site was determined. The laboratory molded specimens were tested in triplicate.

The dynaflect tests were repeated at six points along each path and between the wheel paths. These six sets of deflection readings were taken at the same cross section, as previously shown in Figure 2. The maximum rut depth in each wheel path was measured at 4 cross sections.

The Mays meter rideability value was taken for a half mile on either side of the test site. The SN40 skid resistance value was measured in the vicinity of each test site. A pavement condition evaluation was also performed at each site. The field tests were initially performed in 1982 with the dynaflect and rut depth measurement being repeated in 1983.

Pavement Surface Evaluation

Included in Table IV are the Pavement Condition Rating (PCR), Mays meter Rideability, skid number at 40 MPH (SN40), Crack Rating and Rut Depths. The youngest pavement, at site 4, had the best PCR at 96 percent with a Mays value of 99 percent. The IWP rut depth at site 4 was 2/32 in., with no measureable rut depth in the OWP. Site 15 had the lowest PCR value at 53 percent, with a Mays value of 50 percent. The average rut depth at this site was 7/32 in. The site was overlayed after the 1982 field tests were completed.

The degree of cracking shown in Table IV was based upon the AASHO Road Test (3,20) classification system. Time did not permit the measurement of the amount of cracking and the classifictions are, therefore, based upon the visual appearance of the pavement in the test site area. The most severe cracking was observed at sites 12, 14, 28, and 35. No cracking was observed at sites 1, 2, 4, 5, 6, 8, 10, 24, 29, 37, and 38. Regression analyses to determine the best fitted equation and coefficient of correlation (R) of the values in Table IV were performed. Crack classification related to the maximum rut depth indicated a semi-logarithmic relationship with an R value of 0.556.

Pavement roughness, as measured by the Mays meter, ranged from 22 percent at site 5 to 99 percent at site 4. A Mays reading of 100 percent indicates a very smooth

TABLE IV PAVEMENT SURFACE EVALUATION

Site No.	R-S	PCR %	MAYS %	SKID SN40	CRACKS CLASS	RUT DEPTH IWP	RUT DEPTH OWP
1	65-15	68	60	44	0	12	14
2	65-15	68	60	46	0	16	14
3	71-17	87	87	39	1.8	6	7
4	71-17	97	99	33	0	2	0
5	71-16B	72	22	-	0	34	36
6	71-16B	72	44	-	0	14	22
7	71-16	69	63	55	0.4	9	6
8	71-19	86	90	29	0	7	6
9	271-1	53	44	52	2.0	11	9
10	71-13	83	88	60	0	9	6
11	71-14	69	78	51	0.8	6	16
12	22-3	73	68	54	2.8	6	5
13	65-12A	76	63	50	1.8	9	7
14	65-12A	76	47	55	2.8	7	4
15	167-12	57	50	53	2.2	6	8
16	70-9	84	91	40	1.0	7	4
17	30-22	91	88	59	0.8	10	5
18	30-12	90	78	59	0.6	10	5
19	71-1	85	75	49	1.8	8	3
20	71-2	86	71	51	1.0	11	10
21	82-2	86	80	50	1.8	9	8
22	7-2	86	73	55	1.6	10	12
23	167-2	89	85	51	0.8	9	5

TABLE IV PAVEMENT SURFACE EVALUATION (Concluded)

Site No.	R-S	PCR %	MAYS %	SKID SN40	CRACKS CLASS	RUT DEPTH IWP	OWP
24	167-3	71	56	45	0	14	25
25	81-6	74	66	46	0.8	7	8
26	65-20	80	80	49	1.8	12	10
27	65-18	93	90	44	0.2	8	8
28	82-11	81	80	43	2.4	6	9
29	55-11	81	78	44	0	16	15
30	70-8	73	76	54	2.0	4	3
31	1-8	81	86	40	1.4	1	2
32	1-9	86	90	49	1.0	8	9
33	49-9	91	88	59	1.6	9	9
34	64-13	80	90	49	1.4	5	1
35	64-14	81	82	59	2.4	4	1
36	49-3	83	84	42	0.6	6	7
37	79-6	86	75	45	0	17	10
38	79-6	86	72	47	0	18	12

Note: Rut Depths In 1/32 Inch
Cracks Based On AASHO Definition
PCR = Pavement Condition Rating
MAYS = Mays Meter, 100% = Smoothest Road

pavement. The best correlation of Mays values was with maximum rut depths, having a semi-logarithmic relationship with an R value of 0.704. The Mays values also had a linear relationship with PCR values, having an R of 0.506.

The skid numbers shown in Table IV range from 29 at site 8 to 60 at site 10. These skid numbers may be useful to compare pavement surfaces to indicate the degree of aggregate polish or the richness of the surface mix. The SN40 values for the sites which were skid tested on HRP 38 (21) are compared below with the SN40 values of Table IV. These values, by site number, are: #3, 28 vs 39; #10, 53 vs 60; #11, 34 vs 51; #16, 51 vs 40; #19, 46 vs 49; #20, 43 vs 51; #21, 37 vs 50; and #30, 35 vs 54. The pavement sufaces are unchanged for sites 10, 11, 16, 19, and 20. On the average, the skid number of these five sites increased by 5 points. Since the skid tests of HRP 38 were performed, sites 3, 21 and 30 have been overlayed.

Pavement Layer Description

The pavement layer description and aggregate composition for each site are shown in Table V. The 9 core samples taken at each site were evaluated in the laboratory to determine the type of asphalt mixture in each layer and the classification of the mineral aggregate.

At sites where the job plans were available, the planned application rates for each layer are given in Table V. In some cases, the plans indicated the existing pavement structure which is also shown as part of Table V.

TABLE V PAVEMENT LAYER DESCRIPTION

Site No.	R-S	Top 1	Layer 2	Layer 3	Layer 4	Layer 5	Layer 6	Aggregate Sur.	Aggregate Base
1	65-15	S 165	BI 330	ACHM	PCC	-	-	NS	NS
2	65-15	S 165	BI 330	ACHM	-	-	-	NS	NS
3	71-17	S 165	BI 440	Ba 990	NL	-	-	LS	LS
4	71-17	S 165	BI 495	Ba 1320	NL	-	-	LS	LS
5	71-16B	S 165	BI 220	Bao 300	ACHM	PCC	-	LS	LS
6	71-16B	S 165	BI 220	Bao 300	ACHM	PCC	-	LS	LS
7	71-16	SS	S 172	BI 230	Ba 660	PCC	-	LS	LS
8	71-19	S 165	BI 440	Ba 990	NL	-	-	LS	LS
9	271-1	S 172	BI 230	Ba 690	PCC	-	-	SS	SS
10	71-13	S 165	BI 220	Ba 600	NL	-	-	SS	SS
11	71-14	S	BI	PCC	-	-	-	SS	SS
12	22-3	S 150	BI 200	PCC	-	-	-	SS	SS
13	65-12A	S 175	BI 230	Ba 920	NL	-	-	NS	NS
14	65-12A	S 175	BI 230	Ba 920	NL	-	-	NS	NS
15	167-12	S 230	Ba 690	NL	-	-	-	NS	NS

TABLE V PAVEMENT LAYER DESCRIPTION (Continued)

Site No.	R-S	Top 1	Layer 2	Layer 3	Layer 4	Layer 5	Layer 6	Aggregate	
								Sur.	Base
16	70-9	S	S2	BI	AC	-	-	NOV	NOV
17	30-22	PMS	S 165	BI 165	Bao 300	PCC	-	NS	NS
18	30-12	PMS	S 165	SAMI	BI 355	PCC	-	NOV	NOV
19	71-1	S 165	BI 440	S2 165	BI 220	AC	-	GVL	GVL
20	71-2	S 165	BI 220	Ba 660	AC	-	-	GVL	GVL
21	82-2	S	S2	S3	BI	AC	-	NOV	GVL
22	7-2	S 165	BI 220	Ba 660	NL	_	-	GVL	GVL
23	167-2	S	CS	S 150	BI 200	Ba 350	AC	GVL	GVL
24	167-3	S 165	BI 220	Ba 330	AC	-	-	NS	GVL
25	81-6	S	S2	S3	CS	RM	-	NS	GVL
26	65-20	S 175	BI 230	PCC	-	-	-	NS	NS
27	65-18	PMS	S 165	BI 220	Ba 300	ACHM	PCC	NS	NS
28	82-11	S 150	BI 200	SB2	ACHM	-	-	NS	NS
29	55-8	PMS	S 165	BI 576	Bao 300	PCC	-	LS	LS
30	70-8	S	S2	S3	BI	-	-	SS	NS

TABLE V PAVEMENT LAYER DESCRIPTION (Concluded)

Site No.	R-S	Top 1	Layer 2	Layer 3	Layer 4	Layer 5	Layer 6	Aggregate Sur.	Aggregate Base
31	1-8	S	S2	S3 200	GB2	-	-	LS	GVL
32	1-9	S	S 200	GB2	-	-	-	GVL	GVL
33	49-9	CS	S 150	BI 200	SB2	GB2	-	LS	SS
34	64-13	S	S 175	BI 230	Ba 460	SB2	-	SS	SS
35	64-14	S	S 175	BI 230	Ba 460	GB3	-	SS	SS
36	49-3	S 172	BI 230	Ba 460	RM	-	-	LS	GVL
37	79-6	S 165	BI 220	Ba 880	GB	NL	-	NOV	GVL
38	79-6	S 220	S 172	BI 230	Ba 805	GB	NL	NOV	GVL

Layer Abbreviations:
AC = Existing Asphalt Road, ACHM = Existing Asphalt Concrete Pavement
BI = Binder Course, Ba = Base Course, Bao = Open Graded Crack Relief
CS = Chip Seal, GB = Gravel Base, NL = Road Built On New Location
PCC = Portland Cement Concrete Pavement, PMS = Plant Mix Seal
S = Surface Course, SB = Stone Base, SS = Slurry Seal

Number After The Layer Type Indicates The Planned Application Rate In Pounds Per Square Yard.

Aggregate Abbreviations: LS = Limestone, SS = Sandstone, GVL = Gravel
NS = Syenite, and NOV = Novaculite

A maximum of 6 pavement layers are recorded, however not all of these materials were obtained in the coring operation as the coring was stopped at the bottom of the last asphalt bound layer. The footnotes to Table V indicate the abbreviation used in the table.

The types of aggregate were positively identified after the extraction test. In addition, the type of asphalt mixture was confirmed from the sieve analysis of the extracted aggregate. The binder layer was generally of the same aggregate source as the surface layer, and at sites having no asphalt base the mineral aggregate shown in Table V as base is the aggregate type found in the binder layer. The pavement layer description will be used later to classify each site as to type of construction. The distribution of types of mineral aggregate included in the surface layer of the 38 test sites are: LS (10), SS (7), GVL (5), NS (11), and NOV (5).

Physical Properties of Core Layers

The physical properties of the surface layers are shown in Table VI. The average measured characteristics of each core layer are as follows: resilient modulus; Marshall modulus, stability and flow; bulk specific gravity; maximum specific gravity; asphalt content; air voids; and voids in the mineral aggregate. The Marshall stability values in this report are given in pounds per square inch (psi). Standard Marshall stability specimens are nominally 4 in. in diameter and 2.5 in. thick and their

TABLE VI SURFACE LAYER PHYSICAL PROPERTIES

Site No.	R-S	Mr 1000 psi	Em 1000 psi	Marshall Stab. psi	Marshall Flow In.	Bulk Sp. Gr.	Max. Sp. Gr.	AC - %	Air Voids %	VMA - %
1	65-15	200	5.2	104	8	2.393	2.417	5.6	1.0	13.9
2	65-15	310	7.0	139	8	2.392	2.428	5.4	1.5	14.0
3	71-17	530	9.3	283	12	2.334	2.407	5.6	3.1	15.6
4	71-17	430	5.6	148	9	2.544	2.485	4.6	2.2	13.1
5	71-16B	250	4.2	131	12	2.438	2.461	5.1	0.9	13.1
6	71-16B	260	5.8	124	9	2.436	2.457	5.2	0.9	13.3
7	71-16	370	9.0	209	9	2.438	2.478	5.2	1.6	13.9
8	71-19	350	7.5	176	10	2.443	2.503	4.7	2.4	13.5
9	271-1	440	7.4	224	12	2.358	2.407	5.2	2.0	13.9
10	71-13	260	6.9	245	14	2.366	2.408	5.2	1.8	13.7
11	71-14	445	8.5	228	10	2.374	2.411	5.3	1.5	13.8
12	22-3	670	11.7	186	6	2.382	2.428	5.2	2.0	13.9
13	65-12A	290	5.9	170	12	2.339	2.391	6.0	2.1	15.8
14	65-12A	420	5.6	199	14	2.223	2.395	5.8	7.2	19.8

TABLE VI SURFACE LAYER PHYSICAL PROPERTIES (Continued)

Site No.	R-S	Mr 1000 psi	Em 1000 psi	Marshall Stab. psi	Flow In.	Bulk Sp. Gr.	Max. Sp. Gr.	AC - %	Air Voids %	VMA - %
15	167-12	420	6.4	210	13	2.274	2.393	6.0	5.0	18.2
16	70-9	-	-	-	-	-	-	-	-	-
17	30-22	150	5.6	141	10	2.402	2.424	5.5	0.9	13.7
18	30-12	-	-	-	-	-	-	-	-	-
19	71-1	490	8.3	290	14	2.357	2.446	4.7	3.7	14.4
20	71-2	300	6.9	225	12	2403	2.442	5.0	0.8	12.5
21	82-2	360	7.6	190	10	2.382	2.419	5.0	1.5	13.1
22	7-2	370	7.5	232	12	2.357	2.401	5.4	1.8	14.2
23	167-2	440	6.3	178	12	2.356	2.419	5.0	2.6	14.1
24	167-3	170	5.0	95	8	2.380	2.406	5.9	1.1	41.4
25	81-6	140	5.3	129	10	2.390	2.419	5.4	1.2	13.8
26	65-20	340	7.2	181	11	2.362	2.397	5.5	1.3	13.9
27	65-18	250	6.7	125	8	2.413	2.453	5.2	1.7	13.8
28	82-11	500	7.5	165	10	2.304	2.375	6.6	3.0	17.8

TABLE VI SURFACE LAYER PHYSICAL PROPERTIES (Concluded)

Site No.	R-S	Mr 1000 psi	Em 1000 psi	Marshall Stab. psi	Flow In.	Bulk Sp. Gr.	Max. Sp. Gr.	AC - %	Air Voids %	VMA - %
29	55-11	470	8.4	154	9	2.419	2.458	5.3	1.8	13.8
30	70-8	490	6.4	244	15	2.208	2.348	6.1	5.8	19.0
31	1-8	600	8.0	250	12	2.416	2.506	4.4	3.5	13.3
32	1-9	320	4.1	108	11	2.338	2.387	6.0	2.1	15.6
33	49-9	520	7.4	201	12	2.446	2.501	5.6	2.2	15.6
34	64-13	280	6.8	159	9	2.333	2.395	5.8	2.6	15.6
35	64-14	470	7.2	214	12	2.352	2.424	5.2	3.0	14.8
36	49-3	580	7.0	235	14	2.439	2.527	5.6	3.5	15.8
37	79-6	380	6.5	172	11	2.356	2.402	5.0	1.9	13.5
38	79-6	200	4.9	170	14	2.363	2.388	5.3	1.1	12.7

Note: Mr = Resilient Modulus, Em = Marshall Modulus, Flow In 0.01 Inch

stability value is expressed in pounds. Thus the tabulated stability values need to be multiplied by a factor of 10 to estimate their stability in pounds.

