TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No. FHWATX78-41-2F	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle ECONOMIC BITUMINOUS TREATED BASES		5. Report Date September 1977
		6. Performing Organization Code
7. Author(s) Wayne A. Schoen, Jon A. Epps, Bob M. Gallaway and Dallas N. Little		8. Performing Organization Report No. Research Report 41-2F
9. Performing Organization Name and Address Texas Transportation Institute, Texas A&M University, College Station, Texas 77843		10. Work Unit No.
		11. Contract or Grant No. Research Study 2-6-74-41
12. Sponsoring Agency Name and Address Texas State Department of Highways and Public Transportation; Transportation Planning Division, P. O. Box 5051, Austin, Texas 78763		13. Type of Report and Period Covered Final - September 1973 - September 1977
		14. Sponsoring Agency Code

15. Supplementary Notes

Research performed in cooperation with DOT, FHWA
Research Study Title: "Bituminous Treated Bases"

16. Abstract

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Cores from several pavements containing asphalt treated materials were obtained and compared with the results from the laboratory study on marginal aggregates. In general the laboratory results are within the range of properties obtained from the field cores.

An economic analysis method shows that mixture design and pavement design considerations cannot be separated, if an economic solution is to be provided.

17. Key Words Bituminous Stabilization, Black Base, Pavement Design, Marginal Materials, Economics	18. Distribution Statement No Restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22151.

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	231	

Form DOT F 1700.7 (8-69)

ECONOMIC BITUMINOUS TREATED BASES

by

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and Dallas N. Little

Research Report 41-2F
Bituminous Treated Bases
Research Study 2-6-74-41

Sponsored by:
The Texas State Department of Highways
and Public Transportation
In Cooperation With The
U. S. Department of Transportation
Federal Highway Administration
September 1977

Texas Transportation Institute
Texas A&M University
College Station, Texas 77843

PREFACE

This report is issued under Research Study 2-6-74-41, "Bituminous Treated Bases," and presents laboratory test results obtained on both laboratory compacted samples and field cores. Additionally, a method for economic analysis is presented which considers both material properties and pavement design considerations. This is the second and final report of this study.

DISCLAIMER

ACKNOWLEDGEMENTS

The authors wish to express their appreciation to Texas State Department of Highways and Public Transportation personnel in Districts 5, 11, 13, 15, 17, 18, 20 and 25 as well as representatives from divisions D-6, D-8, D-9, D-11 and D-18 for their time and efforts expended on obtaining the necessary field samples and cost information.

ABSTRACT

Research was conducted to determine the technical and economic suitability of using low-quality, lower cost, local aggregates in asphalt treated base courses in Texas.

Five marginal aggregates were investigated together with three sands and a sand gravel mixture. Results indicate that several of the marginal aggregates can be utilized as bituminous stabilized base courses provided that strict quality and construction control measures are employed. Additionally, these materials must be used under traffic and environmental conditions which are compatible with the stabilized mixture properties.

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An economic analysis method shows that mixture design and pavement design considerations cannot be separated, if an economic solution is to be provided.

KEY WORDS

Bituminous Stabilization, Black Base, Pavement Design, Marginal Materials, Economics

SUMMARY

Although black base construction has gained increasing popularity in recent years, the rising costs of asphalt and asphalt materials have demanded that more research be conducted to evaluate the economic feasibility of using marginal materials for use as black bases. The purpose of this study was to determine the technical and economic suitability of using low-quality, lower cost, local aggregates in asphalt treated base courses in Texas.

Five marginal aggregates from District 15 and 18 were investigated together with three sands and a sand gravel mixture. Extensive laboratory tests were performed on these materials. Results indicate that several of the marginal aggregates can be utilized as bituminous stabilized base courses provided that strict quality and construction control measures are employed. Additionally, these materials must be used under traffic and environmental conditions which are compatible with the stabilized mixture properties.

Cores from several pavements containing asphalt treated materials were obtained and compared with the results from the laboratory study on marginal aggregates. In general the laboratory results are within the range of properties obtained from the field cores.

An economic analysis method has been presented which allows the engineer to consider a number of factors including mixture properties, the effect of asphalt content and the cost of the asphalt and aggregate. From this analysis it is apparent that mixture design and pavement design considerations can not be separated, if an economic solution is to be provided.

The use of a "sandwich" design for the use of marginal materials has been suggested. This concept places the marginal or lower quality material between two layers of a higher quality material and thus lower tensile stresses and shear stresses are imposed on the marginal material than if it were placed as a surface or at the bottom of the asphalt stabilized section.

IMPLEMENTATION STATEMENT

Several materials have been recognized in this study as being suitable for use as bituminous stabilized bases. Concerned districts are encouraged to use these materials as well as the mix design concepts utilized in this study. Districts from which these marginal materials were obtained will be contacted as part of Research Study 2-9-74-214 for possible implementation of these results.

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INTRODUCTION

General

Since the end of World War II, the United States has experienced a continuous increase in traffic in terms of the number of vehicles, the magnitude of wheel loads, and the percentage of heavy vehicles on the roadway. The continuous increase in traffic is demanding more roadways with stronger structural sections to support the heavier loads. In short, there appears to be an increasing demand for highway construction and related construction materials. Due to the increasing costs of a diminishing supply of high-quality aggregates, it has become necessary to investigate the treating of low-quality, local materials for use as base courses.

Aggregates comprise a major portion of the material required in highway construction. Bituminous base courses, for example, generally contain 90 to 95 percent aggregate. The current aggregate consumption in the highway construction field is about one billion tons annually (1). This aggregate consumption is expected to increase at an annual rate of approximately five percent during this decade (2). The demand for high quality aggregates has stripped the sources of supply in many parts of the country. Figure 1 indicates the areas of the United States which lack quality aggregates (2) and Table 1 illustrates the projected supply and demand for the various AASHTO regions for the years 1975 and 1985 (3).

Possible alternatives to supply these regions with acceptable aggregates are being investigated. Among these alternatives are:

1. improved utilization of locally available, low quality aggregates,

2. greater acceptance of manufactured aggregates,

3. development of new materials and construction methods that may prove to be more economical and more efficient than conventional means, and

4. improved handling and transportation of those aggregates which are remote to the construction site.

Although aggregate consumption in the area of highway construction is expected to continue to increase, the production of aggregates from new sources and existing sources has been stifled in many ways. For example, the production of aggregates is being hampered by changes in land use, increases in aggregate production costs, changes in pollution control laws, and a strong reluctance to accept changes in specifications and construction procedures required for new materials.

The reluctance to accept changes in specifications and construction procedures is a major problem facing the highway design engineer today. The utilization of marginal aggregates as a base material in highway construction must include the revision of specifications. In many cases, specifications for aggregates are written to be applied nationwide and may not be suited for all situations in all areas. The specifications followed by many local agencies are, for all practical purposes, duplicates of these national specifications and do not properly reflect local constraints. The altering of certain specifications could allow the use of lesser quality aggregates in those layers which are subject to less stress and/or different environmental conditions. Associated with the acceptance of these marginal aggregates is the need for a materials characterization scheme. Many of the tests used to classify aggregate quality and/or serviceability are open to serious question (4). In many cases pavement structures have been built from materials which meet the required specifications but failed to perform as expected. This may tend to indicate a weakness in construction control and/or materials specifications. This is not to say that the blame rests entirely with construction control and specifications but rather that agencies, engineers and contractors must view these problems with an open mind and be willing to make necessary changes demanded by each situation.

The primary purpose of a base course is to reduce the unit pressure caused by wheel loads on the subgrade. A base course must be of sufficient strength and rigidity so as to sustain the high unit pressure without excessive consolidation, distortion, or lateral flow. As the surface course becomes thinner, the base course must be stronger and more durable.

As a means of better utilizing marginal aggregates in highway construction it has, at times, become necessary to treat these materials with some type of chemical and/or mechanical stabilization. In the last eight years, asphalt has become increasingly popular as a base stabilizer (5). In Texas, these asphalt stabilized base courses are generally known as "black bases." At present, an acceptable national standard procedure for the design and construction of such black bases does not exist. In fact, many black bases have been designed using requirements developed for asphalt concrete surface courses. This practice leads to a base course which is often significantly more expensive and structurally over-designed for the required traffic and environment. Through laboratory and field testing procedures, it may be possible to utilize lower-quality aggregates which can provide suitable base course characteristics at lower costs.

Background

Marginal materials have been used in highway construction for many years and are generally performing quite well. In many cases, this has become possible through the use of bituminous stabilization. Although documentation of its use is fairly recent, the utilization of asphalt as a construction material dates back to ancient times (6). Local and state agencies in the United States have used asphalt stabilized base materials and full-depth asphalt pavements since the late 1800's (7, 8, 9, 10) and by 1904 there were over 6,000 miles of bituminous surfaced roads in the United States (11). Today there are over 3.8 million miles of improved roads of which 50 percent, or approximately 1.8 million miles, are improved by some type of bituminous treatment (12).

The ability of asphalt to stabilize sub-standard materials and provide desirable strength characteristics in terms of stability, durability and tensile and fatigue behavior has long been recognized. The favorable performance of the asphalt stabilized base courses at the AASHO Road Test (13) provided further insight into the field of bituminous stabilization.

In the last decade, new dimensions in research and application have evolved in the field of bituminous stabilization. As early as 1960, researchers began to investigate the characteristics of asphalt treated mixtures using lower-quality, locally available, marginal aggregates. In a National Cooperative Highway Research Program report (14) the authors suggest several alternative solutions to the problem of a diminishing supply of high-quality aggregates. Among the solutions cited is the better utilization of existing and available conventional aggregates through selective use and/or beneficiation. The authors suggest:

1. the revision of specifications to permit the use of aggregates not now meeting current requirements in locations where their performance would be adequate,

2. the use of additives and blending to improve many of the engineering properties of marginal aggregates, and

3. benefication of low quality material by removal of deleterious fractions by washing, impregnation of plastics or cements, coating the aggregate, etc.