The relationship of the resilient modulus to the voids filled with asphalt is shown in Figure 4. The best fitted equation indicated a semi-logarithmic relationship with the MR decreasing as the Vf increases, and an R value of 0.631. A linear relationship between MR and Marshall stability is shown in Figure 5. The MR value increases with an increase in Marshall stability. An R value of 0.661 was obtained.

The relationship between MR and air voids is shown in Figure 6. The best fitted curve was a log-log function having an R value of 0.686. This plot indicates an increase in MR with an increase in AV.

Figure 7 is a plot of the relationship between IWP rut depth and air voids. A log-log relationship gave the best fitted equation with an R value of 0.621. The rut depth decreased with an increase in air voids.

The average rut depth for all 38 test sites was 9/32 in. Sites 1, 2, 5, 6, and 24 had rut depths that were much larger than the average, possibly due to plastic flow of the asphalt material. On the average, the IWP ruts were slightly greater than the OWP ruts. When the sites were grouped by type of construction and the 5 sites that may have plastic flow are discounted, an interesting pattern of average rut depths appears. For ACHM over granular base the IWP rut was 7/32 in. and the OWP was 6/32 in. For ACHM

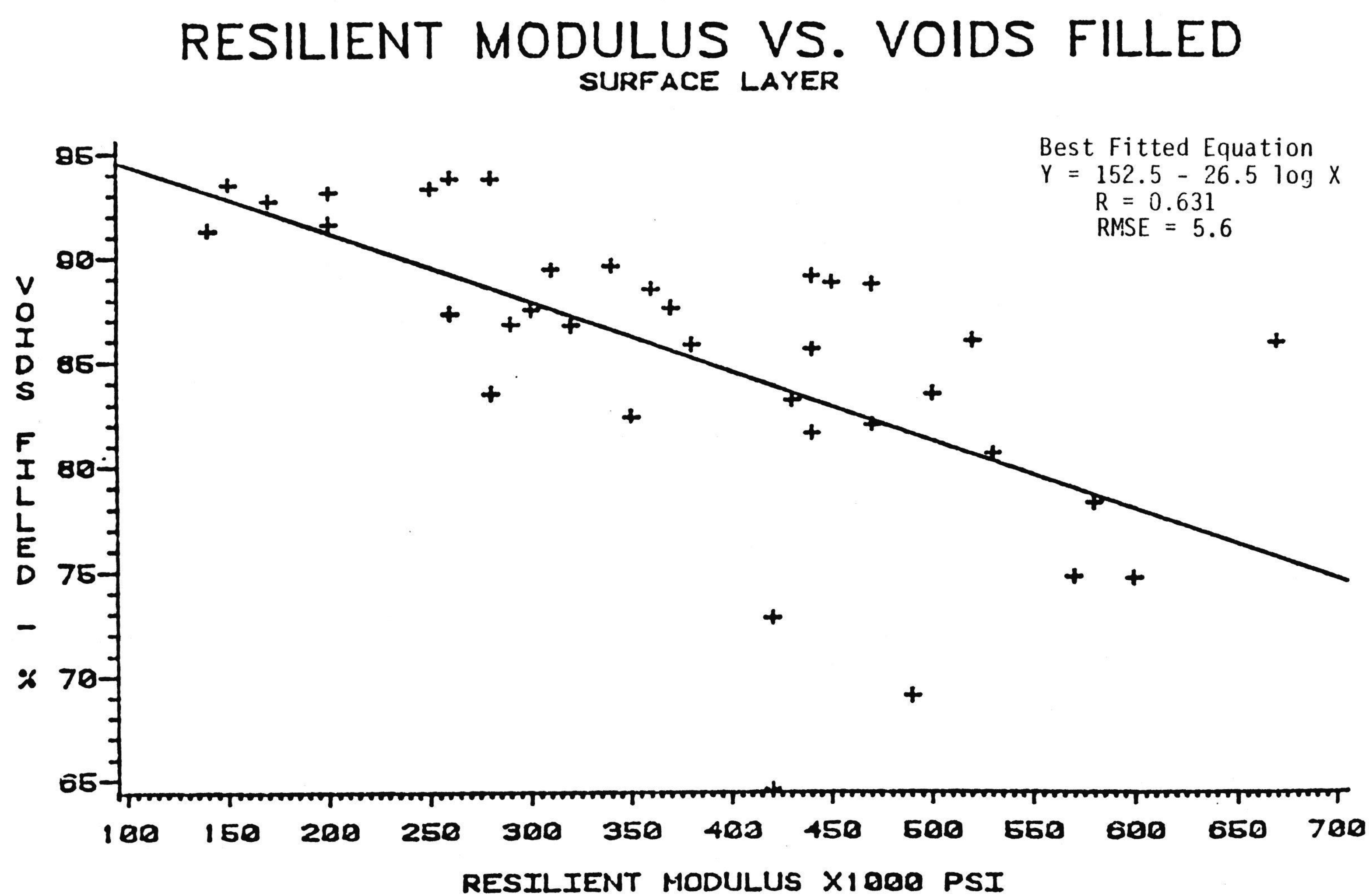


Figure 4. Relationship Between Voids Filled and Resilient Modulus

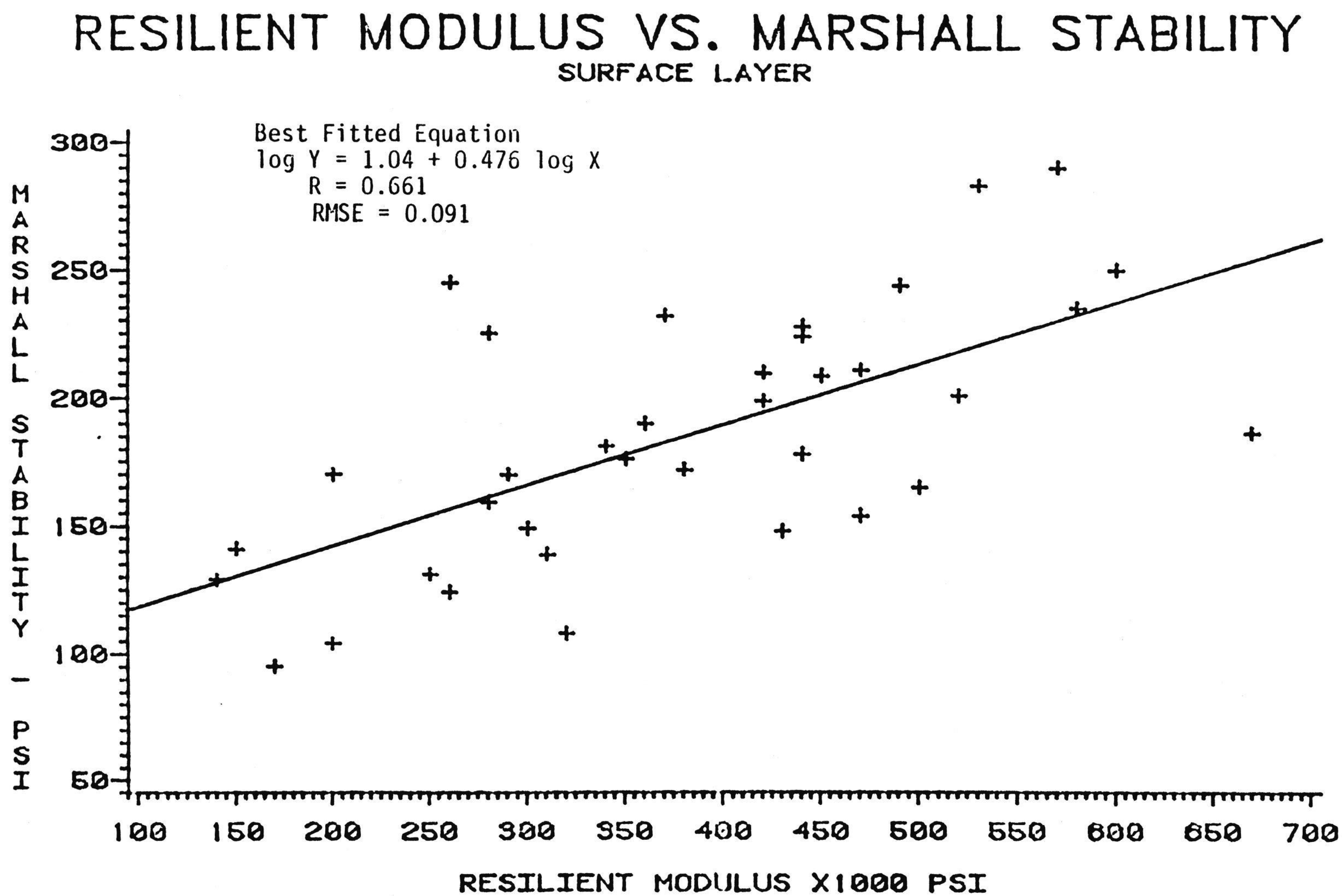


Figure 5. Relationship Between Marshall Stability and Resilient Modulus

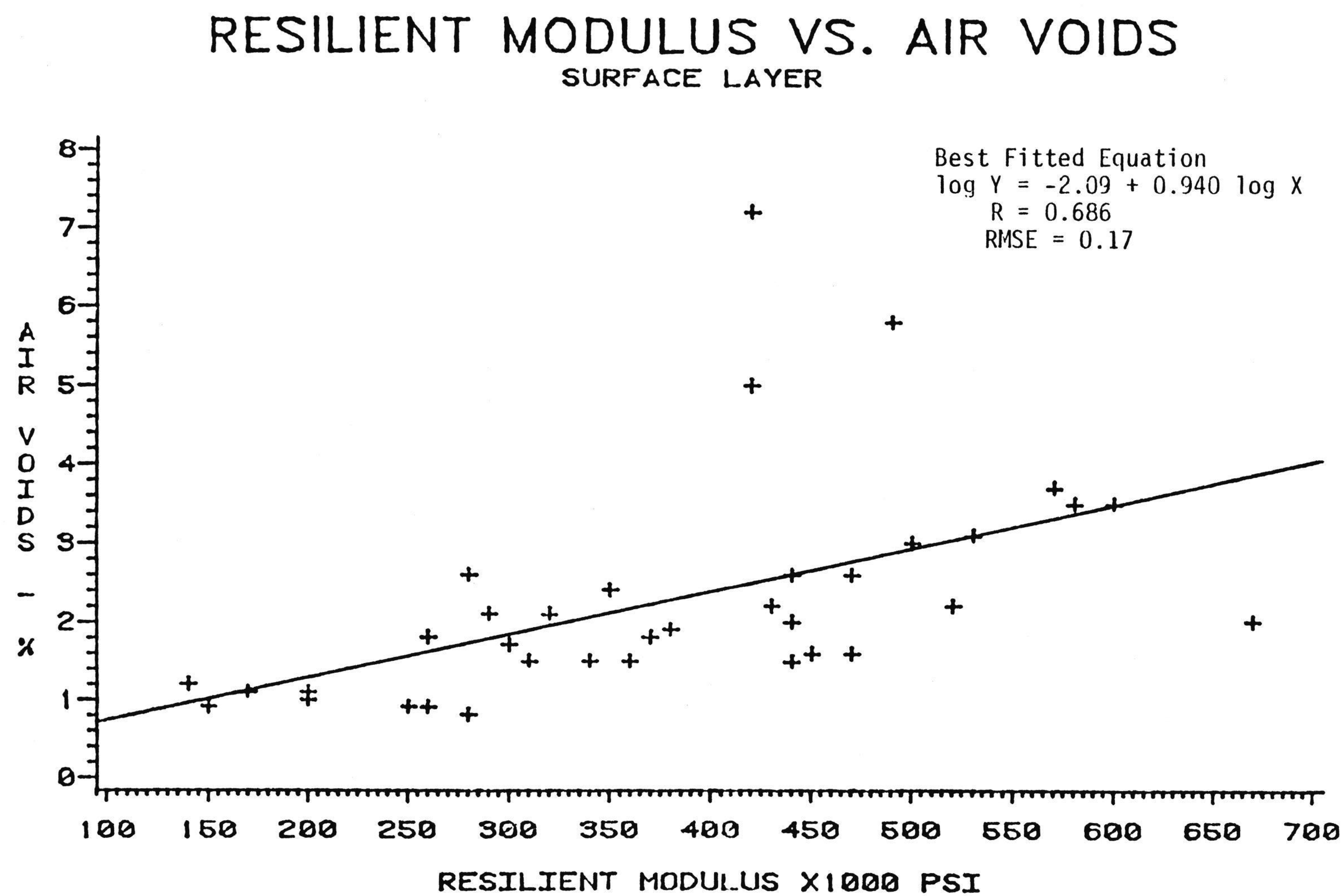


Figure 6. Relationship Between Air Voids and Resilient Modulus

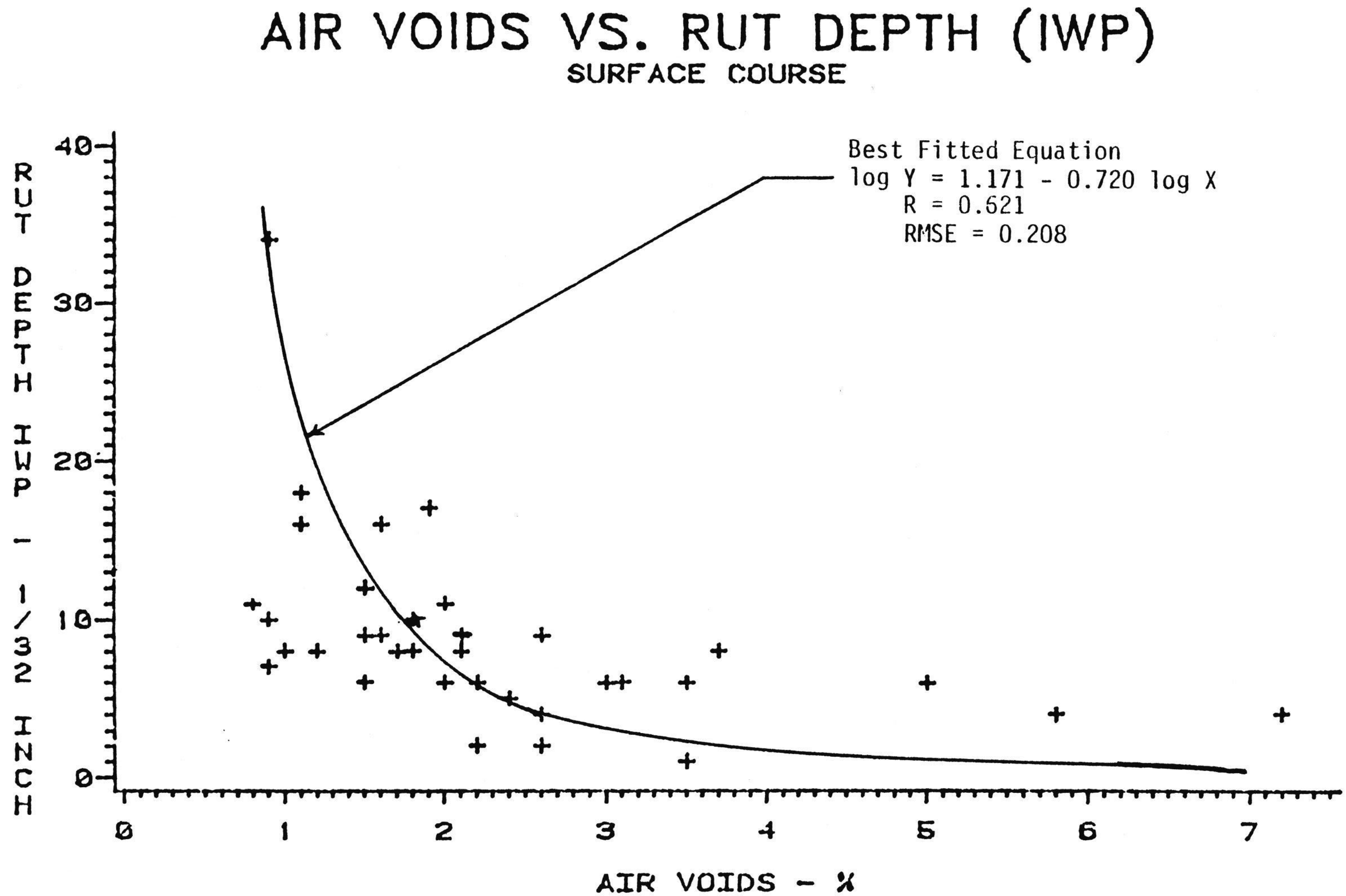


Figure 7. Relationship Between Rut Depth (IWP) and Air Voids

over ABC the IWP rut was 9/32 in. and the OWP rut was 7/32 in. The ACHM over PCC type construction had equal IWP and OWP ruts at 10/32 in. This indicates that maximum rutting will occur in the asphalt pavement where the subgrade support is greatest.