The problems related to a diminishing supply of high-quality, conventional aggregates are not unique to the United States. Great Britain, for example, has realized the need to make use of lower-grade materials both from the standpoint of conserving the supplies of high-quality aggregates and also assist in problems arising from the disposal of excess unwanted materials. The Transport and Road Research Laboratory in Crowthorne, Berkshire conducted a study on the use of low-grade and waste materials in road construction (15). The readily available low-grade aggregate materials which can be used for road construction in Great Britain consist of:

1. wastes which arise from quarrying china clay in quantities such that the quarrying of one ton of china clay gives rise to nearly 9 tons of waste,

2. wastes which arise from the quarrying of slate in which the ratio of waste to slate averages 20:1,

3. hassock, a soft calcareous sand or argillaceous sandstone which is a waste product generally found in the quarrying of a hard, sandy limestone locally known as "ragstone," and,

4. chalk, although it is not a waste product, it represents 15 percent of the major geological formations of England and is frequently encountered in roadworks.

The aggregates which are used in road construction are generally stabilized with cement and termed "cementbound granular material." Due to its inaccessibility, asphalt is rarely used as a stabilizing material of the base or subbase in British highway construction.

In areas where natural aggregate deposits contain insufficient quantities of aggregate larger than the number 4 size, Gregg et al. (16), Hartronft (17) and Warden and Hudson (18) have investigated the feasibility of using sand-asphalt stabilized base courses. Hartronft concluded that sand-asphalt mixtures have indicated excellent performance on medium and low traffic roads. Warden and Hudson concluded that as long as the design and construction of sand-asphalt mixtures is carefully controlled, a suitable base material may be produced.

Extensive research has been conducted in the field of bituminous stabilization and results of many of these research efforts are presented throughout this paper.

Scope of the Investigation

The purpose of the study reported herein is to determine the economical feasibility and technical suitability of using low-quality, local aggregates in asphalt treated base courses in Texas. In order to satisfy the purpose of the study a laboratory program was undertaken. Tests were performed on laboratory prepared samples as well as core samples obtained from pavements. Results of this testing program are included below together with an economic apprasal of the suitability of several of the materials tested. Selected field performance information for black base pavements in Texas is included in the economic analysis (19).

Materials and Testing Procedures

Aggregates

Aggregates which require special treatment and/or processing to meet specification requirements are often referred to as marginal materials. The first step in determining the suitability of such aggregates for black base construction in Texas was to obtain a group of aggregates which were considered to be both marginal and local to many areas of Texas and would be utilized on future highway projects. Several aggregates were selected for study as they appeared to satisfy the above mentioned criteria. Four aggregates were supplied by the Texas State Department of Highways and Public Transportation personnel in San Antonio, Texas (District 15). These aggregates include:

1. sandstone, abbreviated SS, from the Garner-Ross pits in Webb County, fifteen miles west of Encinal on US 83,
2. crushed limestone, abbreviated LS, from the McDonough Brothers San Pedro pit in Bexar County, near San Antonio,
3. crushed caliche gravel, abbreviated CCG, from the Mack pit in Frio County, on US 57, and
4. crushed sandstone, abbreviated CSS, from the "74" Ranch pit in Atascosa County, two miles south of Campbellton on US 281.

A fifth aggregate type selected for study is termed "Austin chalk" obtained from a "cut" section on US 67 south of Dallas, Texas. For convenience, the Austin chalk was abbreviated "DAC."

As indicated above the Texas State Department of Highways and Public Transportation personnel supplied the five aggregates to be used in the laboratory black base program. Along with the aggregates, highway personnel also included a recommended aggregate gradation as shown in Figure 2 together with the aggregates separated into individual sieve size fractions.

The aggregates were recombined to the desired gradation and a washed

Table 2. Physical Properties of Sand and Sand Gravel Aggregates

Sizes	Lamb County 5-FM168 S. of Olton	Wheeler County 25-FM182 S. end of Sweetwater Creek	Jasper County 20-US96 Stockpile (Plant Site)	Hidalgo County 21-6Mc W. Mission Beck Pit (sand-gravel)
1"				
3/4"				16.8
1/2"				28.0
3/8"			0.6	35.0
#4			1.4	50.3
#8			2.5	61.3
#10		0.06	2.7	63.3
#16	0.01	0.3	5.5	67.8
#30	0.1	3.4	17.8	71.9
#40	0.3	11.6	33.6	73.6
#50	2.8	44.3	58.4	76.2
#60	26.0	59.4	71.5	79.2
#80	73.3	67.9	81.8	84.7
#100	86.8	76.8	83.8	87.0
#200	97.2	96.1	86.2	91.8
Sand Equivalent	41.0	41.3	18.0	46.5
Fineness Modulous	0.897	1.25	1.69	4.14
Plastic Index	0	0	7.8	0
Liquid Limit	21.0	20.3	22.8	24.5
Plastic Limit	NP	NP	15.0	NP

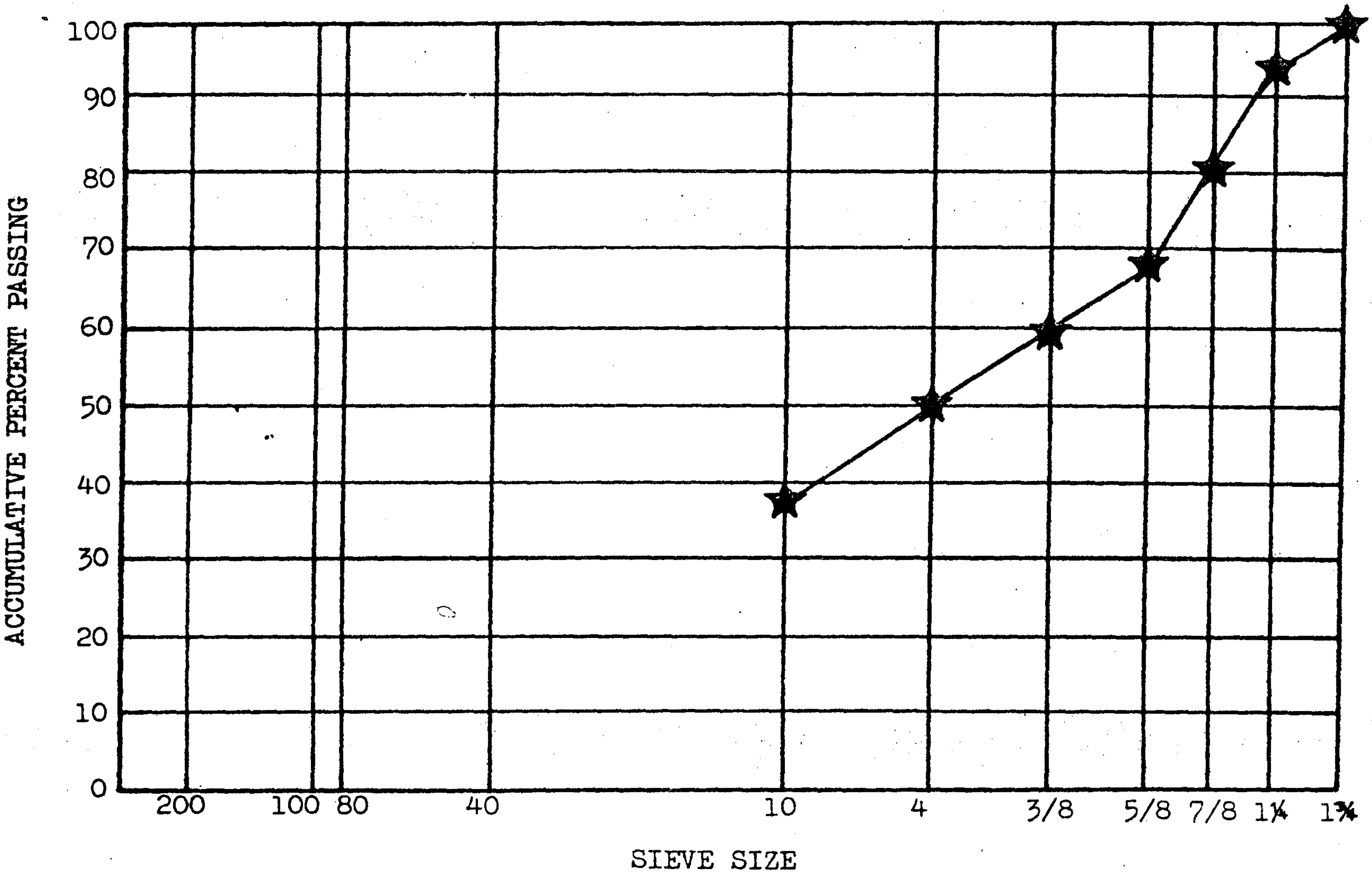

Figure 2. Recommended aggregate gradation for laboratory molded black base mixtures.

sieve analysis was conducted according to American Society of Testing and Materials Designation C 117-69 (20). The purpose of the test was to determine the amount of fine materials adhering to the coarse aggregate particles and also to determine the gradation of the minus No. 10 material.

Figures 3 through 7 demonstrate the results of the washed sieve analysis. The crushed limestone (Figure 4) included sizable prportions of minus No. 40 and minus No. 200 material. The crushed sandstone aggregate (Figure 6) indicated moderate amounts of fine particles on the coarse aggregates. The crushed sandstone included a large amount of minus No. 40 material. The Dallas Austin chalk (Figure 7) indicated excessive amounts of fine material, particularly the minus No. 200 material (38 percent). A large portion of the chalk actually disintergrated during the washing.

The aggregate gradation of the plus No. 10 material utilized for these aggregates are typical of the black bases used in Texas (21). The difference between these so called "marginal" aggregate and other black base aggregates is the amount passing the No. 10 and/or No. 200 sieves. Since most aggregate producing plants have an abundance of these fines, it would be of benefit from a materials conservation standpoint and perhaps an economic standpoint to make use of this "waste" material.

The combined bulk specific of the aggregates were as follows:

Aggregate	Combined Bulk Specific Combined
Garner-Ross sandstone	2.64

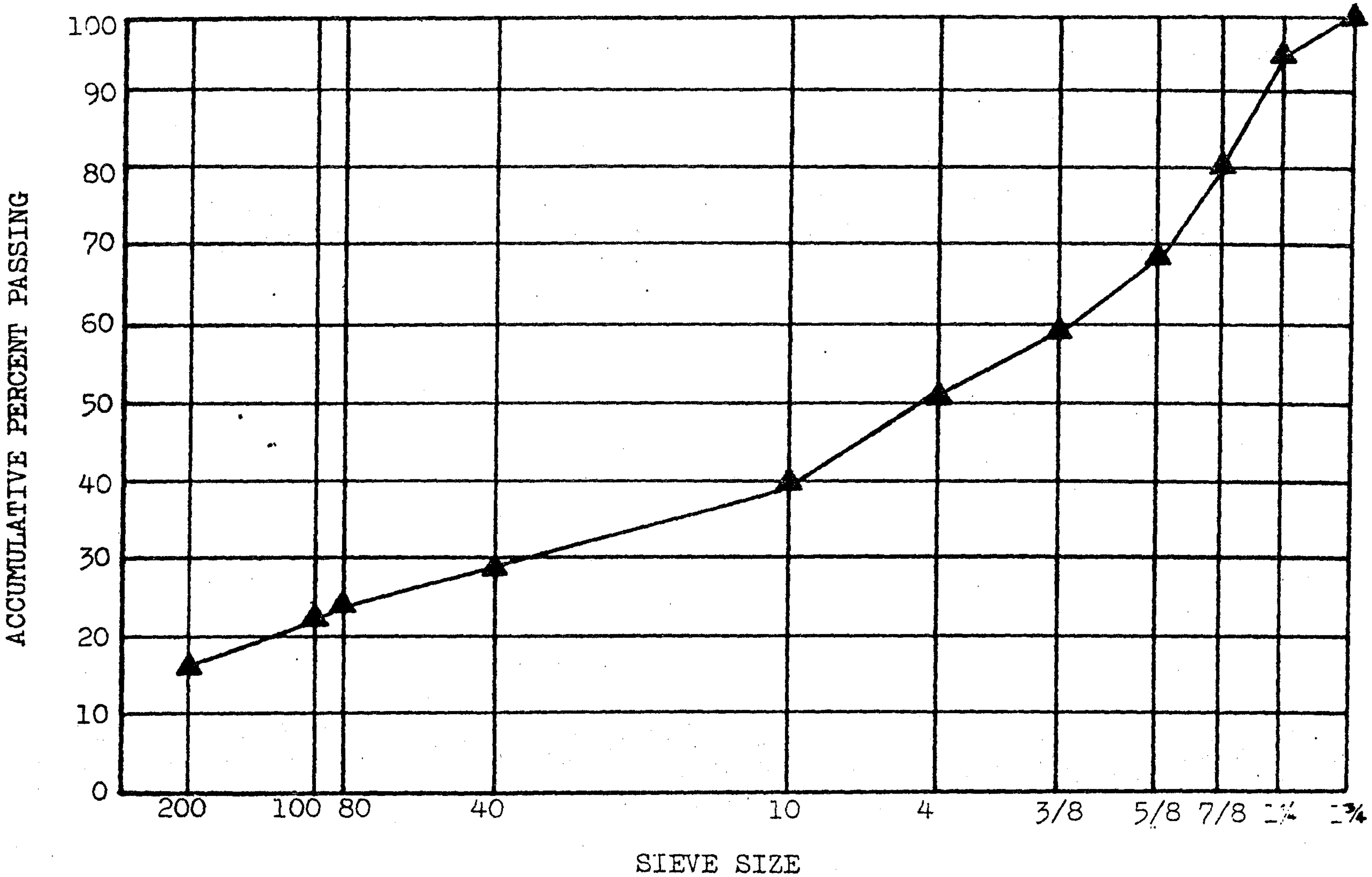

Figure 3. Washed sieve analysis of Garner-Ross sandstone.

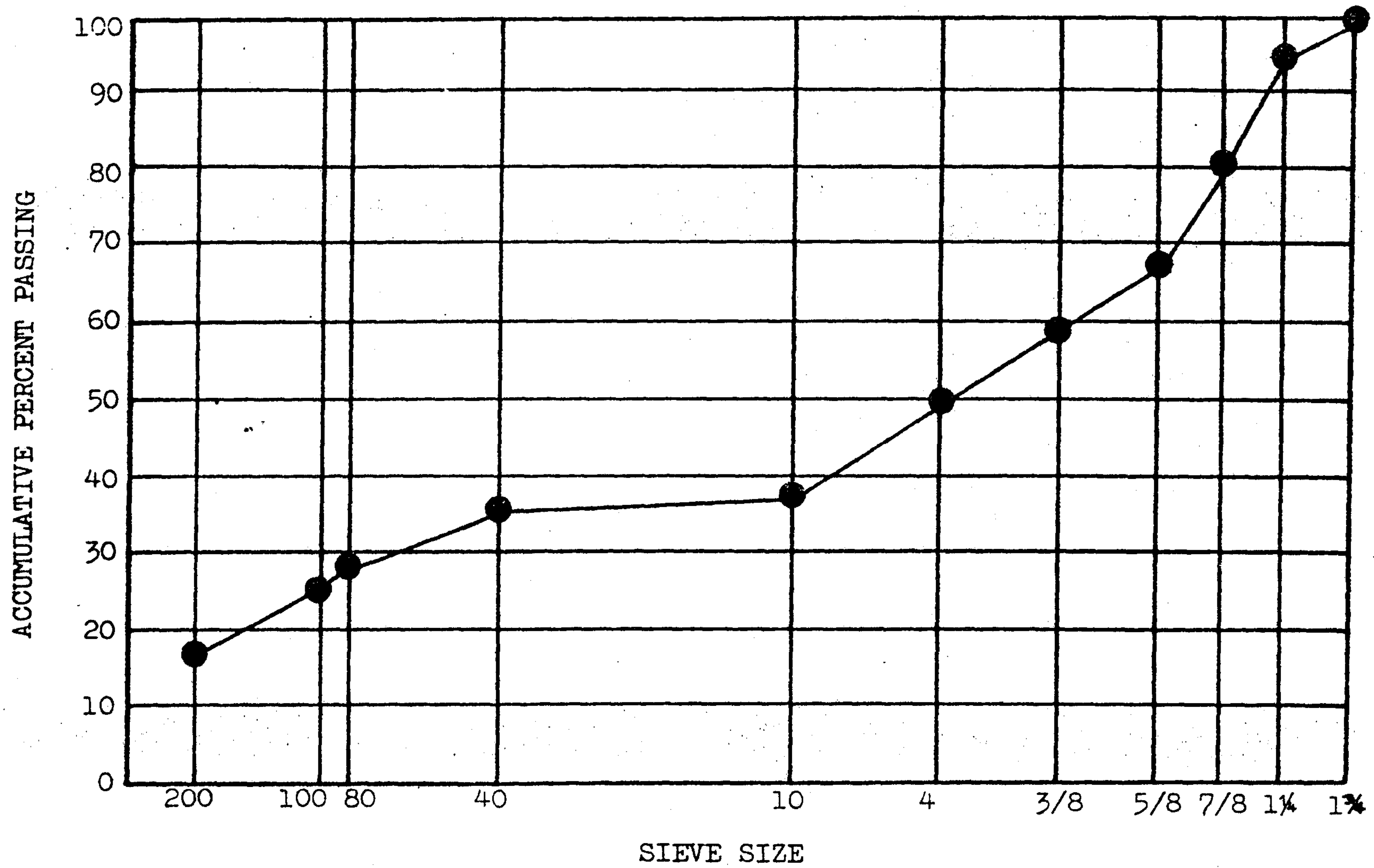

Figure 4. Washed sieve analysis of McDonough Brothers crushed limestone.

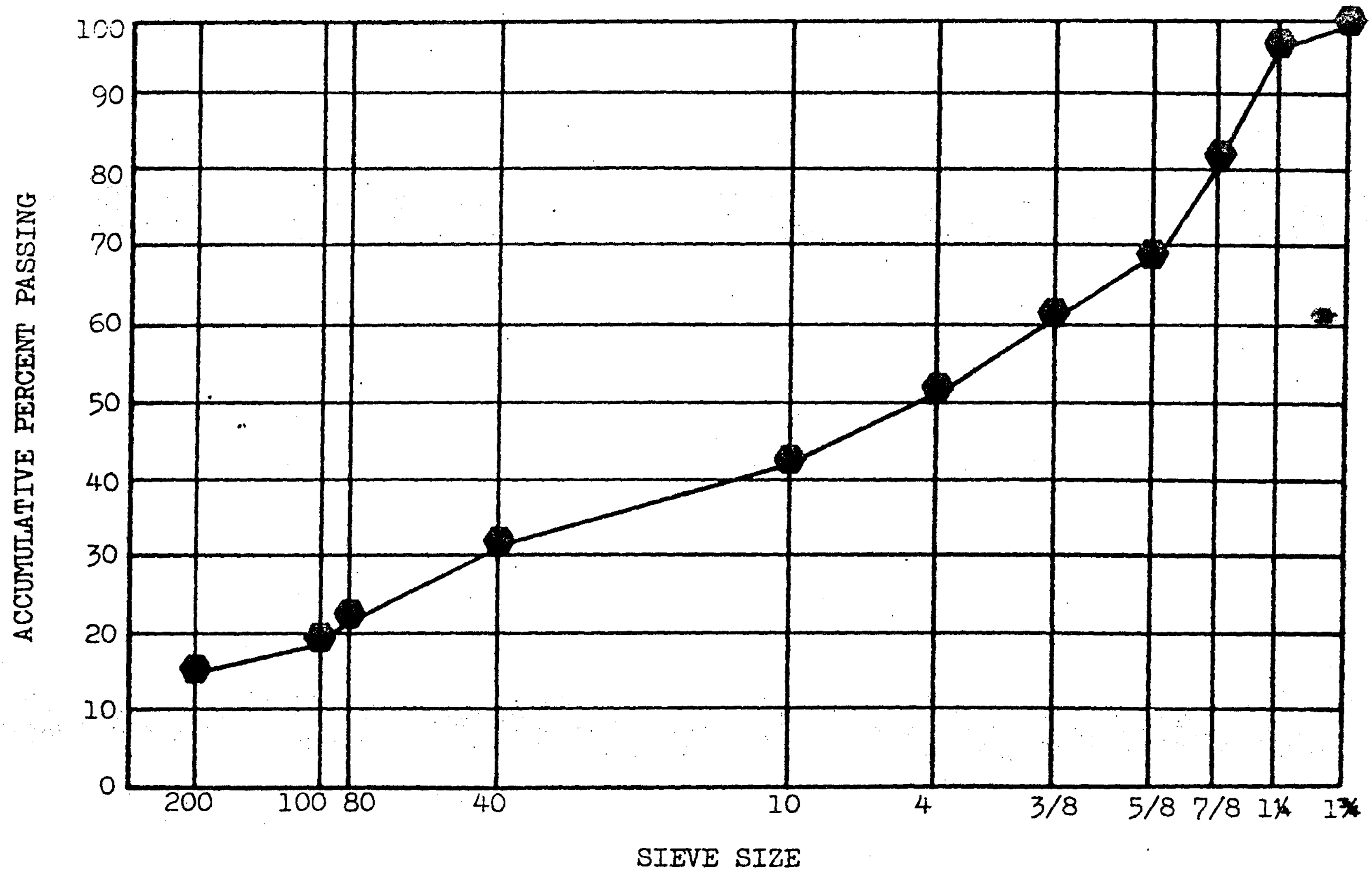

Figure 5. Washed sieve analysis of crushed caliche gravel.