There was a fairly good relationship (R = 0.554) obtained between the cracking index and the maximum rut depth. The semi-logarithmic equation relating the two quantities is:

$$\text{Rut Depth} = 9.39 - 6.51 \log \text{Crack Index}$$

Where: rut depth = 1/32 in.

crack index = class, range 0.1 to 3

The equation indicates that as the degree of cracking increases the rut depth decreases. The mix stability and flow, asphalt cement properties, subgrade support factors, and traffic also influence rutting and cracking. Their contributions to the relationship between cracking and rutting need to be evaluated.

For nine test sites the top surface layer was an overlay over an existing asphalt pavement. Table VII contains the physical properties of these older surface layers found at sites 21, 23, 25, 30, 31, 32, 34, 35, and 38.

The relationship between voids in the mineral aggregate and air voids for the 45 surface layers of Tables VI and VII is shown in Figure 8. The best fitted equation was linear, indicating an increase in VMA with increasing

TABLE VII SECOND SURFACE LAYER PHYSICAL PROPERTIES

Site No.	R-S	Job No.	Mr 1000	Em 1000 psi	Marshall Stab. psi	Marshall Flow psi	Bulk Sp. In.	Max. Sp. Gr.	AC - Gr.	Air Voids %	VMA - % %
21	82-2	3-633	250	5.7	147	8	2.412	2.434	4.8	0.9	12.3
23	167-2	7557	410	5.8	152	11	2.358	2.461	5.7	1.8	14.8
25	81-6	2-536	210	7.5	76	4	2.232	2.448	4.2	8.8	18.0
30	70-8	6-602	380	4.2	124	12	2.184	2.384	6.2	8.4	21.0
31	1-8	11-581	400	7.3	178	10	2.507	2.523	5.3	0.7	13.7
32	1-9	11489	840	10.7	231	9	2.401	2.493	4.9	3.8	15.3
34	64-13	11707	400	9.9	300	12	2357	2440	5.8	3.4	16.6
35	64-14	11682	680	14.6	454	13	2.282	2.404	6.0	5.2	18.4
38	79-6	7615	260	5.1	147	12	2.315	2.362	5.9	1.6	15.5

Note: Mr = Resilient Modulus, Em = Marshall Modulus, Flow in 0.01 Inch

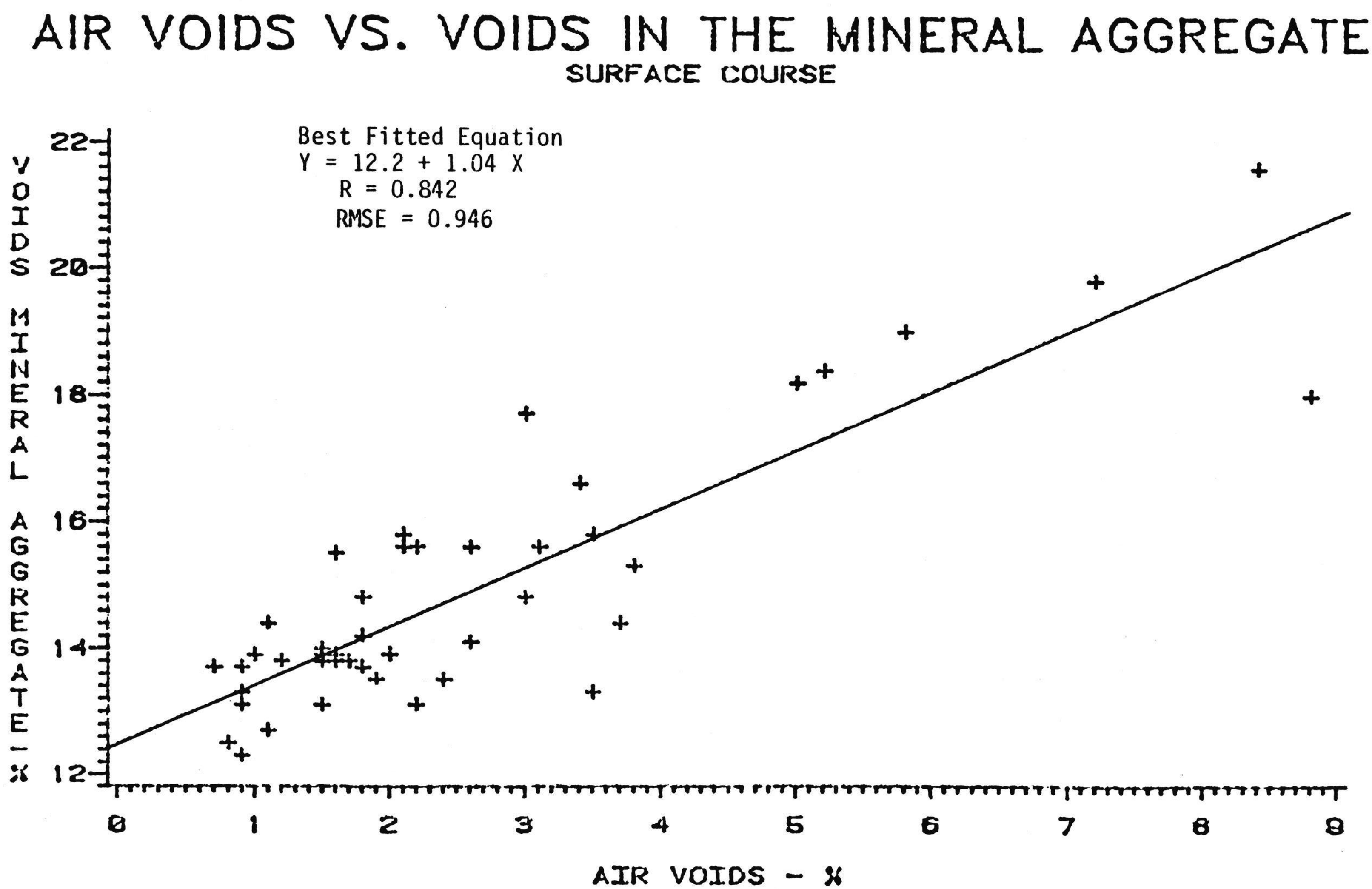


Figure 8. Relationship Between Voids In The Mineral Aggregate and Air Voids

AV. The R value was 0.842.

A very good relationship was obtained for the 36 surface layers between voids filled and air voids as shown in Figure 9. This linear relationship had an R value of 0.974. The voids filled increased with a decrease in air voids.

Figure 10 presents the relationship between MR and Marshall modulus for the 45 surface layers of Tables VI and VII. The best fitted equation gave a linear relationship with an R value of 0.699. The MR value increases with an increase in Em.

It is noted that after the coring operation was complete, overlays were placed at the following sites: 3, 5, 6, 9, 15, 25, and 29. In addition, sites 1 and 2 were rehabilitated by in-place recycling with addition of a plant mix seal surface. A seal coat was also placed over sites 28 and 35. This overlay, reconstruction and maintainance work may indicate that the pavements at these sites had reached their terminal serviceability. An evaluation of traffic carried and the physical characteristics of the pavement may indicate possible changes in the asphalt mixture composition to increase service life.

Physical properties of the binder layers are reported in Table VIII. No tests were performed on a binder layer for sites 16, 18, 23, 25, 30, 31, and 32 for reasons indicated in the table. A binder mixture may have