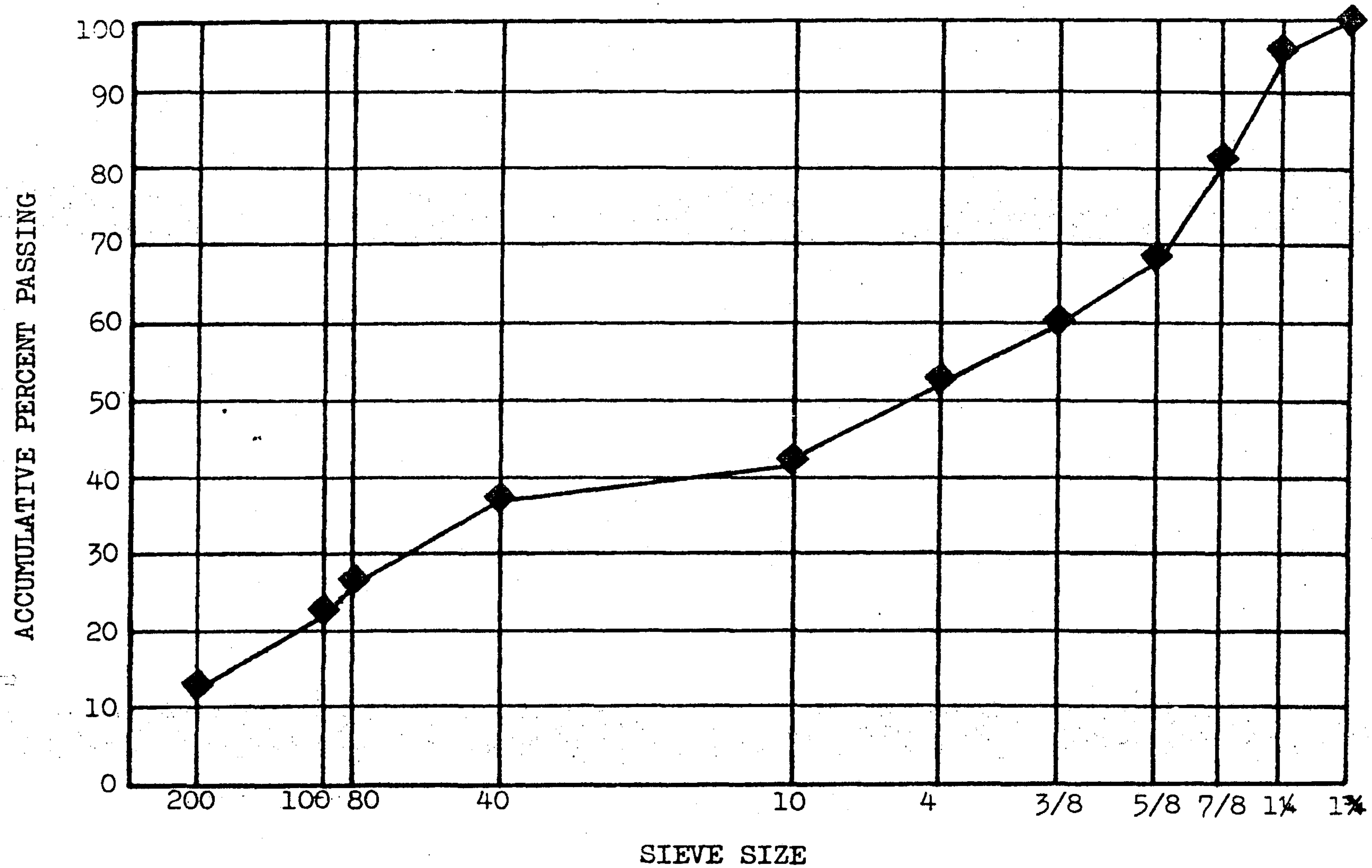

Figure 6. Washed sieve analysis of crushed sandstone.

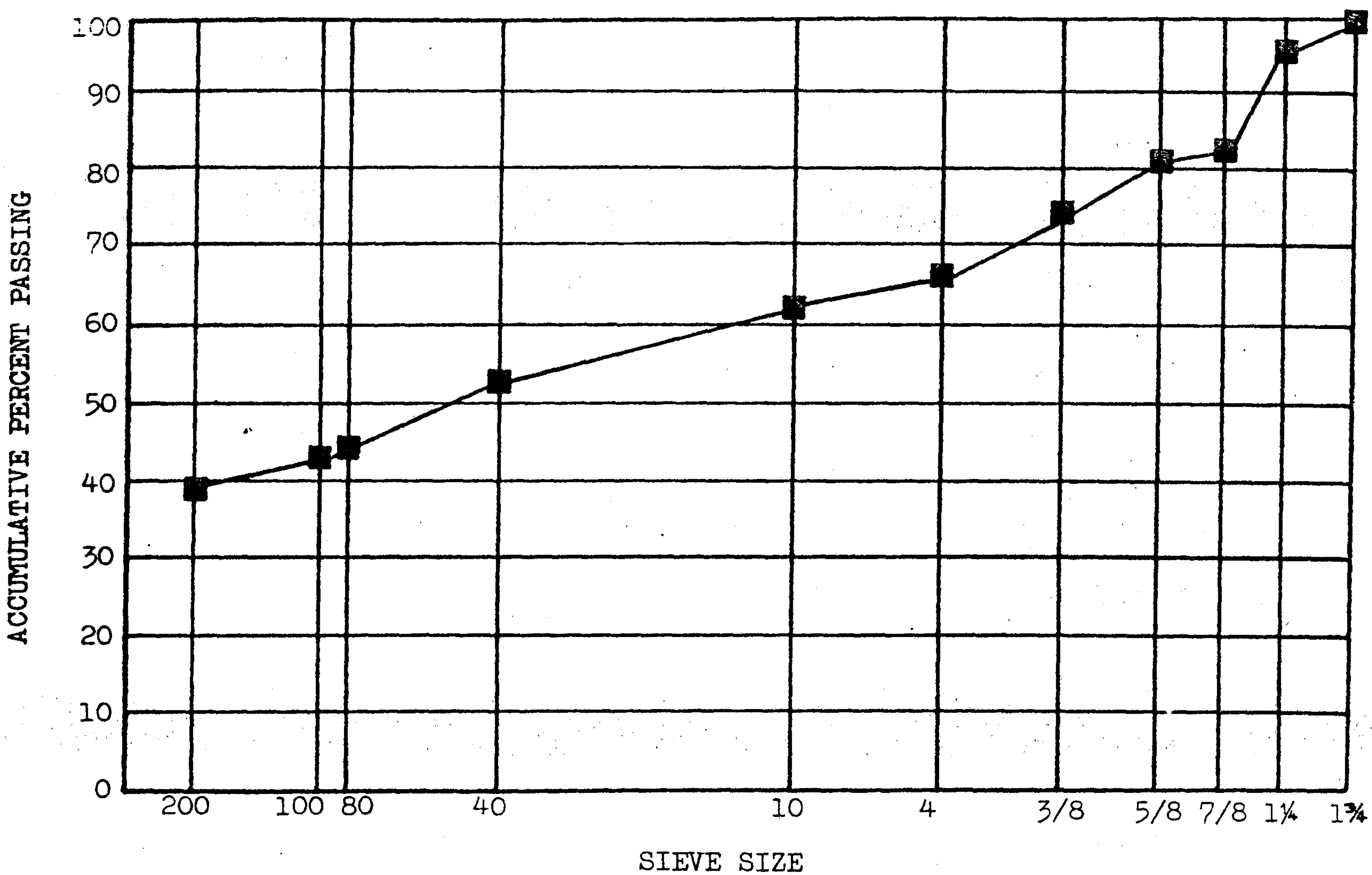

Figure 7. Washed sieve analysis of Dallas Austin chalk.

McDonough Brothers limestone	2.70
Mack Pit crushed caliche gravel	2.67
"74" Ranch Pit crushed sandstone	2.54

In addition to the five aggregates described above three sands were selected together with a sand gravel material. All of these materials are locally abundant and represent marginal aggregate for use as a base course. The properties of these aggregates are shown in Table 2 while the gradations are shown in Figures 8 to 11. The Lamb County sand is typical of "blow sands" found in the high plains of West Texas. The sample was obtained from the right-of-way along FM 168 south of Olton, Texas.

The Wheeler County sand was taken from Sweetwater Creek near FM 182. This sand is typical of river sands found in areas to the immediate east of the Texas high plains.

The Jasper County sand is a typical East Texas sand and was utilized as hot mixed stabilized base course on U.S. 96 in Jasper County.

The Hidalgo County sand-gravel was obtained from the Beck Pit which is located about 6 miles west of Mission.

Asphalt. Texas specifications generally require the use of either an AC 10 or AC 20 in hot plant-mixed, bituminous aggregate base courses (22). Most of the black base highway test sections which were investigated by field performance criteria were constructed with AC 10. Consequently, the asphalt cement used in the laboratory testing program was an AC 10 supplied by EXXON Refinery in Baytown, Texas.

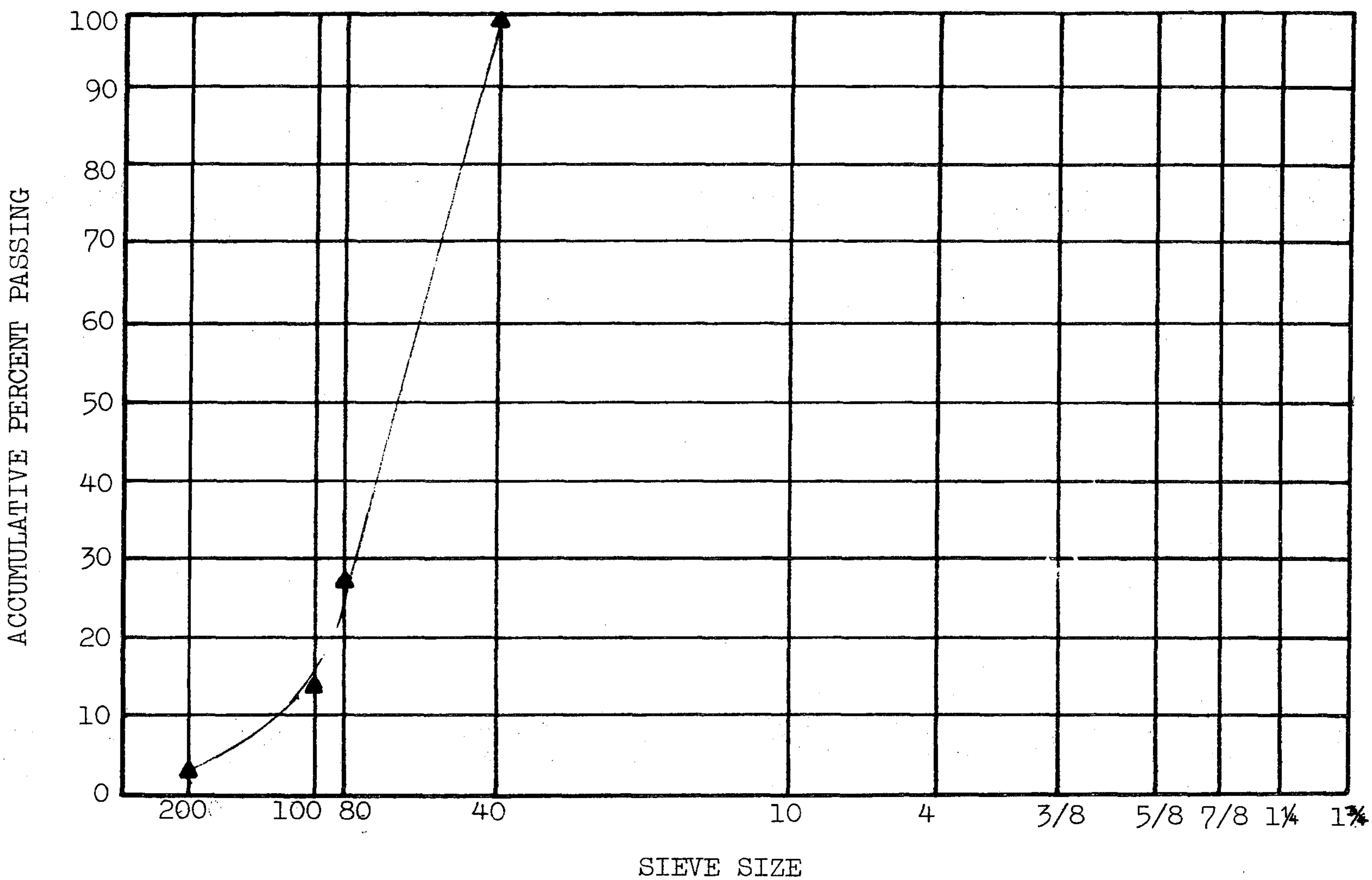

Figure 8. Washed sieve analysis of Lamb County Sand (District 5-FM 168).

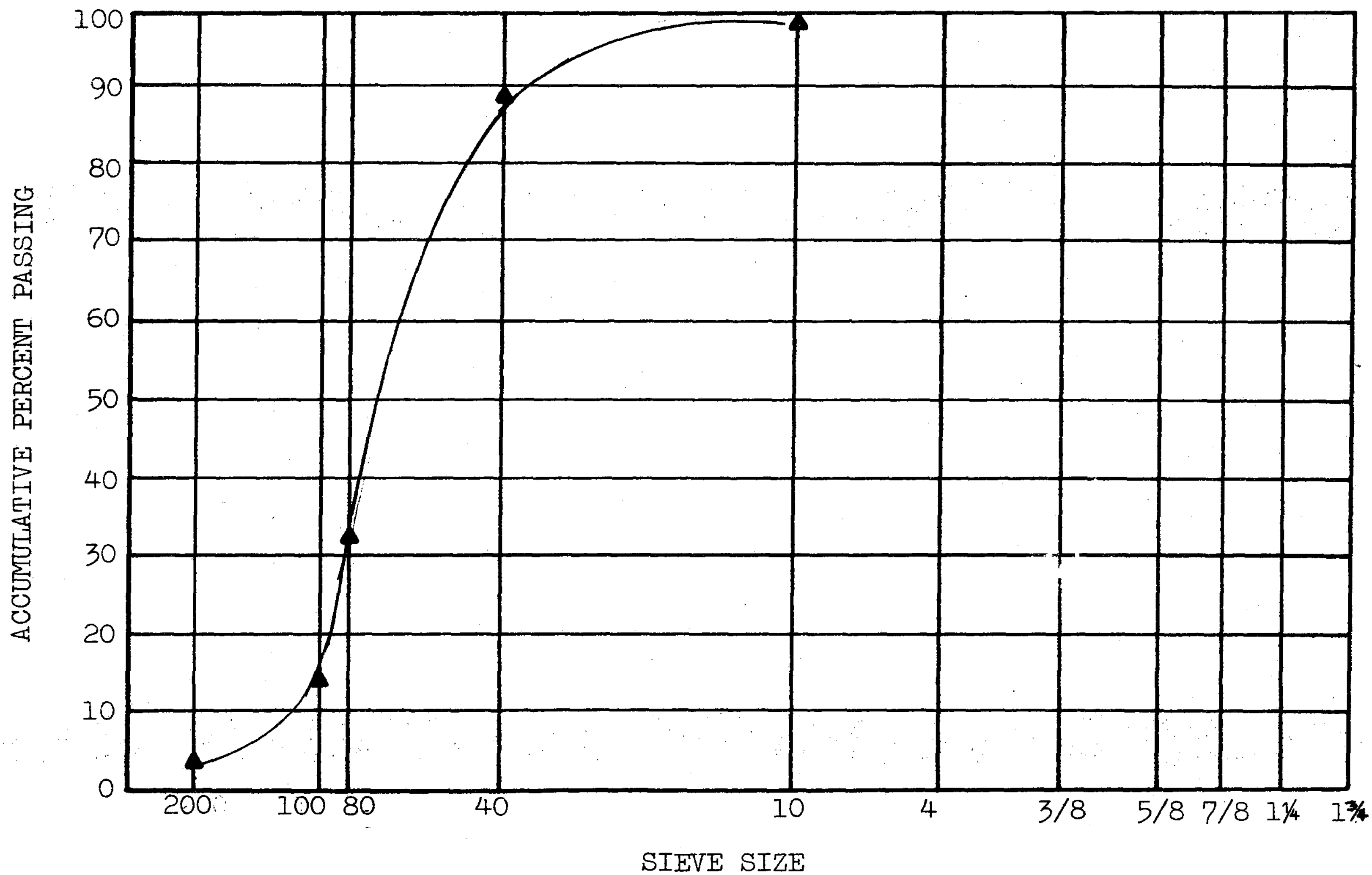

Figure 9. Washed sieve analysis of Wheeler County sand (District 25-FM 3182).

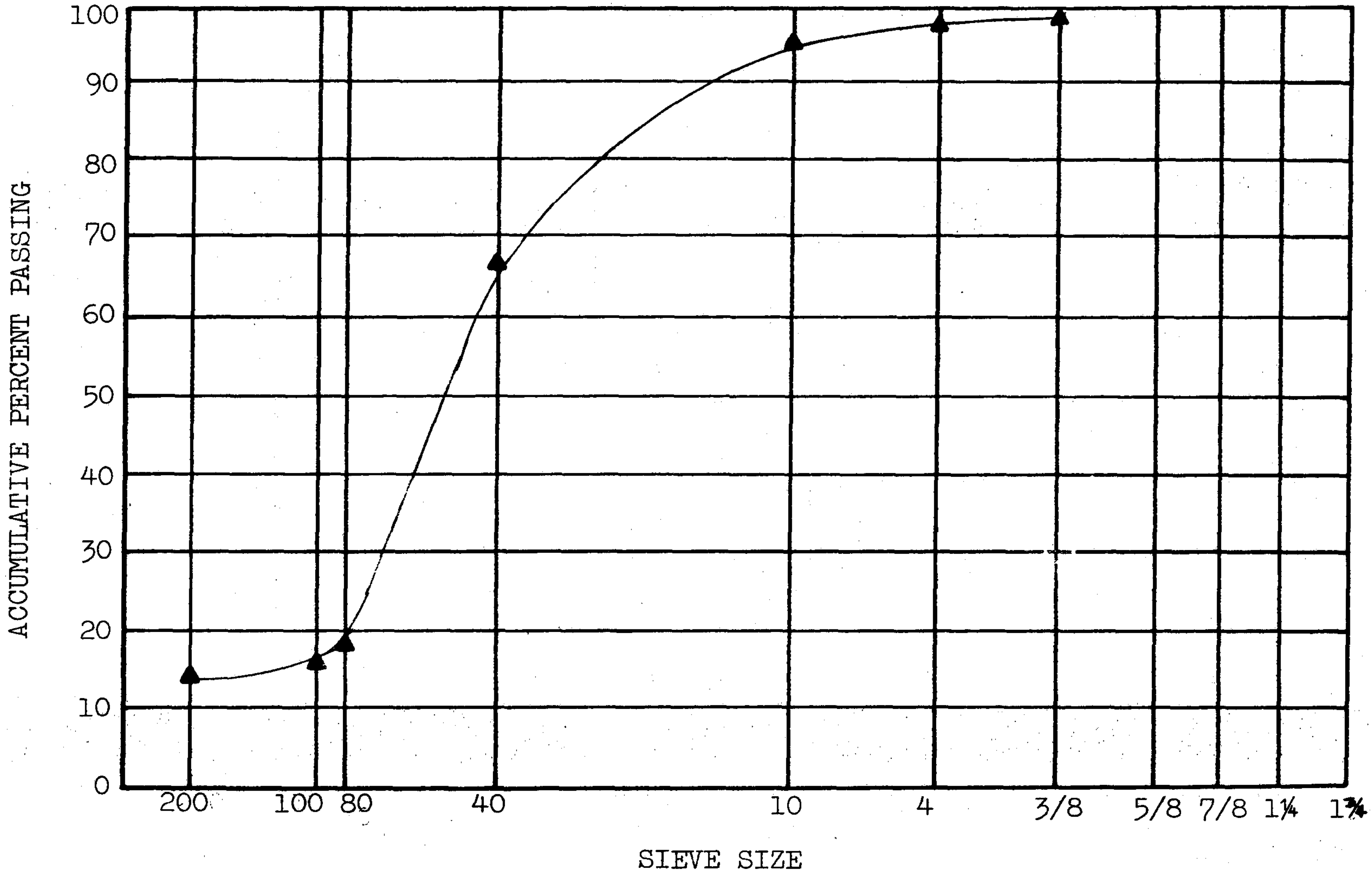

Figure 10. Washed sieve analysis of Jasper County Sand (District 20-US 96).