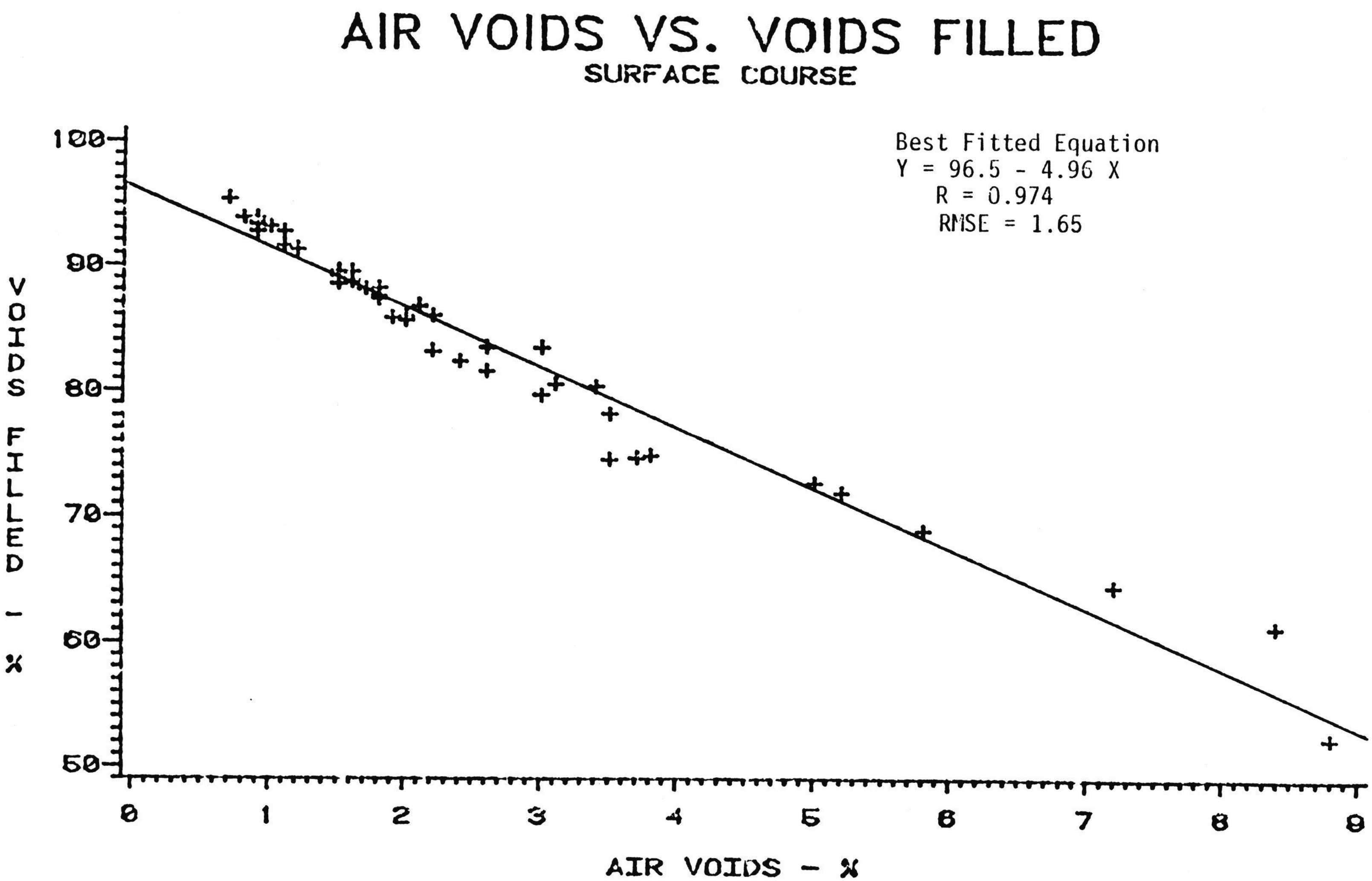


Figure 9. Relationship Between Voids Filled and Air Voids

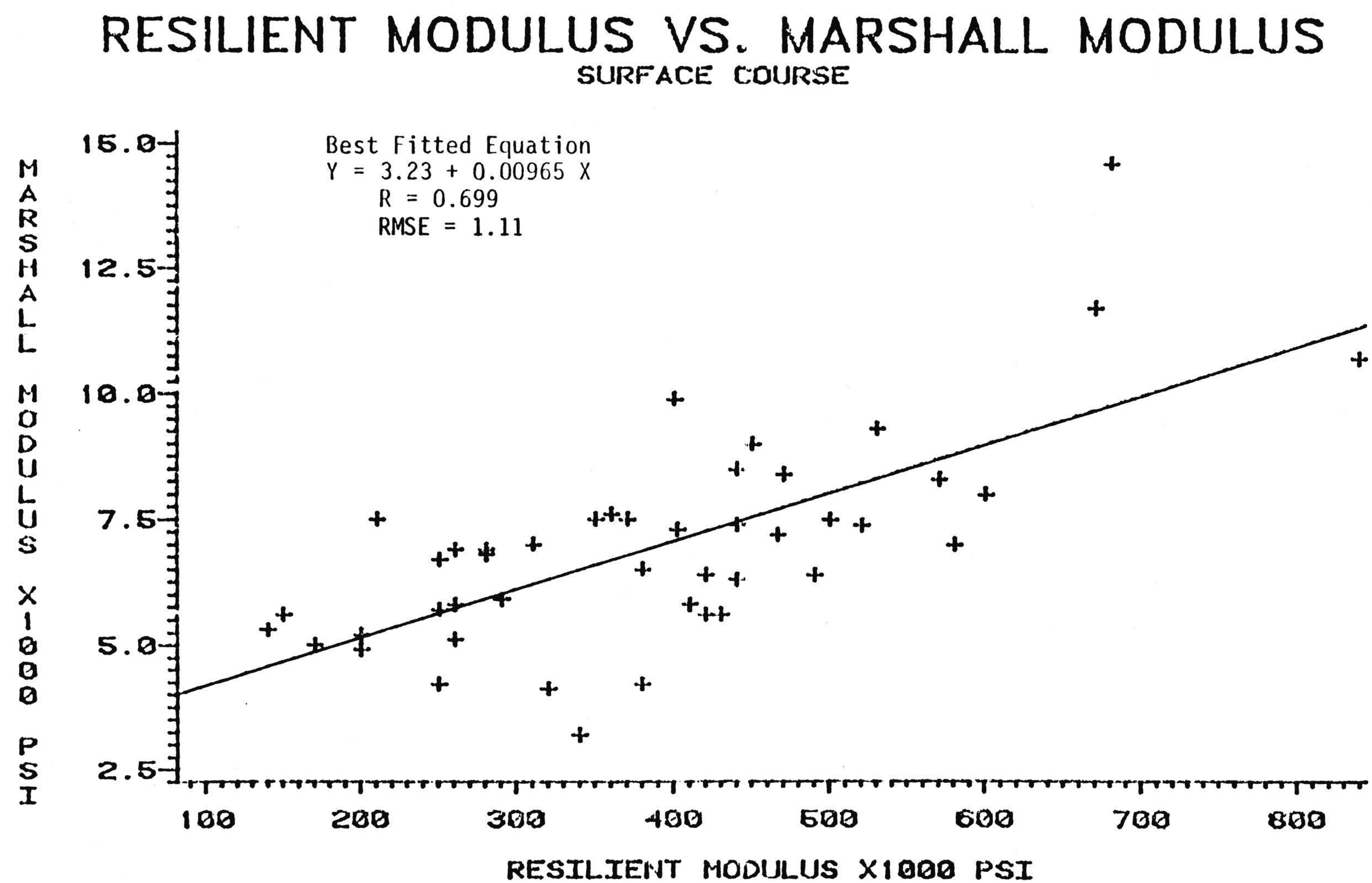


Figure 10. Relationship Between Marshall Modulus and Resilient Modulus

TABLE VIII BINDER LAYER PHYSICAL PROPERTIES

Site No.	R-S	Mr 1000 psi	Em 1000 psi	Marshall Stab. psi	Marshall Flow In.	Bulk Sp. Gr.	Max. Sp. Gr.	AC - %	Air Voids %	VMA - %
1	65-15	190	5.7	187	13	2.397	2.451	4.7	1.8	13.0
2	65-15	270	7.1	142	8	2.401	2.451	4.6	2.1	12.8
3	71-17	410	7.0	176	10	2.310	2.456	5.0	6.0	17.2
4	71-17	470	7.4	157	9	2.442	2.483	4.9	1.6	13.3
5	71-16B	480	8.0	215	11	2.444	2.473	4.5	1.2	12.2
6	71-16B	570	9.3	205	10	2.439	2.476	3.9	1.5	10.6
7	71-16	510	8.9	229	10	2.479	2.507	4.4	1.1	11.6
8	71-19	450	7.1	221	12	2.457	2.499	4.0	1.7	11.5
9	271-1	410	9.1	341	15	2.316	2.406	4.7	3.9	14.5
10	71-13	420	10.0	283	12	2.297	2.422	4.0	5.1	14.2
11	71-14	500	10.6	280	12	2.336	2.435	4.3	4.0	13.8
12	22-3	600	12.1	249	8	2.363	2.429	4.5	2.7	15.3
13	65-12A	290	7.4	209	12	2.378	2.413	4.5	2.2	12.6
14	65-12A	410	6.8	230	14	2.326	2.426	4.9	4.1	15.1

TABLE VIII BINDER LAYER PHYSICAL PROPERTIES (Continued)

Site No.	R-S	Mr 1000 psi	Em 1000 psi	Marshall Stab. psi	Marshall Flow In.	Bulk Sp. Gr.	Max. Sp. Gr.	AC - %	Air Voids %	VMA - %
15	167-12	350	5.0	230	18	2.300	2.440	4.6	5.7	16.1
16	70-9	STRIPPED-NO CORE								
17	30-22	370	5.8	149	10	2.313	2.443	4.5	5.6	15.9
18	30-12	SAMI LAYER-BINDER LAYER NOT RECOVERED								
19	71-1	610	8.9	220	10	2.339	2.484	3.6	5.8	14.0
20	71-2	390	8.3	170	8	2.406	2.434	4.6	1.2	11.8
21	82-2	180	7.0	236	14	2.380	2.424	5.0	1.3	13.2
22	7-2	420	10.0	255	10	2.312	2.430	4.3	4.8	14.6
23	167-2	NOT TESTED								
24	167-3	170	5.6	176	12	2.398	2.426	5.3	1.2	13.4
25	81-6	THIRD LAYER TYPE 3 SURFACE, NOT TESTED								
26	65-20	420	7.2	177	10	2.399	2.432	4.5	1.4	11.8
27	65-18	310	10.6	205	8	2.416	2.559	4.6	1.8	12.4
28	82-11	770	14.0	228	6	2.396	2.448	4.4	2.1	12.3

TABLE VIII BINDER LAYER PHYSICAL PROPERTIES (Concluded)

Site No.	R-S	Mr 1000 psi	Em 1000 psi	Marshall Stab. psi	Marshall Flow In.	Bulk Sp. Gr.	Max. Sp. Gr.	AC - %	Air Voids %	VMA - %
29	55-11	430	5.6	117	9	2.434	2.478	4.6	1.6	12.6
30	70-8	NO BINDER-OLD PAVEMENT								
31	1-8	NO BINDER-OLD PAVEMENT								
32	1-9	NO BINDER								
33	49-9	690	11.4	245	9	2.366	2.445	4.3	3.2	13.1
34	64-13	360	8.3	341	16	2.366	2.458	4.7	3.7	14.4
35	64-14	440	8.9	375	14	2.322	2.431	4.7	4.5	15.2
36	49-3	450	9.2	255	11	2.498	2.561	4.5	2.2	13.1
37	79-6	460	7.1	156	9	2.333	2.420	4.6	3.6	14.2
38	79-6	300	6.2	196	13	2.348	2.409	4.6	2.6	13.2

Note: Mr = Resilient Modulus, Em = Marshall Modulus, Flow in 0.01 Inch

aggregate up to 1.0 in. in size, which generally results in a lower asphalt content than found in the surface mixture. The asphalt content of these binder mixtures ranged from 4.0 to 5.3 percent.

Table IX contains the results of the physical properties of the asphalt base course. A total of 18 sites had an asphalt base course. Asphalt content of these ABC courses ranged from 2.7 to 4.1 percent. It is noted that four sites were constructed with an open graded ABC, with aggregate up to 2.5 in. in diameter. These sites were 5, 6, 17, and 29. This base material was not tested. Site 27 also had an open graded ABC mixture on top of an ACHM pavement which was not tested. As a matter of record for site 27, the fifth layer description in Table V indicates ACHM. This layer was constructed as Job No. 2626 in 1962, and consisted of a surface and binder overlay over PCC. It may be presumed that this ACHM layer had reached its terminal serviceability in 1978 when the present surface was constructed. A similar situation with old asphalt pavement over PCC concrete is noted for sites 1, 5, and 6.

The average layer thickness and average resilient modulus test results for all pavement layers are shown in Table A1 in the appendix. The surface layer MR ranged from 151,000 psi at site 17 to 667,000 psi at site 12. The highest average MR measured was 812,000 psi for layer 2 of site 28. This binder mixture was taken from the oldest pavement sampled and was about 23 years old.

TABLE IX BASE LAYER PHYSICAL PROPERTIES

Site No.	R-S	Mr 1000 psi	Em 1000 psi	Marshall Stab. psi	Flow In.	Bulk Sp. Gr.	Max. Sp. Gr.	AC - %	Air Voids %	VMA - %
1	65-15	ALL BINDER MIX								
2	65-15	ALL BINDER MIX								
3	71-17	390	3.6	196	22	2.118	2.423	4.1	12.6	21.1
4	71-17	380	3.4	205	25	2.355	2.521	3.8	6.6	15.2
5	71-16B	OPEN GRADED ASPHALT BASE, OVER PCC								
6	71-16B	OPEN GRADED ASPHALT BASE, OVER PCC								
7	71-16	430	3.3	140	17	2.290	2.522	3.9	12.4	17.9
8	71-19	420	3.2	170	22	2.325	2.538	3.0	8.5	14.9
9	271-1	390	9.9	232	10	2.247	2.462	3.5	8.8	16.4
10	71-13	210	9.0	293	19	2.259	2.466	3.8	8.4	16.6
11	71-14	PORTLAND CEMENT CONCRETE PAVEMENT								
12	22-3	PORTLAND CEMENT CONCRETE PAVEMENT								
13	65-12A	400	8.5	200	10	2.288	2.470	3.8	7.4	15.7
14	65-12A	600	11.3	400	14	2.264	2.480	3.4	8.7	16.1

TABLE IX BASE LAYER PHYSICAL PROPERTIES (Continued)

Site No.	R-S	Mr 1000 psi	Em 1000 psi	Marshall Stab. psi	Flow In.	Bulk Sp. Gr.	Max. Sp. Gr.	AC - %	Air Voids %	VMA - %
15	167-12	460	8.3	258	14	2.227	2.475	3.8	10.0	18.1
16	70-9	NONE								
17	30-22	OPEN GRADED ASPHALT BASE, OVER PCC								
18	30-12	PORTLAND CEMENT CONCRETE PAVEMENT								
19	71-1	NONE								
20	71-2	520	7.0	271	16	2.211	2.487	3.4	11.1	18.4
21	82-2	NONE								
22	7-2	450	6.6	150	10	2.151	2.467	3.6	12.8	20.4
23	167-2	710	7.8	228	12	2.246	2.490	2.7	9.8	15.6
24	167-3	280	4.5	111	10	2.163	2.480	3.0	12.8	19.2
25	81-6	NONE								
26	65-20	PORTLAND CEMENT CONCRETE PAVEMENT								
27	65-18	PORTLAND CEMENT CONCRETE PAVEMENT								
28	82-11	NONE								

TABLE IX BASE LAYER PHYSICAL PROPERTIES (Concluded)

Site No.	R-S	Mr 1000 psi	Em 1000 psi	Marshall Stab. psi	Flow In.	Bulk Sp. Gr.	Max. Sp. Gr.	AC - %	Air Voids %	VMA - %
29	55-11	OPEN GRADED ASPHALT BASE, OVER PCC								
30	70-8	NONE								
31	1-8	NONE								
32	1-9	NONE								
33	49-9	NONE								
34	64-13	320	5.3	172	13	2.172	2.507	3.4	13.4	20.6
35	64-14	210	7.5	214	11	2.174	2.471	3.5	12.0	19.5
36	49-3	530	5.5	190	14	2.252	2.457	3.0	8.4	15.0
37	79-6	470	5.1	175	14	2.200	2.456	3.4	10.4	17.8
38	79-6	540	4.4	135	12	2.246	2.444	3.1	8.1	15.0

Note: Mr = Resilient Modulus, Em = Marshall Modulus, Flow in 0.01 Inches

The thickness of the surface layers reported in Table A1 ranged from 0.99 in. to 2.82 in. with an average thickness of 1.67 in. The surface layer thickness at sites 17, 18, 27, and 29 include a 50 PSY plant mix seal.

One area of the MR test results that needs further study is the effect of specimen thickness on the measured MR values. The Retsina device used to measure MR values was designed to accomodate specimen thicknesses from 1.5 to 3 in. Surface layers from 12 sites were less than 1.5 in. thick, and 7 binder layers were also less than 1.5 in. thick. The effect of this deviation in thickness upon resilient modulus readings is unknown.

Pavement Classification, Thickness and Deflection

The average thickness of the asphalt pavement and the average Dynaflect deflections for all sites are shown in Table X. The pavement deflection is a function of the subbase and subgrade support and the pavement structure. Each site is identified by its type of construction in Table X.

The type of construction classification is based on the subgrade support factors. Type X1 pavements are ACHM placed over a granular base. Type X2 pavements are overlayed type X1 pavements. The average thickness of the nine type X pavements was 5.4 in. Type Y1 pavements are ACHM placed over ABC, while type Y2 pavements are overlayed type Y1 pavements. The average thickness of the 17 type Y pavements was 11.5 in. Type Z pavements were supported by

TABLE X DYNAFLECT DEFLECTIONS AND TOTAL PAVEMENT THICKNESS

Site No.	R-S	Type Const.	ACHM Inch	Dynaflect Reading (mils x 100)* D1	D2	D3	D4	D5
1	65-15	Z3	9.9	36	34	29	26	23
2	65-15	Y2	12.7	59	53	41	34	27
3	71-17	Y1	14.7	52	45	34	28	23
4	71-17	Y1	17.6	33	30	24	22	20
5	71-16B	Z3	8.3	40	34	23	18	13
6	71-16B	Z3	9.6	32	26	17	13	10
7	71-16	Z3	9.4	40	25	14	8	6
8	71-19	Y1	14.2	42	32	20	14	11
9	271-1	Z3	9.3	61	45	26	14	9
10	71-13	Y1	9.2	54	37	19	12	9
11	71-14	Z1	4.7	57	46	33	23	17
12	22-3	Z1	3.3	70	32	10	5	4
13	65-12A	Y1	12.0	58	50	35	27	20
14	65-12A	Y1	11.8	63	54	37	28	21
15	167-12	Y1	10.8	90	76	52	38	29
16	70-9	X2	3.8	46	24	8	5	4
17	30-22	Z2	9.3	24	20	14	13	12
18	30-12	Z2	5.7	37	35	29	27	24
19	71-1	X2	7.4	41	29	16	12	9
20	71-2	Y2	9.2	46	36	21	15	12
21	82-2	X2	10.9	94	86	72	59	51
22	7-2	Y1	8.5	90	68	44	32	26
23	167-2	Y2	9.7	55	47	32	23	18

TABLE X DYNAFLECT DEFLECTIONS AND TOTAL PAVEMENT THICKNESS
(Concluded)

Site No.	R-S	Type Const.	ACHM Inch	Dynaflect Reading (mils x 100)*				
				D1	D2	D3	D4	D5
24	167-3	Y2	12.2	38	34	26	22	20
25	81-6	X1	5.2	126	98	67	48	36
26	65-20	Z1	4.4	74	71	63	55	49
27	65-18	Z3	9.9	52	47	40	35	31
28	82-11	X2	3.1	146	104	75	64	55
29	55-11	Z2	9.9	36	35	31	30	29
30	70-8	X1	5.6	107	60	24	13	8
31	1-8	X2	5.1	115	86	56	40	32
32	1-9	X2	4.0	87	68	49	39	32
33	49-9	X1	3.3	94	65	41	31	25
34	64-13	Y1	11.3	98	74	50	33	24
35	64-14	Y1	10.8	119	89	54	37	24
36	49-3	Y2	7.2	79	62	40	28	23
37	79-6	Y1	11.2	75	66	51	41	35
38	79-6	Y1	13.1	73	65	49	39	32

Note: Type of Construction Reflects Base Support Factors.

X1 = ACHM Over Granular Base, X2 = ACHM Over Asphalt Pavement

Y1 = ACHM Over Asphalt Base, Y2 = Type Y1 Over Asphalt Pavement

Z1 = ACHM Over PCC Pavement, Z2 = Type Y1 Over PCC Pavement

Z3 = Type Y2 Over PCC Pavement

* 1 Mil = 1/1000 inch

an underlying layer of PCC pavement. Type Z1 pavement was ACHM placed over PCC pavement, and each suffix to the Z letter indicates another ACHM layer. The average thickness of the 12 type Z pavements was 7.8 in.