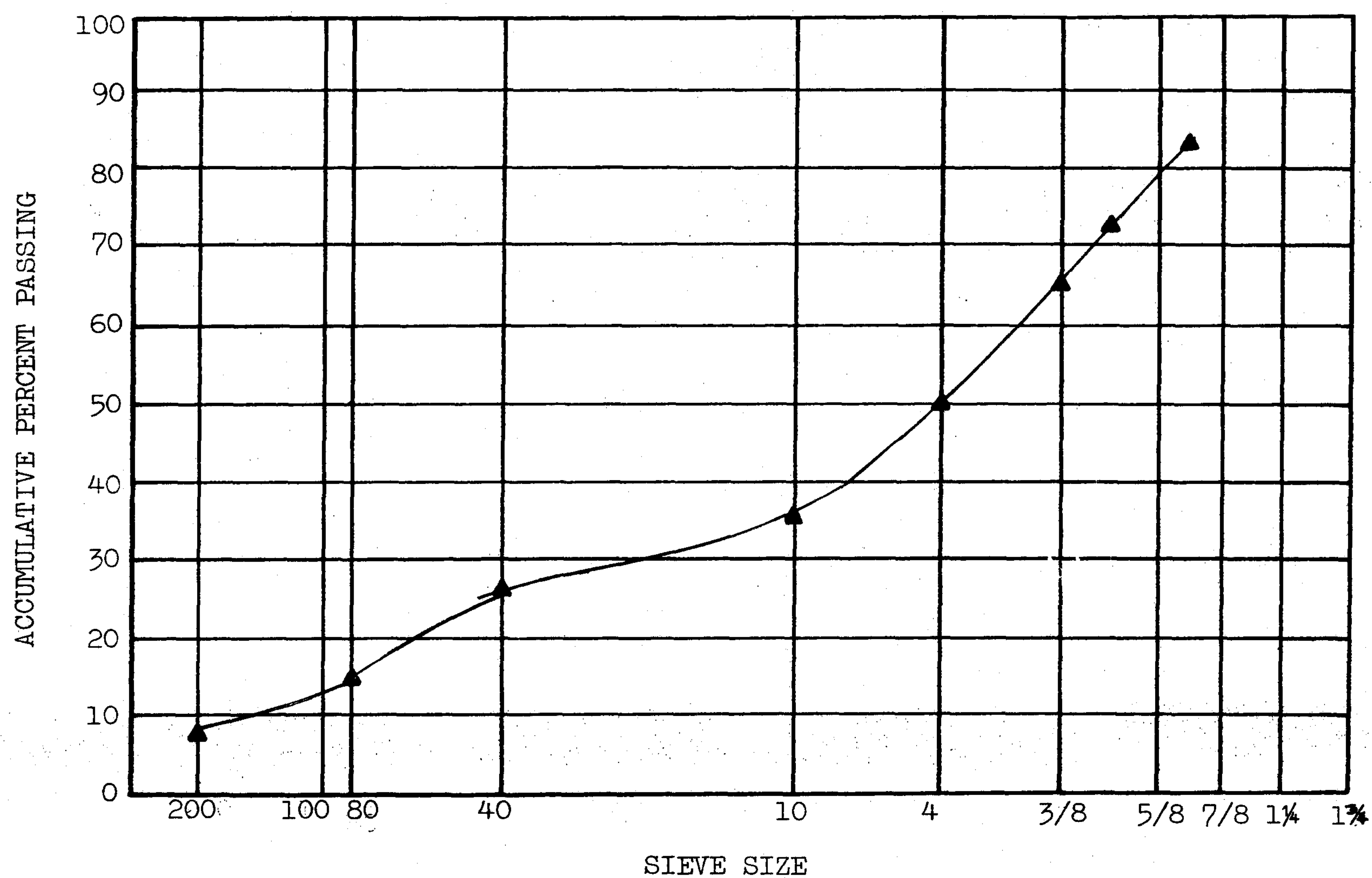

Figure 11. Washed sieve analysis of Hidalgo County Sand Gravel (District 21 Beck pit).

The tests which were conducted on the asphalt cement include (20 and 23, respectively):

1. penetration at 39.2F (4C), ASTM D 5, AASHTO T 49,
2. penetration at 77F (25C), ASTM D 5, AASHTO T 49,
3. thin-film viscosity at 77F (25C), ASTM proposed,
4. absolute viscosity by vacuum capillary tube at 140F (60C), ASTM D 2170, AASHTO T 202,
5. kinematic viscosity by gravity-flow capillary tube at 275F (135C), ASTM D 2171, AASHTO T 201, and
6. the ring and ball softening point, ASTM D 36, AASHTO T 53.

Test Results and Test Methods. Results of these standard tests are shown in Table 3. Penetration ratio, penetration index and the stiffness of the asphalt cement were determined. The penetration ratio is the penetration at 39.2F divided by the penetration at 77F and is an indication of the temperature susceptibility of the asphalt. The penetration index was determined from a nomograph which uses the ring and ball softening point and penetration at 77F (25C) as parameters (24). The penetration index gives an indication of the rheology, or flow characteristics, of an asphalt and an asphalt with a penetration index between minus two (-2) and plus two (+2) is considered to be a normal asphalt. The stiffness modulus of the asphalt cement was determined from a nomograph which uses loading time, ring and ball softening point, and penetration index as parameters (25).

Test Program and Laboratory Samples. The laboratory testing program is illustrated in Figure 12. The five black base mixtures were molded

Table 3. Asphalt properties

Sample Number	Original Asphalt	Recovered Asphalt LS-5	CCG-3	DAC-5	Average
Penetration					
@ 39.2°F (4°C)	15	14	14	12	13
@ 77 F (25 C)	77	56	50	46	51
Viscosity					
@ 77°F (25°C) (poises)	0.91×10^6	3.3×10^6	4.2×10^6	2.6×10^6	3.37×10^6
@ 140°F (60°C) (poises)	1550		3630	2960	3300
@ 275°F (135°C) (stokes)	2.80	3.45	3.66	3.70	3.60
Ring and Ball Softening Point	120 F		122 F	120 F	121 F
Penetration Ratio	0.190	0.25	0.28	0.26	0.26
Penetration Index	-0.6		-1.1	-1.7	-1.4
Stiffness @ 68 F(20 C) (0.1 seconds loading)	850 psi		1700 psi	2810 psi	2700 psi

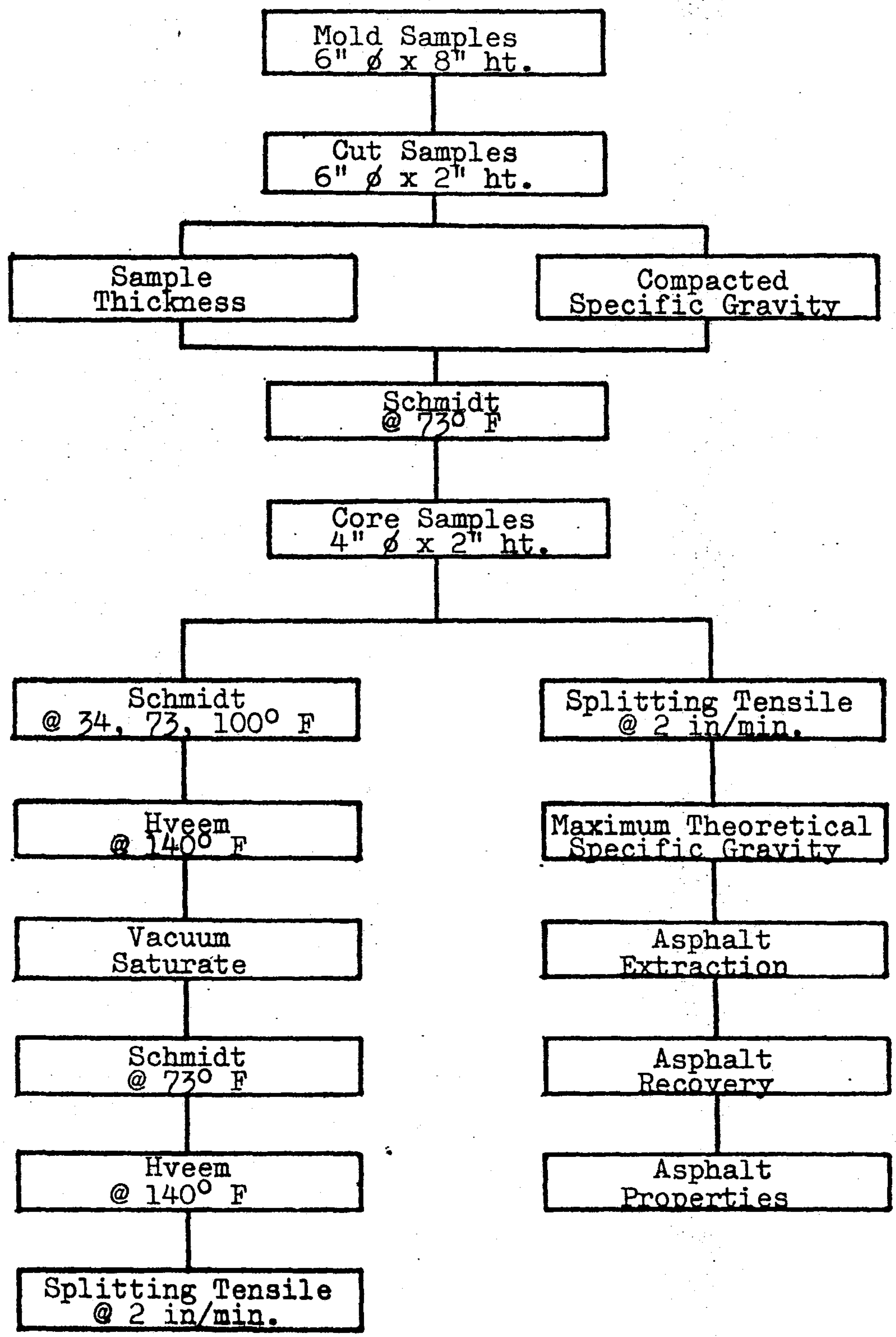

Figure 12. Testing program for the laboratory molded black base mixtures.

according to test method Tex-126-E (26). Black base samples were molded at various asphalt contents such that an optimal asphalt content for each aggregate type could be determined. Test method Tex-126-E requires that the 6-inch in diameter by 8 inches in height samples be failed in unconfined compression and from these results an optimum asphalt content determined. Rather than failing the samples, however, it was concluded that the samples should be first subjected to an available non-destructive testing program. Consequently, in order that the samples be of proper dimensions for further testing, the 6-inch by 8-inch samples were sliced to approximately 2-inch thicknesses with a 20-inch diameter, diamond embedded, water cooled, hand-fed saw. For similar reasons, the samples were later cored to 4 inches in diameter by 2 inches in height with a water cooled diamond bit.

Because both the slicing and coring operations required the use of water, the samples were allowed to dry to a constant weight at 90°F and 25 percent relative humidity. After drying, an average sample thickness was determined. The compacted specific gravity, or bulk density, of the samples was determined (ASTM D2726).

A method for determining the resilient modulus (M_R) of asphalt treated mixes has been presented by Schmidt (27). The resilient modulus is generally defined as the apparent Young's modulus, or stiffness, E, of a viscoelastic material under short-duration, dynamic loads. The loading condition corresponds to various studies relating laboratory-measured M_R values to field behavior within the frame-work of multilayer elastic design theory. In the procedure, a light pulsating load is applied through a load cell across the vertical diameter of the specimen. This causes an elastic deformation across the horizontal diameter. The test procedure is also termed the diametral method of test for the resilient modulus of asphalt-treated mixes. A load duration of 0.1 seconds is repeated 20 times per minute across the vertical diameter of the specimen. The deflection of the specimen is monitored by a pair of compensating, highly sensitive, Schaevitz transducers (0.005 inches full-scale deflection). The transducers are mounted directly on the specimen and ride with any vertical movement while measuring the dynamic deformation. Output from the transducers and load cell are recorded on a strip chart recorder.

The resilient modulus, assumed equal to the modulus of elasticity (E), is calculated from the expression shown in equation 1 (27).

$$M_R = P\ (v + 0.2732)\ /\ t\Delta \qquad (1)$$

where, the dynamic load (P) and the total deformation (Δ) are taken from the recorded traces, t is the thickness of the specimen, and v is Poisson's ratio, assumed to be 0.35 for asphalt concrete. A comparison study was also conducted between the M_R values obtained through direct tension, compression, and Schmidt or diametral test methods and it was found that "even when a relatively side range of values is assumed for Poisson's ratio, the diametral method gives M_R values within 25 percent of the values found by direct measurement of the tensile or compressive M_R on asphalt concrete mixes" (27).

After slicing and allowing to dry and prior to coring, the 6-inch diameter by 2-inch high laboratory molded specimens were tested by the diametral method at 73°F. The test apparatus utilized was a modification of that reported by Schmidt. Rather than using the Schaevitz transducers, the modified apparatus was equipped with two compensating, spring-loaded, linear variable differential transformers (LVDT). The electronics and framework were assembled by Texas Transportation Institute personnel.

The samples were cored to 4-inch diameter by 2 inches in height and again tested for resilient modulus by the diametral test method at 73°F. By comparing M_R values for the 6" x 2" samples and the 4" x 2" samples, the effect of coring on the black base mixtures was investigated. Rather than using the modified apparatus the 4-inch cores were tested with a newly acquired Mark III Resilient Modulus Apparatus manufactured by the Retsina Company (Figures 12a and 12b). It was first necessary to investigate the accuracy of the two sets of testing equipment such that comparable results could be obtained. Lucite samples were tested on both pieces of equipment at various loadings. From the results, it was found that for deformation measurements greater than 80 micro-inches the two moduli devices obtained quite comparable results. For deformation measurements of less than 80 micro-inches, resilient modulus values for the modified Schmidt apparatus became questionable. Likewise, for deformations less than 20 micro-inches, M_R values obtained with the Mark III Apparatus may also be questionable. Deformations of less than 80 micro-inches at 73°F generally occurred only on the sandstone and limestone mixtures at or near optimum asphalt contents. Deformations of less than 20 micro-inches occurred only when the samples were tested at 34°F.

The 4-inch by 2-inch samples were tested according to the Schmidt procedure at 34, 73, and 100°F. Through this testing program, the effect of temperature on the black base mixtures was evaluated.

After subjecting the black base mixtures to the Schmidt procedure,

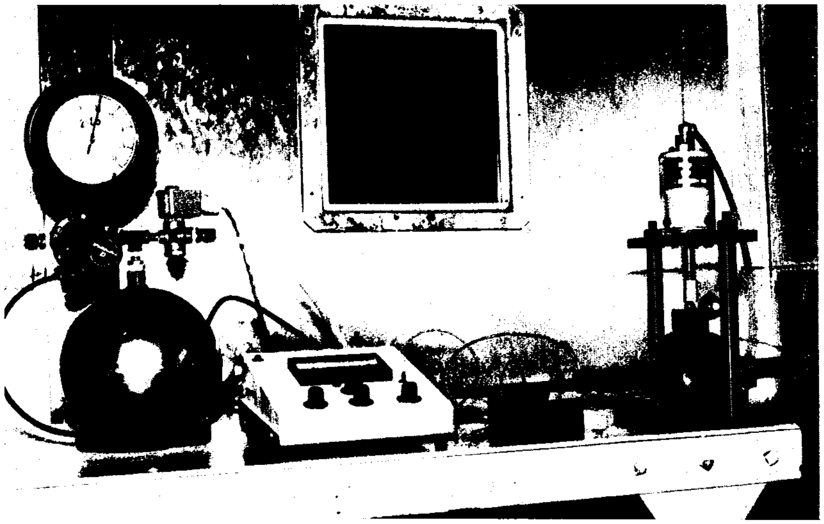

Figure 12a. Overall View of Mark III Resilient Device.

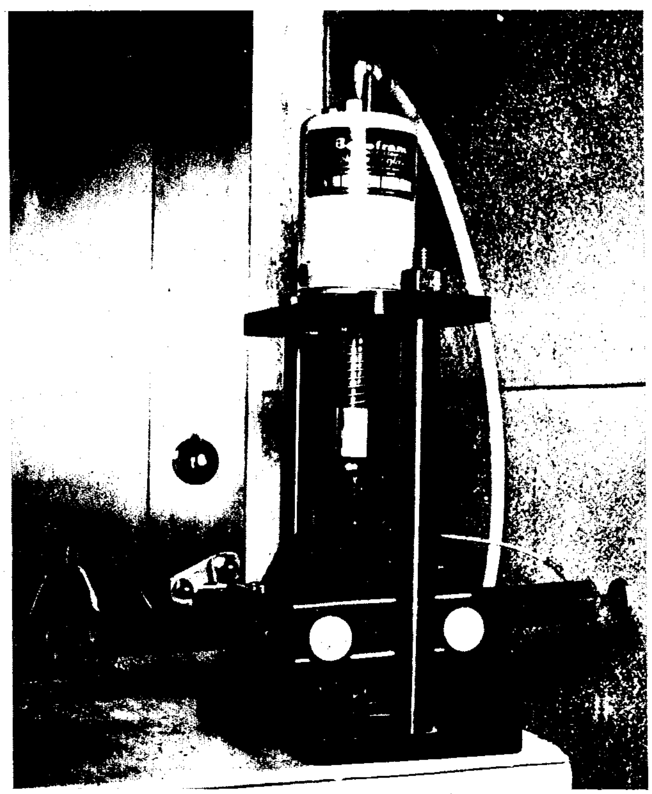

Figure 12b. Close-up View of Loading Frame and Transducers.

the samples were tested for stability according to the method prescribed by the Texas State Department of Highways and Public Transportation, Tex-208-F, Test for (Hollm) Stabilometer Value of Bituminous Mixtures (26).

Because the original specimens were 8 inches in height and were then sliced to approximate 2-inch thickness, there were four samples at each asphalt content. The four samples were divided into two sets such that two samples would be subjected to a vacuum saturation procedure and two would not. The purpose of the vacuum saturation procedure was to determine the effect of water on the black base mistures. At present, an acceptable national standard vacuum saturation procedure does not exist. Therefore, a test procedure which encompasses a broad range of known saturation tests was written. As a matter of convenience, the test procedure was one which most effectively utilized the available testing equipment. The test procedure is as follows:

1. weigh the samples at room temperature,
2. place samples in pycnometer and cover with water,
3. evacuate the pycnometer to a pressure of 27 psi vacuum and hold for two hours,
4. release the vacuum and remove the samples,
5. blot the samples dry and weigh,
6. place the samples in a container and cover with water,
7. place the container and samples in an environmentally controlled room at 73°F and 95% relative humidity for a period of seven days,
8. at the end of seven days soaking, remove samples from container, blot dry and weigh.

The purpose of the vacuum saturation procedure outlined above is to represent the worst possible field condition.

At the end of the above 7-day treatment the samples were tested for resilient modulus according to the diametral method. The change in M_R during the vacuum saturation and soaking process, made possible an evaluation of the effect of water on these mixtures. After testing, the saturated samples were allowed to dry to a constant weight in an environmentally controlled room at 140°F and 25 percent relative humidity.

The dry samples were again tested according to Tex-208-F and the stability values obtained. The purpose of the test sequence was to evaluate the effect of water on the stability of the bituminous mixtures.

The final step in the testing of the compacted black base samples was to fail the samples by indirect tension or splitting tensile as the test is more commonly known. The splitting tensile test was developed simultaneously, but independently, by Carneiro and Barcellos (28) in Brazil and Akazawa (29) in Japan. The test consists of loading a cylinder in diametral compression and was developed to measure the tensile strength of portland cement concrete. The suitability of using the splitting tensile for asphaltic concrete samples was demonstrated in studies by Anderson and Hahn (30), Breen and Stevens (31), and Livneh and Schlarsky (32). Kennedy and others (33, 34, 35, 36, 37) at the Center for Highway Research, University of Texas at Austin have conducted extensive studies in this field.