No particular significance is placed on the type of construction classification at this time. The pavement types may be analyzed for their field performance by grouping them into the above classifications. A like analysis may be performed based on traffic classification, rut depth, resilient modulus, air voids, pavement condition rating or performance.

The average D1 deflections by type of construction are: X, 0.95 mils; Y, 0.66 mils; and Z, 0.47 mils. The pavement thickness ranged from 3.1 in. at site 28 to 17.6 in. at site 4. The maximum deflection of 146 (0.00146 in.) was obtained at site 28, with the smallest deflection of 24 (0.00024 in.) being recorded at site 17. Sensor D1 is under the load while sensor D5 is 4 ft. from the load. The greatest deflection at D5 was 51 for site 28, with the minimum D5 deflection of 4 being recorded at sites 12 and 16.

Application of elastic layer theory along with computer analysis may explain the relationship between deflection and pavement thickness for the different types of construction. Likewise, the analysis to determine the stresses (and strains) developed in the asphalt layers may explain the relationship between fatigue and the resulting crack development (13, 14, 33, 34). The magnitude of the pavement deflections that were obtained indicated no significant relationship with pavement performance or measured pavement characteristics.

Aggregate Gradations

The average aggregate gradations from the extracted cores are shown in Table A2 in the appendix. The 95 gradations shown were obtained from about 285 extractions of core layers. The type of mineral aggregate used and the construction Job No. is also shown in the table. The average gradation for each type of mixture is shown at the end of Table A2. The general distribution of the types of mixtures extracted by number and maximum particle size was: type 3 surface (-0.5 in.), 14; type 2 surface (-0.75 in.), 32; binder (-1.0 in.), 32; base (-1.12 in.), 5; and base (1.5 in.), 12.

It was observed during the sieve analysis that some of the larger aggregate particles showed signs of being sawed by the core bit. The process of taking the pavement cores with the 4 in. diameter core bit caused a degradation of the aggregate. The true degradation of the aggregate due to coring is unknown. However, the comparison of available job mix gradations with the extracted gradations indicate that the coring operation may have increased the percentage passing the coarser sieves from 2 to 5 percent. The degradation of aggregate due to coring also resulted in different maximum mixture specific gravities between core samples from the same site.

Performance and Mix Characteristics

As suggested earlier, to compare various pavement mix parameters the data may be sorted into different groups. This procedure will permit the evaluation of meaningful relationships between the asphalt mixture characteristics and the performance of the pavement. Possible divisions of data may be as follows: a) divide pavements into groups based upon the previously described types of construction, X, Y and Z; b) divide into groups based upon a traffic classification of: light, medium and heavy; or c) divide into groups based upon measured pavement performance, such as, good, average and poor.

Preliminary investigation of the possible grouping of data indicated that the best approach would be a division of test sites into groups based upon their performance. A division of sites was made based upon condition rating and the Mays rideability values in relation to the total number of EAL's. Pavement sites with a high ratio of EAL's to decrease in Mays rideability (dMay) were classed as good. The best performing ten sites were: 3, 4, 8, 10, 26, 27, 28, 33, 34, and 36. These sites had an average of 1002 EAL's (x1000) with a dMay from 100 to 87.6, giving a ratio of 81. Sites with a low ratio of EAL's to decrease in Mays rideability were classed as poor. The lowest performing ten sites were: 1, 2, 5, 6, 9, 15, 22, 24, 25 and 30. These 10 sites had an average of 560 EAL's (x1000) with a dMay from 100 to 55.1 for a ratio of 12. Sites 11, 17 and 29 were deleted from this analysis because of variability of the test data obtained at these sites. The 33 sites remaining, including both good and bad sites, had an average of 1054 EAL's (x1000) with a dMay from 100 to 72.2 giving a ratio of 38.

Asphalt mixture properties found to have significant relationship with pavement rutting and cracking include: resilient modulus, air voids and stability. Mays meter values were also found to be related to pavement rutting and cracking. Stepwise linear regression was used to determine the best fitted equation for each dependent variable and their relationship with other mix characteristics and performance parameters. The data analysis was performed on the University of Arkansas IBM 360/370 computer using the CMS/SAS system. The six best linear equations that follow are based upon the pavement surface evaluation and surface layer properties previously presented for the 33 sites of this group.

Mays meter (MM) value was affected by the amount of rutting and cracking as given by equation 1.

(EQ 1) $MM = 104.9 - 2.23\ RUT - 8.07\ CI$

where: RUT = rut depth, 1/32 inch

CI = cracking index, class

A rut depth of 10/32 in. and crack index of 1.0 would indicate a Mays meter value of 74.5. The coefficient of determination, R square, is 0.605 and the standard error of estimate (RMSE) is 11.2 percent MM.

The crack index (CI) was related to rut depth, average air voids and resilient modulus as shown by equation 2.

(EQ 2) $CI = -0.171 + 0.00257\ MR + 0.213\ AAV - 0.0155\ RUT$

where: MR = resilient modulus, psi x 1000

AAV = average air voids, percent

RUT = rut depth, 1/32 inch

An MR of 300,000 psi, AAV of 3 percent, and a RUT of 10/32 in. would give a crack index of 1.1. R square equals 0.472 with a RMSE value of 0.69 CI for this equation.

The resilient modulus (MR) relationship to Marshall stability, flow and voids filled is given in equation 3.

(EQ 3) $MR = 1138 + 1.73\ STAB - 28.7\ FLOW - 9.0\ VF$

where: STAB = Marshall stability, psi

FLOW = Marshall flow, 1/100 inch

VF = voids filled, percent

With a Marshall stability of 1500 pounds, flow of 8 and voids filled of 80 percent, the resilient modulus would equal 448,000 psi. R square equals 0.651 with a RMSE of 80.8 or 80,800 psi MR.

The average air void (AAV) content was found to be related to Marshall stability and VMA as shown in equation 4.

(EQ 4) $AAV = -8.49 + 0.00925\ STAB + 0.624\ VMA$

where: STAB = Marshall stability, psi

VMA = voids in mineral aggregate, percent

With a Marshall stability of 1500 pounds and a VMA of 15 percent, the air voids equal 2.3 percent. R square equals

0.808 with a RMSE of 0.65 percent AAV.

The average air voids were also found to be related to VMA and percentage asphalt as shown in equation 5.

(EQ 5) AAV = -1.91 + 1.13 VMA - 2.28 PAC

where: VMA = voids in mineral aggregate, percent

PAC = asphalt content, percent

For a mixture with a VMA of 15 percent and 5.1 percent asphalt, the air voids are 3.4 percent. R square equals 0.968 with a RMSE of 0.26 percent AAV.

Marshall stability (STAB) is related to resilient modulus and flow in equation 6.

(EQ 6) STAB = -26.2 + 12.0 FLOW + 0.207 MR

where: FLOW = Marshall flow, 1/100 inch

MR = resilient modulus, psi x 1000

With a flow of 9 and MR of 270,000 psi the Marshall stability would be 138 psi of 1380 pounds. R square equals 0.690 with a RMSE of 28.8 or 288 pounds STAB.

The coefficient of correlation for equations 1 through 6 were found to be highly significant. Likewise, the regression equations shown on Figures 4, 5, 6, 7, 8, 9, and 10 had coefficients of correlations that were found to be highly significant. It is also noted that the standard error of estimate is given for each best fitted equation shown on each figure. For plotting purposes the 95 percent confidence limits were intentionally left off the figures. The approximate 95 percent confidence interval may be easily obtained by using as limits the Y value plus and minus two times the standard error of estimate.

Of particular interest is the performance and mix characteristics of the ACHM pavements grouped by: all 33 sites, 10 good sites and 10 poor sites. These values are shown in Table XI. The comparison of mix characteristics between good and poor pavement sites needs to be prefixed with the understanding that most of the test sites were well designed and constructed ACHM pavements. Some of the "poorer" pavements may in fact indicate better mix characteristics in some respects than the "good" pavements.

The relationship between some of the different variables may be visualized in conjunction with the graphs presented in Figures 4 through 10. Thus, the range in mix characteristics that indicated good performance include: air voids, 2 to 5 percent; voids filled, 75 to 85 percent; voids in the mineral aggregate, 13 to 15 percent; Marshall modulus, 6000 psi minimum; and Marshall stability, 160 psi minimum. The data also indicates that with the above mixture characteristics a rut depth of 8/32 in. and a crack index of less than 1.0 may be expected.

As an example of using the Marshall test data to estimate pavement performance, asssume an inplace asphalt mixture placed on a well designed and constructed base with the following physical characteristics: Marshall stability 1600 pounds, flow of 10, air voids of 3 percent and asphalt content of 5.2 percent. From the equation of Figure 9, the voids filled is 82 percent; from equation 5, the VMA is

TABLE XI AVERAGE TEST VALUES BY GROUP

No.	Item	ID.	Units	SITES All	SITES Good	SITES Poor
1.	Mays Value	MM	%	72.2	87.6	55.1
2.	Rut Depth	RUT	1/32 in.	10.6	7.5	15.6
3.	Crack Index	CI	class	1.1	1.0	0.9
4.	Resilient Modulus	MR	1000psi	376	404	305
5.	Average Air Voids	AAV	%	2.4	2.4	2.1
6.	Marshall Modulus	EM	1000psi	6.8	7.2	6.0
7.	Marshall Stability	STAB	psi	185	192	163
8.	Voids in Min. Agg.	VMA	%	14.6	14.8	14.8
9.	Asphalt Content	PAC	%	5.4	5.4	5.5
10.	Marshall Flow	FLOW	1/100 in.	11.1	10.9	10.7
11.	Acc. 18K Axles	EAL	No.x1000	1054	1002	560
12.	Dust Ratio *	DR	%	1.60	1.54	1.58

* Dust Ratio = % minus #200 divided by Percent Asphalt

14.8 percent; from equation 3, the MR is 390,000 psi; from the equation of Figure 7 the rut depth is 7/32 inch; from equation 2, the crack index is 1.4 and from equation 1 the Mays Meter rideability is 79 percent. Using the ratio of EAL to dMays of 81 (for a good pavement), then a total of 1,700,000 EAL's would cause a reduction in the rideability from 100 to 79 percent.

Laboratory Mixtures

Laboratory specimens were prepared using the Marshall method of molding and testing in accordance with ASTM D 1559 (22). The laboratory mixtures were simillar in gradation to the asphalt pavement cores. All specimen were molded using a single source AC-30 viscosity graded asphalt cement. Generic aggregate types used in these mixtures include: limestone, sandstone, syenite and gravel. Mixtures were prepared using three different aggregate gradation limits: Mix A was an Arkansas type 2 surface mix (-3/4" top size), Mix B was an Arkansas type 2 binder mix (-1" top size) and Mix C was a base mix (-1.5" top size).

Tests performed on these laboratory mixtures include: compacted bulk specific gravity (ASTM D 2726), maximum mixture specific gravity (ASTM D 2041), resilient modulus, and Marshall stability and flow. Voids analysis was performed as per the procedure given in The Asphalt Institute MS-2 (39). The aggregate effective specific gravity was used in calculating air voids and voids in the mineral aggregate. Triplicate specimen were prepared for each asphalt content and the test results are summarized for 78 sets of laboratory samples in Table B1 in the appendix.

Stepwise linear regression of the test results was used to obtain the relationships between resilient modulus and the other physical properties of the laboratory molded samples. Siginificant relationships were obtained between the resilient modulus (MR), Marshall stability (STAB), Marshall flow (FLOW), asphalt content (AC) and air voids (AV). The regression analysis coefficient of determination, R square, ranged from 0.59 to 0.98 with various combinations of test results.

The best fitted equation (EQ 7) using all 78 data points for the surface, binder and base mixtures, is:

$$MR = 598 + 0.856\ STAB - 92.7\ AC - 11.6\ AV$$

This equation indicates that resilient modulus increases with stability and decreases with asphalt content and air voids. With a stability of 195 psi, asphalt content of 4.8 and air voids of 3.8 a resilient modulus value of 276 or 276,000 psi is obtained. An R square value of 0.621 was obtained, with a standard error of estimate (RMSE) of 54.7 or 54,700 psi MR.

Using the 40 data points for an Arkansas Type 2 surface mix at 50 blows compaction, the best fitted equation (EQ 8) relating the mix properties is:

$$MR = 424 + 0.956\ STAB - 63.3\ AC - 14.5\ AV$$

An R square value of 0.744 was obtained for this equation, with a RMSE of 34.8 or 34,800 psi MR. Using the 9 data points for an Arkansas binder mix, the best fitted equation (EQ 9) is:

$$MR = 263 + 27.1\ EM + 30.6\ AC - 29.0\ AV$$

R square equals 0.889 with a RMSE value of 26.5 or 26,500 psi MR. With the 19 data points for the black base design evaluation, the best fitted equation (EQ 10) is:

$$MR = 660 + 0.816\ STAB - 98.6\ AC - 15.1\ AV$$

An R square value of 0.793 was obtained for this relationship, with a RMSE of 47.5 or 47,500 psi MR. The combination of data from the 10 samples molded at 75 blows indicated an R square value of 0.980. The equation (EQ 11) for this data is:

$$MR = -2490 + 3.17\ STAB + 320\ AC + 144\ AV$$

The RMSE value was 26.5 or 26,500 psi MR. These 10 data points were obtained using a limestone aggregate with specimen of simillar combined grading for an Arkansas type 2 mix.

These equations may be used to estimate the resilient modulus values for a laboratory molded Marshall job mix design. The coefficient of correlations for equations 7 through 11 were very significant. The characteristics of the asphalt cement used in the job mix will directly influence the resilient modulus. The effect of aggregate gradation is indicated by the different equations obtained for resilient modulus as shown above. Equations 7 through 11 were obtained using an AC-30 viscosity graded asphalt cement.

One other equation obtained using the labortory test vaules related the air voids, asphalt content and voids in the mineral aggregate (VMA). By stepwise linear regression, the best fitted equation (EQ 12) was:

$$AV = -1.78 + 1.14\ VMA - 2.38\ AC$$

The R square values for the 78 data points used in the above equation was 0.974, with a RMSE value of 0.43 percent AV.

The effect of aggregate gradation on the relationship between AV, VMA and AC may be obtained by combining the test results of smiliar mixtures and performing the stepwise linear regression.

With the 40 data points from an Arkansas type 2 surface mix molded at 50 blows compaction, equation 12A is:

$$AV = - 1.71 + 1.15\ VMA - 2.42\ AC$$

R square equals 0.988, with a RMSE of 0.21 percent.
With the 9 data points for an Arkansas binder mix, the best fitted equation 12B is:

$$AV = -1.54 + 1.16\ VMA - 2.48\ AC$$

R square equals 0.972, with a RMSE of 0.33 percent.
Using the 19 data points for the black base mixes, the best fitted equation 12C is:

$$AV = 4.95 + 1.16\ VMA - 1.59\ AC$$

R square equals 0.945, with a RMSE of 0.76 percent.
The 10 data points for the 75 blow compaction of the Arkansas type 2 surface mix gave a best fitted equation 12D

of:

$$AV = -2.54 + 1.22\ VMA - 2.48\ AC$$

R square equals 0.99, with a RMSE of 0.05 percent. These equations, 12 through 12D, relating asphalt content, voids in the mineral aggregate and air voids may be used with confidence for asphalt mixtures having simillar compositions to those used in this study.

It is of interest to compare the predicted air voids obtained for the laboratory mixtures and field cores. Using an asphalt content of 5.0 percent and a VMA of 14.0 and Equation 5 (for cores), the AAV equals 2.5 percent. Using Equation 12A (for laboratory mixes), the AV equals 2.3 percent. This calcuation indicates that a pavement mix will have slightly more air voids than the corresponding laboratory mix, after compaction by traffic. The mixture VMA and asphalt content may be adjusted to obtain a desired air void content, using equations 12A, 12B, 12C or 12D, depending upon the type of mixture.

Pavement Initial Resilient Modulus

Marshall job mix design data for the surface layer of 30 test sites were available from the AHTD. The specific job mix design actually used in construction was not determined. Most of the jobs had several different proposed job mixtures, and in some cases more than one job mix was used in the construction of different pavement segments. Construction reports that would indicate the actual asphalt cement and aggregate used in the construction were available for only a few test sites. These AHTD laboratory job mix designs included 20 Arkansas type 2 mixes (top size 3/4 in.) and 10 Arkansas type 3 mixes (top size 1/2 in.). The asphalt cements used in these job mixes, by type asphalt and site number, respectively, were: grade 60-70, sites 3, 7, 9, 10, 12, 13, 14, 15, 19, 20, 26, 28, 29, 33 and 36; grade AC-20, sites 1, 2, 5, 6, 22, 27, 30, 32, 37 and 38; grade AC-30, sites 8, 23, 34 and 35; and grade AC-40 at site 31.

The initial resilient modulus for each job mix was calculated by use of equation 8. This equation was developed from mixtures made with an Arkansas type 2 surface mix. The increase in MR per year was then estimated by subtracting the initial MR from the measured core MR and dividing by the number of years in service. Analysis of the data for the 30 job mixes gave an average MR increase of 35,000 psi per year. The range in values were from 2200 psi to 142,000 psi. Regression analysis of all 30 data points, relating age (in years) versus increase in MR (dMR) gave this relationship:

$$AGE = 3.03 + 0.0306\ dMR$$

The R value was 0.555, which explains only about 31 percent of the relationship of dMR with age. This equation indicates that MR increases with age of the pavement, but the equation may be only used as a trend line, because of the scatter of data.

To determine the effect of the grade of asphalt in the mixture upon the change in MR, the 19 sites having mix designs using grade 60-70 and AC-30 were grouped together for regression analysis. It is thought that these asphalt cements were of simillar penetration to the AC-30 used in preparation of the project laboratory mixes. Ten of these jobs were an Arkansas type 3 surface mix and 9 jobs were for an Arkansas type 2 surface mix. The best fitted equation is:

$$AGE = 4.65 + 0.0364\ dMR$$

A coefficient of correlation of 0.728 was obtained for this relationship. The equation indicates an average increase in MR of about 28,000 psi per year of service. This grouping of data for jobs thought to be constructed with simillar asphalt cements improved the estimation of change in resilient modulus with age. The effect of the different aggregate gradations on the MR for these 19 sites is unknown. For further analysis, the relationship of MR with the physical properties of an Arkansas type 3 surface mix is needed.

It is concluded that the effect of asphalt cement properties and aggregate grading on the MR of both laboratory and pavement cores is needed in order to obtain a more accurate estimate of the increase (or decrease) of MR under service conditions. The physical properties of the asphalt cement used in the initial construction are needed in order to obtain a more exact relationship of change in resilient modulus with time.

Summary

The average values of the field test results are presented in Table IV and X. These results include pavement condition rating, Mays rideability, skid value, crack classification, rut depth, and Dynaflect deflection.

Average physical properties of the cores are presented in Tables V, VI, VII, VIII, IX, A1, and A2. These results include description of the pavement layers, aggregate type and gradation, resilient modulus, bulk specific gravity, maximum specific gravity, percent asphalt, air voids, voids in the mineral aggregate, and Marshall modulus, stability, and flow.

Regression analysis was performed to determine the relationships between field and laboratory data. The variables correlated against each other included pavement thickness, Dynaflect Spreadability Index, total EAL, crack index, pavement condition rating, Mays rideability, maximum rut depth, resilient modulus, Marshall modulus, Marshall stability, air voids, voids in the mineral aggregate, and percent asphalt content. Air voids and resilient modulus of the surface layer were also correlated with the pavement age, Dynaflect D1 (OWP and IWP), rut depth (OWP and IWP), and voids filled with asphalt.

Graphs relating the surface layer resilient modulus with voids filled, Marshall stability, air voids, and Marshall modulus have been shown. Graphs showing the relationships between surface layer air voids versus IWP rut depth, voids in the mineral aggregate, and voids filled have been presented. The best fitted equation along with the coefficent of correlation and standard error of estimate are indicated on each graph.

Mix characteristics and pavement performance were related with each other by equations 1 through 6. Asphalt pavement mixtures may be analyzed by the use of these equations to predict pavement performance of the designed ACHM.

Test results of the Marshall laboratory mixtures made to simulate the pavement cores are shown in Table B1. The relationship of resilient modulus with physical properties of these laboratory mixtures are shown in equations 7 through 11. The relationship between air voids, voids in the mineral aggregate and asphalt content are shown in equations 12 through 12D for various aggregate gradations.

The pavement initial resilient modulus for 30 test sites was estimated. Equations are presented to relate the effect of aging on the resilient modulus of the pavement surface layer.

CHAPTER V

BLACK BASE MIX DESIGN

The physical characteristics of the 18 black bases under evaluation have been reported in Table IX. The gradations of these black bases are shown in Table A2 of the appendix. The results of the laboratory mix designs using mixtures representative of the better black base will follow. A better black base is one that provides support to the surface and binder layers that results in superior pavement performance, with other factors being equal. In general, the "good" pavements discussed in Chapter IV had better bases than the "poor" pavements. The bases of sites 3, 4, 8, 10, 13, 23, and 36 were considered to be good. As a contrast, the bases of sites 14, 15, 24, 34, 35 and 38 were considered to be poor. A rational method of black base design along with mix design criteria for selecting the optimum asphalt content is presented.

Base Mix Design

The laboratory mix designs were prepared in accordance with the Marshall Mix Design Method (39). The base mix designs were prepared using these four aggregate types: limestone, sandstone, gravel and syenite. A Tosco AC-30 paving grade asphalt was used in all of the laboratory mixtures. A voids analysis was performed using the aggregate effective specific gravity to calculate the air voids, voids in the mineral aggregate and voids filled. The Marshall specimens were tested in the same manner as was previously presented for the pavement core samples.

The aggregate gradations were selected to represent simillar gradations for the better black bases for each aggregate type under investigation. All of the gradations selected met the requirements of the AHTD specifications (40) for aggregate used for base construction. The mix design gradations used for each aggregate type along with the specification limits for both an SB-2 and GB-3 aggregate are shown in Table XII.

Triplicate samples were prepared at each preselected asphalt content for all four aggregate types. The asphalt contents were varied by 0.5 percent to define the Marshall optimum asphalt. In order to use the Marshall mix design procedure, the plus one inch material was removed from each mixture prior to addition of the asphalt cement. After mixing and molding at 50 blows per side the specimen were cooled overnight prior to determination of their height and bulk specific gravity. Next, the resilient modulus of each specimen was determined using the test procedure previously described in Chapter III. The specimen were then tested for their Marshall stability and flow in accordance with ASTM D 1559 (22). Optimum asphalt content was estimated for these designs and additional samples of sandstone, syenite and gravel mixtures with the plus one inch material were prepared at the optimum asphalt content and tested as

TABLE XII BASE MIX DESIGN GRADATIONS

Sieve Size	Total Percent Passing									
	Limestone		Sandstone		Syenite		Gravel		AHTD Spec.*	
	+1"	-1"	+1"	-1"	+1"	-1"	+1"	-1"	SB-2	GB-3
1.5	100	100	100	100	100	100	100	100	100	100
1.12	97	100	97	100	95	100	97	100	---	---
1	94	100	94	100	89	100	95	100	---	
3/4	80	85	80	85	69	78	84	88	50-90	60-100
1/2	61	65	60	64	54	61	68	72	---	---
3/8	53	56	50	53	49	55	54	57	---	40-80
4	40	43	36	38	43	48	38	40	25-50	30-60
10	27	29	26	28	37	42	28	29	---	20-45
20	20	21	18	19	30	34	23	24	---	---
40	15	16	14	15	24	27	18	19	10-30	10-35
80	9	10	11	12	9	10	12	13	---	---
200	5	5	5	5	4	5	6	6	3-10	3-12

* reference 40

above.

In order to further delineate the effects of the plus one inch material on the results of the Marshall mix design procedure, addition testing was performed on the limestone mixtures. Samples were prepared of the original limestone mixture gradations. The plus one inch material was not removed, but was used in the mixture. The asphalt content of these additional limestone mixes was the same as used in the initial limestone mixes. The laboratory specimens were compacted at 50 blows per side and tested using the Marshall procedure.

The detailed results of the Marshall mix design tests performed and analysis of data are reported by Cross (41). The average test results for these base mixtures are shown in Table B1 in the appendix. The limestone (ls) mixes are ID # 57-62; the plus 1 in. ls mixes are ID # 60-62. The syenite (ns) mixes are ID # 66-70; the plus 1 in. ns mix is ID # 69. The sandstone (ss) mixes are ID # 71-74; the plus 1 in. ss mix is ID # 73. The gravel (gvl) mixes are ID # 75-78; the plus 1 in. gvl mix is ID # 77.

The physical characteristics of the laboratory mix designs at optimum asphalt content and the average values of the field cores, by aggregate type, are shown in Table XIII. The optimum asphalt content was calculated using the criteria for base mixes by The Asphalt Institute (TAI) method (39).

In comparison with The Asphalt Institute criteria

TABLE XIII BASE MIX DESIGN VALUES VS. FIELD CORE VALUES

Aggregate Type	AC - (%)	MR psi 1000	Marshall STAB (psi)	Marshall FLOW 1/100	AV - (%)	VF - (%)	VMA - (%)	Bulk Sp. Gr.
Limestone								
Field core	3.7	385	178	21.5	9.4	47.7	17.7	2.267
Laboratory(-1")	3.5	350	220	10.0	4.3	66.0	12.6	2.429
Laboratory(+1")	3.8	310	238	13.3	2.8	77.0	11.8	2.458
Sandstone								
Field core	3.6	280	228	13.2	10.6	37.8	23.0	2.213
Laboratory(-1")	4.1	150	152	9.0	7.8	54.0	16.8	2.269
Syenite								
Field core	3.6	480	286	12.7	8.7	48.0	16.7	2.260
Laboratory(-1")	4.0	260	188	6.5	9.3	48.0	18.0	2.245
Gravel								
Field core	3.3	500	180	12.6	9.9	42.0	17.0	2.221
Laboratory(-1")	4.3	260	135	10.5	3.7	73.0	13.5	2.354

Typical Design Criteria:

1) The Asphalt Institute mix design criteria for a 50 blow base mixture are: Marshall stability = 500 lb. minimum (50 psi); Flow (1/100 in) = 8 to 18; Air voids = 3 to 8 percent and VMA = 13 percent minimum. (39)

2) The Association of State Highway and Transportation Officials mix design criteria for a 50 blow base mixture are: Marshall stability = 500 lb. minimum (50 psi); Flow (1/100 in.) = 8 to 18; Air Voids = 3 to 11 percent; and Voids Filled = 65 to 75 percent. (35)

shown in Table XIII, the sandstone and gravel mixes met all of the requirements, while the VMA of the limestone mix was slightly below the minimum value. The syenite mix had air voids greater than 8 percent with a flow of 6.5 and does not meet the TAI criteria.

In comparison with the American Association of State Highway and Transportation Officials (AASHTO) criteria (35) shown in Table XIII, the limestone and gravel mixes met all of the requirements. The sandstone, syenite and gravel mixes met all of the AASHTO requirements except for having too few voids filled.

The results of the mix design value at optimum asphalt content for the limestone aggregate with the plus one inch aggregate included in the 4 inch Marshall mold are also shown in Table XIII. The stability values for the plus one inch mix, at optimum asphalt content, are slightly higher than for the minus one inch mix. It should be noted that the stability values for the plus one inch mix were very erratic and because of this, selection of the optimum asphalt content was somewhat suspect. Erratic values for the plus one inch mix were also encountered in the air voids, voids in the mineral aggregate and bulk specific gravity plots as well. The optimum asphalt content for the plus one inch mix was 0.3 percent higher than for the minus one inch mix.

<u>Base Laboratory Values Compared with Field Core Values</u>

The comparison of the field core values, by aggregate type and the mix design values are also shown in Table XIII. The sandstone, syenite and gravel mix designs all had higher optimum asphalt contents, equal or higher percent voids filled, and lower air voids than the average of the field cores.

The limestone mix design had a lower optimum asphalt content than the average of the limestone field cores. The bulk specific gravities were higher for the mix designs in every case except for the syenite mix, which was lower. As expected, the resilient modulus values of the cores were higher in each case than those of the laboratory mixes.

Rational Design Method for Arkansas Black Bases

The following rational mix design method for Arkansas black bases is presented. The method is based on the analysis of data for peformance and mix characteristics of Chapter IV, along with the black base mix design test data, and the preceeding discussion.

The Marshall method of mix design (39), with the following modifications, will provide good black base job mixes. The gradation of the black bases should be similar to the "good" bases of the test sites. The grading limits need to be more closely controlled than presently used for SB-2 and GB-3 mixes. The recommended grading for black bases by sieve size and percentage passing, respectively is: 1 1/2", 100%; 1", 94%; 1/2", 62%; No. 4, 37%; No. 10, 23%; No. 40, 15%; No. 80, 9%; No. 200, 4%. The grading band should be set to the meet these percentages based on the present tolerances used for ACHM.

After the trial grading is selected, the plus 1 inch material should be removed from the trial mix and a regular Marshall mix design procedure employed to obtain the optimum asphalt content. This asphalt content then should be decreased approximately 0.1 percent (on total weight basis) to provide sufficient asphalt to coat the plus 1 inch material. This decrease in asphalt content is necessary because the surface area of the plus 1 inch mix is less than the surface area of the minus 1 inch mix.

The recommended criteria for the Marshall asphalt content for an Arkansas black base is as follows. Marshall stability of 1500 pounds minimum, flow of from 6 to 14, air voids of from 3 to 8 percent, VMA of from 11 to 13 percent minimum depending upon air void content, and optimum asphalt content. The relationship shown by equation 12D of Chapter IV may be used to determine the proper VMA for any desired air void and asphalt content combination. This type of base mixture would be suitable for all levels of traffic if sufficient thickness of pavement structure is provided for the design EAL.

CHAPTER VI

CONCLUSIONS AND RECOMMENDATIONS

On the basis of the experimental work covered by this report and within the limitations of the test procedures, materials, and conditions utilized in this investigation, the following conclusions are warranted:

1. There is an identifiable relationship between the resilient modulus and the other physical characteristics of the pavement core. The best relationship of resilient modulus with the core physical properties is shown by equation 3 and equation 6. The independent variables in these equations are Marshall stability, flow and voids filled with asphalt. These equations have an R square value of 0.68, which would indicate that about 32 percent of the variation of pavement resilient modulus was caused by other factors. These other factors include asphalt content, voids in the mineral aggregate, air voids, asphalt characteristics and aggregate characteristics.

2. The relationships obtained between the field performance evaluations and asphalt pavement properties are presented in equations 1 and 2, and the equations shown on Figures 7 and 9. These performance and mixture variables include: Mays Meter value, rut depth, crack index, resilient modulus, air voids and voids filled with asphalt. By use of these equations the performance of an asphalt

pavement under varying levels of traffic may be predicted.

3. The data indicates that the pavement resilient modulus increates with time, as the rate of about 30,000 psi per year of service. This change in resilient modulus is shown to increase the amount of cracking by equation 2, when air voids and rut depths are held constant.

4. The resilient modulus of the laboratory molded samples was related to Marshall stability, flow, asphalt content and air voids by equations 7 through 11 for different types of asphalt mixtures. Asphalt mix designs may be evaluated by use of these relationships to relate mix properties with pavement performance, as shown in conclusion 2, prior to actual pavement construction. The resilient modulus of the laboratory asphalt mixture and the aging pavement resilient modulus may be used by the Design Engineer as a key to the design of superior asphalt pavements.

5. An excellent relationship between air voids and voids filled was obtained as shown in Figure 9. The AASHTO recommendations for voids filled in a surface course is 75 to 85 percent for daily EAL's from 50 to 500 (35). It is concluded that air voids between 2.3 and 4.5 percent will meet this voids filled criteria.

6. The air voids in the pavement is indicative of the measured rut depth in the inner wheel path as shown in Figure 7. Air voids over 2.5 percent were associated with rut depths of 10/32 inch or less. The correlation

coefficient of 0.568 between these variables indicates that other factors, such as traffic, subgrade support, stability, aggregate gradation, and asphalt cement properties need to be considered for control of pavement rutting.

7. The relationship between air voids, voids in the mineral aggregate and asphalt content for pavement cores is shown by equation 5. For laboratory mixtures these variables are related to one another in equation 12A and 12D for an Arkansas type 2 surface mix. For binder and base mixtures, the relationships are shown in equation 12B and 12C. It is concluded that VMA is not an independent variable in asphalt mixtures but is dependent upon air voids and asphalt content. These equations (5, 12A, 12B, 12C and 12D) may be used to analyze asphalt mixtures and to adjust the asphalt content and voids in the mineral aggregate for any desired air void content.

8. A rational mix design for Arkansas Black Bases is presented. The method will permit the Design Engineer to select the aggregate gradation and asphalt content to provide excellent asphalt bases, using the Marshall mix design procedure and equipment.

Recommendations

1. The analysis of the data of this study on the basis of type of constuction may indicate that different controlling factors should be used in design of asphalt pavement mixtures for each construction type. For example,

a much stiffer pavement structure may be desirable for PCC overlays than is needed for conventional designs of ACHM over a granular base. Therefore, this type of analysis is recommended.

2. The determination of the pavement residual asphalt physical properties and their relationship to resilient modulus and Marshall mix design factors is highly recommended. This evaluation may indicate which physical characteristics of the asphalt cement best relates to the different parameters of pavement performance. To improve the performance capability of the pavement a change in the character of the asphalt cement used for different layers or types of construction may be dictated from this work.

3. Since pavement deflection is a viable estimate of pavement performance, the determination of pavement deflections under 18 to 22 kip axle loads for these pavement sites of known physical characteristics is warranted. Measurement of the pavement deflection basin with heavy axle loads would permit deflections to be related to the physical properties of the pavement structure. The knowledge of the pavement deflection of a pavement structure under heavy wheel loads would permit more economical design of asphalt pavements.

4. It is recommended that a more basic unit of measurement of pavement roughness, such as roughness in inches per mile, be used for pavement roughness evaluation, rather than the percent Rideability that is reported

herein. The inches per mile unit of roughness measurement could then be used for comparison with results of other research agencies. The actual roughness of the pavement test sites rather than the relative roughness value reported may yield more useful correlations, in particular with the different levels of traffic.

5. A limited number of laboratory base mix designs were prepared for this study. It is recommended that the test results used as the basis for the recommended base design procedure by verified by additional testing of duplicate mixtures. While it is thought that novaculite aggregate will behave simillar to the syenite and gravel base mixtures, novaculite aggregate should be added to the base mixture test work.

6. The use of the aggregate effective specific gravity, as measured in the Rice's maximum specific gravity test (ASTM D 2041), should be used in the voids analysis of asphalt mixtures. The required air voids and voids in the mineral aggregate should be based on their relationship with the asphalt content as reported herein.

7. It has been concluded from prior research using simillar Arkansas asphalt mixtures (24) that air void contents greater than 5 percent are detrimental to good pavement durability. Based on the results of this work and the above, it is strongly recommended that inplace asphalt pavements have a residual air void content of from 2.5 to 5 percent for optimum pavement performance.

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APPENDIX

TABLE A1 LAYER RESILIENT MODULUS AND THICKNESS

Site No.	R-S	Layer 1 Inch	Mr psi	Layer 2 Inch	Mr psi	Layer 3 Inch	Mr psi	Layer 4 Inch	Mr psi
1	65-15	1.31	199	2.75	182	5.80	452	-	-
2	65-15	1.39	307	6.97	277	4.30	-	-	-
3	71-17	2.14	429	3.33	410	9.28	410	-	-
4	71-17	2.11	416	3.43	467	12.05	378	-	-
5	71-16B	1.53	247	2.07	448	4.73	330	-	-
6	71-16B	0.99	257	1.47	568	7.15	438	-	-
7	71-16	1.92	454	2.16	518	5.35	433	-	-
8	71-19	1.27	348	4.33	433	8.65	403	-	-
9	271-1	1.56	439	2.39	411	5.38	451	-	-
10	71-13	1.55	254	1.97	392	5.71	295	-	-
11	71-14	1.47	454	1.37	569	1.83	443	-	-
12	22-3	1.19	667	2.13	596	-	-	-	-
13	65-12A	1.49	289	1.92	334	8.63	416	-	-
14	65-12A	1.52	404	1.86	408	8.37	517	-	-

TABLE A1 LAYER RESILIENT MODULUS AND THICKNESS (Continued)

Site No.	R-S	Layer 1 Inch	Mr psi	Layer 2 Inch	Mr psi	Layer 3 Inch	Mr psi	Layer 4 Inch	Mr psi
15	167-12	1.61	407	2.14	346	7.02	440	-	-
16	70-9	1.51	439	2.27	486	-	-	-	-
17	30-22	1.99	151	2.08	350	5.22	-	-	-
18	30-12	2.11	621	-	-	-	-	-	-
19	71-1	1.46	487	3.66	661	2.29	558	-	-
20	71-2	1.42	300	1.85	446	5.90	564	-	-
21	82-2	1.78	334	1.08	252	2.44	178	5.58	4.34
22	7-2	1.64	368	1.91	434	4.98	461	-	-
23	167-2	1.97	437	1.56	416	1.42	628	4.79	625
24	167-3	1.51	166	3.02	204	7.62	422	-	-
25	81-6	2.03	137	1.13	265	2.07	381	2.72	-
26	65-20	1.67	386	2.69	375	-	-	-	-
27	65-18	1.84	253	1.78	311	2.75	338	3.53	532
28	82-11	1.16	547	1.89	812	-	-	-	-

TABLE A1 LAYER RESILIENT MODULUS AND THICKNESS (Concluded)

Site No.	R-S	Layer 1 Inch	Mr psi	Layer 2 Inch	Mr psi	Layer 3 Inch	Mr psi	Layer 4 Inch	Mr psi
29	55-11	2.82	532	5.52	445	1.58	-	-	-
30	70-8	2.05	492	1.03	362	1.33	84	1.14	-
31	1-8	1.46	595	1.47	400	2.12	533	-	-
32	1-9	1.89	328	1.87	800	-	-	-	-
33	49-9	1.26	498	2.03	673	-	-	-	-
34	64-13	2.01	275	1.78	417	2.50	328	4.87	402
35	64-14	2.07	451	1.62	678	2.02	334	5.08	173
36	49-3	1.19	577	2.03	453	3.99	628	-	-
37	79-6	1.37	384	2.02	416	7.78	488	-	-
38	79-6	2.15	203	1.47	255	2.50	307	7.47	578

Note: Mr = Resilient Modulus in 1000 psi

TABLE A2 EXTRACTED AGGREGATE GRADATION

Site No.	R-S	Lay. ID	Job No.	Type Agg.	Total Percent Passing Sieve Size Number											
					1.50	1.12	1.00	0.75	0.50	0.38	#4	#10	#20	#40	#80	#200
1	65-15	S	2798	NS	-	-	-	100	96	85	63	45	36	27	15.0	7.5
1	65-15	BI	2798	NS	-	-	100	90	72	65	42	31	26	21	10.5	3.0
2	65-15	S	2798	NS	-	-	-	100	94	85	62	47	38	29	15.0	7.2
2	65-15	BI	2798	NS	-	-	100	95	75	62	40	30	24	20	9.2	2.9
3	71-17	S	9563	LS	-	-	-	100	92	82	62	44	33	26	15.0	9.0
3	71-17	BI	9563	LS	-	-	100	97	73	56	40	29	25	21	12.0	6.0
3	71-17	Ba	9563	LS	100	97	96	78	56	47	33	23	17	13	9.5	6.2
4	71-17	S	4827	LS	-	-	-	100	97	90	57	42	34	28	14.5	6.0
4	71-17	BI	4827	LS	-	-	-	100	95	83	55	41	34	28	14.5	7.0
4	71-17	Ba	4827	LS	100	99	97	82	65	55	37	25	19	15	9.5	6.0
5	71-16B	S	4698	LS	-	-	-	100	91	77	56	41	32	27	15.0	9.5
5	71-16B	BI	4698	LS	-	-	100	90	64	53	42	31	25	21	14.5	7.5
6	71-16B	S	4698	LS	-	-	-	100	91	82	61	45	35	27	16.0	9.0
6	71-16B	BI	4698	LS	-	-	100	88	63	52	38	29	23	19	12.0	7.0
7	71-16	S	9432	LS	-	-	-	100	94	86	65	46	35	29	17.0	10.0

TABLE A2 EXTRACTED AGGREGATE GRADATION (Continued)

Site No.	R-S	Lay. ID	Job No.	Type Agg.	Total Percent Passing Sieve Size Number											
					1.50	1.12	1.00	0.75	0.50	0.38	#4	#10	#20	#40	#80	#200
7	71-16	Bl	9432	LS	-	-	100	97	76	66	48	35	27	21	13.0	8.0
7	71-16	Ba	9432	LS	-	-	100	90	70	58	42	31	24	20	16.0	10.7
8	71-19	S	9579	LS	-	-	-	100	94	86	66	49	38	30	13.8	7.5
8	71-19	Bl	9579	LS	-	-	100	91	77	68	52	37	28	23	14.5	9.0
8	71-19	Ba	9579	LS	100	98	96	84	72	62	45	32	23	20	16.0	12.0
9	271-1	S	4491	SS	-	-	-	-	100	96	70	45	35	27	17.0	11.0
9	271-1	Bl	4491	SS	-	-	100	97	71	59	41	30	25	21	14.5	7.0
9	271-1	Ba	4491	SS	100	93	90	77	60	52	49	30	25	23	18.0	8.0
10	71-13	S	4547	SS	-	-	100	98	93	83	58	40	31	25	17.0	9.0
10	71-13	Bl	4547	SS	-	-	100	93	68	55	37	27	23	21	17.0	7.5
10	71-13	Ba	4547	SS	100	96	94	77	62	53	37	26	21	19	18.0	8.0
11	71-14	S	4-687	SS	-	-	-	100	97	86	61	45	33	25	17.0	9.5
11	71-14	Bl	4-687	SS	-	-	100	92	73	61	41	29	23	21	15.0	7.0
12	22-3	S	4473	SS	-	-	-	100	99	92	62	46	37	29	15.0	10.8
12	22-3	Bl	4473	SS	-	-	100	92	70	59	40	27	21	17	13.0	7.0

TABLE A2 EXTRACTED AGGREGATE GRADATION (Continued)

Site No.	R-S	Lay. ID	Job No.	Type Agg.	Total Percent Passing Sieve Size Number											
					1.50	1.12	1.00	0.75	0.50	0.38	#4	#10	#20	#40	#80	#200
13	65-12A	S	6779	NS	-	-	-	-	100	97	76	52	36	27	15.0	10.3
13	65-12A	Bl	6779	NS	-	100	99	91	68	55	41	30	23	19	9.0	5.0
13	65-12A	Ba	6779	NS	-	100	90	69	52	47	42	37	33	28	11.0	4.6
14	65-12A	S	6779	NS	-	-	-	-	100	97	77	53	35	25	14.0	8.1
14	65-12A	Bl	6779	NS	-	-	100	91	69	59	44	32	25	21	9.0	4.7
14	65-12A	Ba	6779	NS	100	92	87	67	53	48	44	38	33	28	11.0	4.4
15	167-12	S	6782	NS	-	-	-	-	100	98	81	59	42	31	12.2	8.0
15	167-12	Bl	6782	NS	-	100	99	89	63	52	35	27	20	16	8.0	4.0
15	167-12	Ba	6782	NS	100	96	90	76	59	53	46	39	32	27	9.0	4.0
17	30-22	S	6997	NS	-	-	-	100	93	82	57	38	26	19	12.0	7.9
17	30-22	Bl	6997	NS	-	-	100	94	76	66	44	29	23	19	12.3	8.9
19	71-1	S	3655	GVL	-	-	-	100	90	82	66	42	32	23	12.0	6.5
19	71-1	Bl	3655	GVL	-	-	100	92	77	67	51	34	26	19	9.5	5.0
20	71-2	S	3623	GVL	-	-	-	100	89	75	55	40	31	24	14.0	5.5
20	71-2	Bl	3623	GVL	-	-	100	93	76	65	52	33	27	23	15.0	4.9

TABLE A2 EXTRACTED AGGREGATE GRADATION (Continued)

Site No.	R-S	Lay. ID	Job No.	Type Agg.	Total Percent Passing Sieve Size Number											
					1.50	1.12	1.00	0.75	0.50	0.38	#4	#10	#20	#40	#80	#200
20	71-2	Ba	3623	GVL	-	-	100	91	75	65	48	39	36	27	14.0	7.0
21	82-2	S	7819	NOV	-	-	-	100	93	83	64	49	40	31	17.0	9.0
21	82-2	S2	3-633	GVL	-	-	-	100	91	77	57	45	37	26	11.0	6.1
21	82-2	S3	3-633	GVL	-	-	-	100	94	82	60	41	34	27	15.5	9.0
21	82-2	BI	3-633	GVL	-	-	100	95	87	78	60	46	36	26	12.0	8.7
22	7-2	S	7710	GVL	-	-	-	100	93	77	54	40	30	23	10.0	5.5
22	7-2	BI	7710	GVL	-	-	100	98	73	63	45	33	27	24	10.0	4.9
22	7-2	Ba	7710	GVL	-	-	100	99	85	71	52	39	34	31	20.0	6.8
23	167-2	S	7845	GVL	-	-	-	100	95	91	65	44	35	27	12.0	7.6
23	167-2	S2	7557	GVL	-	-	-	100	96	87	59	42	35	29	12.5	7.8
23	167-2	Ba	7557	GVL	100	97	96	85	71	59	42	34	31	27	8.0	3.1
24	167-3	S	7716	NS	-	-	-	100	94	82	63	48	39	33	16.0	8.9
24	167-3	BI	7716	NS	-	-	100	96	83	67	45	33	28	25	10.0	4.0
24	167-3	Ba	7716	GVL	-	100	99	98	82	69	51	41	37	34	10.7	4.3
25	81-6	S	2-663	NS	-	-	-	100	93	85	65	51	41	31	16.0	8.6

TABLE A2 EXTRACTED AGGREGATE GRADATION (Continued)

Site No.	R-S	Lay. ID	Job No.	Type Agg.	Total Percent Passing Sieve Size Number											
					1.50	1.12	1.00	0.75	0.50	0.38	#4	#10	#20	#40	#80	#200
25	81-6	S2	2-536	NS	-	-	-	100	98	94	71	52	41	31	14.2	7.2
26	65-20	S	2659	NS	-	-	-	-	100	97	75	39	33	27	18.0	10.2
26	65-20	BI	2659	NS	-	-	100	93	72	59	41	30	23	19	13.0	5.4
27	65-18	S	2824	NS	-	-	-	100	88	77	56	41	32	24	14.0	6.5
27	65-18	BI	2824	NS	-	-	100	97	82	64	45	35	31	25	14.5	5.0
28	82-11	S	2534	NS	-	-	-	-	100	97	75	49	32	22	11.0	7.7
28	82-11	BI	2534	NS	-	-	100	92	65	53	37	30	26	18	7.0	4.0
29	55-11	S	11829	LS	-	-	-	100	93	88	66	46	34	27	16.0	11.2
29	55-11	BI	11829	LS	-	-	100	97	87	79	48	34	24	15	10.0	7.9
30	70-8	S	6-677	SS	-	-	-	100	97	86	59	41	33	30	22.0	12.1
30	70-8	S2	6-602	NS	-	-	-	-	100	98	71	45	30	21	13.0	9.0
31	1-8	S	11955	LS	-	-	-	-	100	97	64	43	32	21	14.0	9.5
31	1-8	S2	11-582	LS	-	-	-	-	100	97	70	48	36	28	18.0	10.3
32	1-9	S	11-730	GVL	-	-	-	100	93	84	67	52	41	26	14.0	7.1
32	1-9	S2	11489	LS	-	-	-	100	97	86	66	48	38	27	14.0	8.6

TABLE A2 EXTRACTED AGGREGATE GRADATION (Continued)

Site No.	R-S	Lay. ID	Job No.	Type Agg.	Total Percent Passing Sieve Size Number											
					1.50	1.12	1.00	0.75	0.50	0.38	#4	#10	#20	#40	#80	#200
33	49-9	S	11519	LS	-	-	-	-	100	97	67	46	34	26	14.0	8.2
33	49-9	BI	11519	SS	-	-	100	90	68	55	39	31	25	20	9.0	4.1
34	64-13	S	11956	SS	-	-	-	100	95	82	61	43	34	22	10.0	5.4
34	64-13	S2	11707	SS	-	-	-	-	100	95	67	46	37	32	22.0	10.9
34	64-13	BI	11707	SS	-	-	100	98	74	61	44	31	25	22	16.0	6.2
34	64-13	Ba	11707	SS	100	99	98	84	65	57	41	31	25	21	18.0	8.1
35	64-14	S	11956	SS	-	-	-	100	94	83	62	46	38	25	13.0	7.8
35	64-14	S2	11682	SS	-	-	-	-	100	97	73	47	35	31	20.0	9.0
35	64-14	BI	11682	SS	-	-	100	88	70	53	43	31	25	22	16.0	5.0
35	64-14	Ba	11682	SS	100	98	95	85	73	65	40	27	22	20	17.0	8.0
36	49-3	S	10677	LS	-	-	-	-	100	97	70	48	36	31	16.0	12.4
36	49-3	BI	10677	LS	-	-	100	97	77	64	44	32	26	20	7.0	3.5
36	49-3	Ba	10677	GVL	100	98	97	83	62	52	37	31	27	24	11.0	4.9
37	74-6	S	7753	NOV	-	-	-	100	89	79	59	43	35	30	13.0	6.9
37	79-6	BI	7753	NOV	-	-	100	94	76	62	47	35	27	24	12.0	3.8

TABLE A2 EXTRACTED AGGREGATE GRADATION (Concluded)

Site No.	R-S	Lay. ID	Job No.	Type Agg.	Total Percent Passing Sieve Size Number											
					1.50	1.12	1.00	0.75	0.50	0.38	#4	#10	#20	#40	#80	#200
37	79-6	Ba	7753	GVL	100	99	95	87	69	58	41	31	28	25	12.0	6.2
38	79-6	S	7753	NOV	-	-	-	100	95	87	64	42	30	25	18.0	10.0
38	79-6	S2	7615	NOV	-	-	-	-	100	97	73	47	33	25	14.0	8.4
38	79-6	BI	7615	NOV	-	-	100	97	75	59	46	33	22	17	8.0	4.6
38	79-6	Ba	7615	GVL	-	100	98	93	74	59	37	27	22	18	9.0	6.0
Average of - 0.5 In. Size									100	97	72	44	35	27	14.2	9.5
Average of - 0.75 In. Size								100	94	84	61	44	35	27	14.0	8.1
Average of - 1.0 In. Size							100	94	75	63	45	33	25	22	12.6	6.1
Average of - 1.12 In. Size						100	97	88	68	56	41	32	27	23	9.5	4.8
Average of - 1.5 In. Size					100	97	94	80	70	50	41	31	22	20	13.1	6.1

TABLE B1 LABORATORY MIXTURE TEST RESULTS

MR 1000 psi	EM 1000 psi	STAB psi	AC %	FLOW .01 in.	AV %	VMA %	VF %	ID # Mix	AGG ID Type
514.0	12.45	319.0	4.5	10.3	4.07	14.5	72.1	1.	A-ls
447.0	10.09	261.0	5.0	10.3	3.54	15.1	76.7	2.	A-ls
420.9	7.32	264.0	5.5	14.5	1.97	14.8	86.9	3.	A-ls
233.3	5.80	166.6	4.5	11.5	3.20	14.1	77.2	4.	A-gvl
176.3	3.97	131.2	5.2	13.3	1.60	13.7	88.5	5.	A-gvl
142.8	2.72	117.8	5.7	17.3	1.20	14.5	91.7	6.	A-gvl
208.8	8.44	225.6	4.7	10.7	4.67	15.4	69.6	7.	A-ss
220.1	8.31	242.2	5.2	11.7	2.3	14.3	84.0	8.	A-ss
211.1	6.24	216.3	5.7	13.9	1.27	14.5	91.2	9.	A-ss
195.9	7.77	210.2	4.7	10.8	7.07	17.5	59.6	10.	A-ns
193.8	7.98	242.2	5.2	12.2	5.07	16.8	69.7	11.	A-ns
168.2	6.24	186.9	5.7	12.0	4.92	17.7	72.3	12.	A-ns
205.3	6.86	211.7	6.0	12.3	2.35	16.9	80.3	13.	A-ns
240.7	7.32	150.9	4.6	8.3	3.52	14.0	75.0	14.	A-gvl

TABLE B1 LABORATORY MIXTURE TEST RESULTS (Continued)

MR 1000 psi	EM 1000 psi	STAB psi	AC %	FLOW .01 in.	AV %	VMA %	VF %	ID # Mix	AGG ID Type
233.7	5.20	147.6	5.1	11.3	2.14	13.9	84.6	15.	A-gvl
205.3	4.22	134.5	5.6	12.8	1.50	14.4	89.5	16.	A-gvl
297.0	10.69	219.5	4.6	8.3	6.11	16.4	62.7	17.	A-ss
267.5	9.56	225.3	5.1	9.5	3.87	15.5	75.0	18.	A-ss
325.7	9.07	279.5	5.6	12.3	2.13	15.0	85.9	19.	A-ss
215.2	6.53	212.4	6.1	13.0	1.54	15.5	90.1	20.	A-ss
224.0	7.65	212.0	5.1	11.2	6.24	17.5	64.3	21.	A-ns
268.9	9.35	217.8	5.6	9.5	3.45	16.1	78.56	22.	A-ns
249.9	7.01	230.3	6.1	13.5	1.95	15.8	87.79	23.	A-ns
324.0	11.10	256.2	4.5	9.0	3.74	14.3	73.8	24.	A-ls
339.3	8.82	255.4	5.0	11.8	2.74	14.5	81.1	25.	A-ls
239.5	6.96	234.6	5.5	13.5	1.46	14.4	89.9	26.	A-ls
346.1	7.29	211.5	4.0	11.7	3.68	13.1	71.9	27.	B-ls
380.3	6.06	207.8	4.5	13.8	2.64	13.1	80.9	28.	B-ls

TABLE B1 LABORATORY MIXTURE TEST RESULTS (Continued)

MR 1000 psi	EM 1000 psi	STAB psi	AC %	FLOW .01 in.	AV %	VMA %	VF %	ID # Mix	AGG ID Type
285.7	4.05	206.5	5.0	20.7	2.06	13.8	85.2	29.	B-ls
217.7	5.94	139.4	4.0	9.3	6.48	15.4	58.2	30.	B-ns
173.7	4.26	86.6	4.5	8.3	6.13	16.1	62.1	31.	B-ns
208.0	3.21	75.2	5.0	9.3	6.05	17.1	64.7	32.	B-ns
273.3	6.03	129.8	4.0	8.8	5.57	14.5	61.7	33.	B-gvl
296.3	6.00	169.9	4.5	11.3	4.81	14.3	66.4	34.	B-gvl
316.9	4.87	149.8	5.0	12.3	2.72	14.1	80.7	35.	B-gvl
227.0	7.90	180.8	4.8	9.2	4.88	15.7	68.9	36.	A-ns
198.0	6.17	162.1	5.5	10.5	3.16	15.6	80.0	37.	A-ns
159.0	4.51	145.9	6.2	13.0	1.69	15.8	89.3	38.	A-ns
215.0	6.78	157.9	4.8	9.3	4.32	15.1	71.4	39.	A-gvl
174.3	5.23	150.5	5.5	11.5	2.28	14.8	84.7	40.	A-gvl
140.3	3.33	124.2	6.2	15.2	0.93	15.1	93.9	41.	A-gvl
308.0	10.06	269.8	4.8	10.7	6.30	17.0	63.0	42.	A-ss

TABLE B1 LABORATORY MIXTURE TEST RESULTS (Continued)

MR 1000 psi	EM 1000 psi	STAB psi	AC %	FLOW .01 in.	AV %	VMA %	VF %	ID # Mix	AGG ID Type
265.0	8.22	243.4	5.5	11.8	4.45	16.8	73.5	43.	A-ss
197.0	6.06	234.3	6.2	15.5	2.25	16.3	86.3	44.	A-ss
208.8	9.60	211.2	4.5	9.0	4.14	14.6	71.7	45.	A-ls
194.8	9.17	258.1	5.0	11.3	1.86	13.7	86.5	46.	A-ls
154.6	6.85	239.3	5.5	14.0	0.90	14.0	93.6	47.	A-ls
93.3	4.46	195.1	6.0	17.5	0.50	14.7	96.6	48.	A-ls
353.0	7.05	264.5	4.8	15.0	1.21	12.6	90.4	49.	A-ls
314.0	5.67	231.2	5.3	16.3	0.92	13.4	93.2	50.	A-ls
243.0	3.66	200.1	5.8	22.0	0.39	14.1	97.2	51.	A-ls
328.5	10.08	258.5	4.5	10.3	3.20	13.7	76.8	52.	A-ls
441.0	7.22	269.1	4.9	15.0	2.15	13.6	84.3	53.	A-ls
314.0	7.38	261.3	5.3	14.2	1.39	13.9	90.0	54.	A-ls
313.7	6.24	246.8	5.7	15.8	0.55	14.0	95.9	55.	A-ls
334.0	7.18	269.2	4.8	15.0	1.08	12.5	91.4	56.	A-ls

TABLE B1 LABORATORY MIXTURE TEST RESULTS (Continued)

MR 1000 psi	EM 1000 psi	STAB psi	AC %	FLOW .01in.	AV %	VMA %	VF %	ID # Mix	AGG ID Type
355.0	5.35	235.3	5.2	17.7	0.76	13.1	94.2	57.	A-ls
227.3	3.79	241.6	5.6	25.5	0.39	13.7	97.2	58.	A-ls
216.3	2.60	182.9	6.0	28.3	0.34	14.5	97.6	59.	A-ls
485.0	7.11	240.0	3.3	13.5	4.90	12.7	61.3	60.	C-ls
390.0	7.13	214.0	3.8	12.0	3.20	12.3	73.7	61.	C-ls
308.0	5.32	222.0	4.3	16.7	1.30	11.6	89.0	62.	C-ls
448.0	4.59	195.0	3.3	17.0	2.80	12.9	78.7	63.	C-ls
476.0	7.23	208.0	3.8	11.5	3.40	12.4	72.7	64.	C-ls
291.0	7.58	214.0	4.3	11.3	5.60	13.4	52.7	65.	C-ls
189.0	6.40	112.0	3.0	7.0	11.70	18.2	35.5	66.	C-ns
209.0	5.84	92.0	3.5	6.3	11.20	18.7	40.2	67.	C-ns
238.0	6.80	114.0	4.0	6.7	9.30	18.0	48.3	68.	C-ns
377.0	8.63	220.0	4.0	10.2	6.50	15.1	59.6	69.	C-ns
313.0	8.14	179.0	4.5	8.8	6.10	16.5	60.6	70.	C-ns

TABLE B1 LABORATORY MIXTURE TEST RESULTS (Concluded)

MR 1000 psi	EM 1000 psi	STAB psi	AC %	FLOW .01 in.	AV %	VMA %	VF %	ID # Mix	AGG ID Type
275.0	4.55	91.0	3.3	8.0	10.90	18.0	39.4	71.	C-ss
313.0	6.00	168.0	3.8	11.2	8.50	16.8	49.6	72.	C-ss
208.0	4.76	137.0	3.8	11.5	9.70	17.9	45.9	73.	C-ss
253.0	5.14	131.0	4.3	10.2	7.50	17.0	55.6	74.	C-ss
349.0	5.65	106.0	3.3	7.5	7.10	14.5	50.9	75.	C-gvl
312.0	6.50	130.0	3.8	8.0	5.30	13.9	62.0	76.	C-gvl
308.0	4.78	104.0	3.8	8.7	5.40	14.0	72.5	77.	C-gvl
253.0	5.08	136.0	4.3	10.7	3.70	13.5	61.4	78.	C-gvl

Binder Mixes = ID # 28-35. Base Mixes = ID # 60-78.
Surface Mixes = All Other ID #.
All Mixes Compacted At 50 Blows except ID # 1-3, 24-26 and 45-48,
Which Were Compacted At 75 Blows.