The splitting tensile test involved loading a 4-inch diameter by 2-inch high specimen in diametral compression (Figures 12c and 12d). The test was performed on a model TT-D Instron Universal Tester. The samples were failed at a uniform stress rate (loading rate) of 2.0 inches per minute and a temperature of 73°F. Horizontal deformation was measured by two cantilever strain

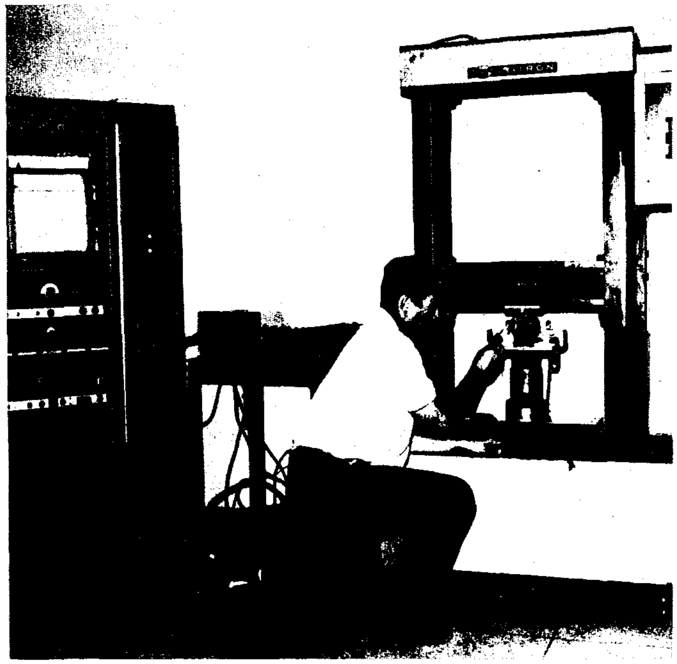

Figure 12c. Splitting Tensile Tester.

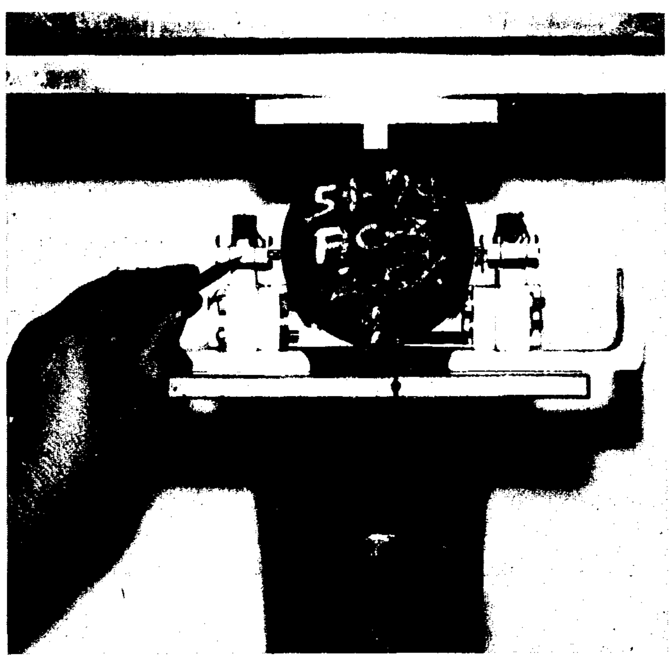

Figure 12d. Specimen in Test Frame of Splitting Tensile Tester.

gage transducers and deflections of these transducers, as well as the applied load, were recorded through a B and F Oscillograph Model 3006/DL chart recorder on light sensitive paper.

The splitting tensile test data were analyzed through the use of a computer program, a listing of which is available in Appendix III. Stress values were determined according to the procedure outlined by Britten, Bynum and Ledbetter (38).

Using the y-axis as the axis of load application, the compressive stress, σ_y, is given as (38).

$$\sigma_y = \frac{-2P}{t}\left(\frac{2}{d-2y} + \frac{2}{d+2y} - \frac{1}{d}\right) \qquad (2)$$

where P = thickness of specimen

t = diameter of specimen

y = distance from the origin.

The horizontal compressive stress along the horizontal diameter varies from a maximum of 6P/ntd at the center to zero at the center to zero at the circumference of the specimen. The horizontal stress, σ_x, normal to the axis of loading is tensile and is given by (38).

$$\sigma_x = \frac{2P}{td} \qquad (3)$$

The modulus of elasticity values were determined using horizontal deformation criteria and employing the equation submitted by Schmidt (27): that is,

$$E = \frac{P\ (v + 0.2723)}{t\Delta} \qquad (4)$$

where v = Poisson's ratio

Δ = total horizontal deformation.

Assuming purely elastic behavior, the strain across the specimen, ε_x, may be calculated by

$$\varepsilon_x = \frac{\sigma_x}{E}$$

After the samples were failed by indirect tension they were broken into smaller pieces and the maximum specific gravity determined (ASTM D2041) (20). These results were used in calculating the percent air voids (ASTM D3203) (20). Extraction and recovery tests were run on the bituminous mixtures and the properties of the residual asphalt cement determined (Table 3).

LABORATORY TEST RESULTS - DISTRICT 15 AND 18 MATERIALS

Specific Gravity and Air Voids Contents

Figures 13 through 17 illustrate the relationship between compacted specific gravity and asphalt content for the laboratory molded black base mixtures. The sandstone (Figure 13) and limestone (Figure 14) curves indicate that maximum density occurs near nine percent and five percent asphalt, respectively. The crushed sandstone mixture appears to attain maximum density between nine and ten percent asphalt (Figure 15). From figures 16 and 17, it appears that neither the crushed caliche gravel nor the Austin chalk mixtures had attained maximum density over the range of asphalt contents investigated. Individual test results can be found in Appendix A.

Typically, density determinations are represented by the percent air voids in the bituminous mixture. Depending upon the design criteria selected, the percent air voids desired may or may not be a specification requirement. Using the Marshall design criteria, for example, the percent air voids in a hot-mix asphalt concrete base should be in the range of three to eight percent (39). Although not a routine part of the Hveem design method, it is generally suggested that an effort be made to provide a minimum percent air voids of approximately four percent (39). In general, most states specify a range of air voids content from three to seven percent (22). Texas, however, specifies that the optimum asphalt content shall be "the percentage slightly higher than the break in the Asphalt-Voids Ratio (AVR) curve" (26).

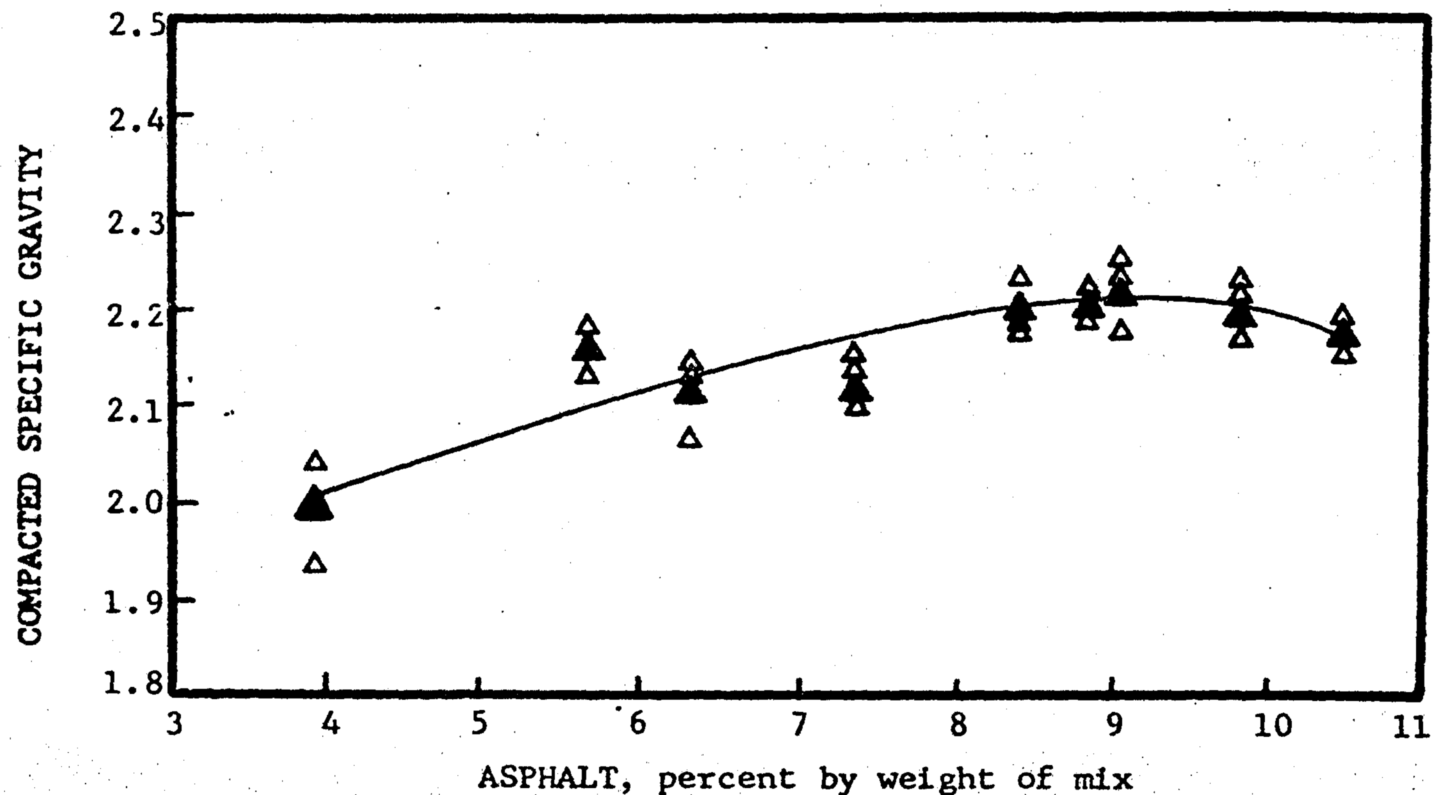

Figure 13. Compacted specific gravity versus asphalt content for sandstone mixtures.

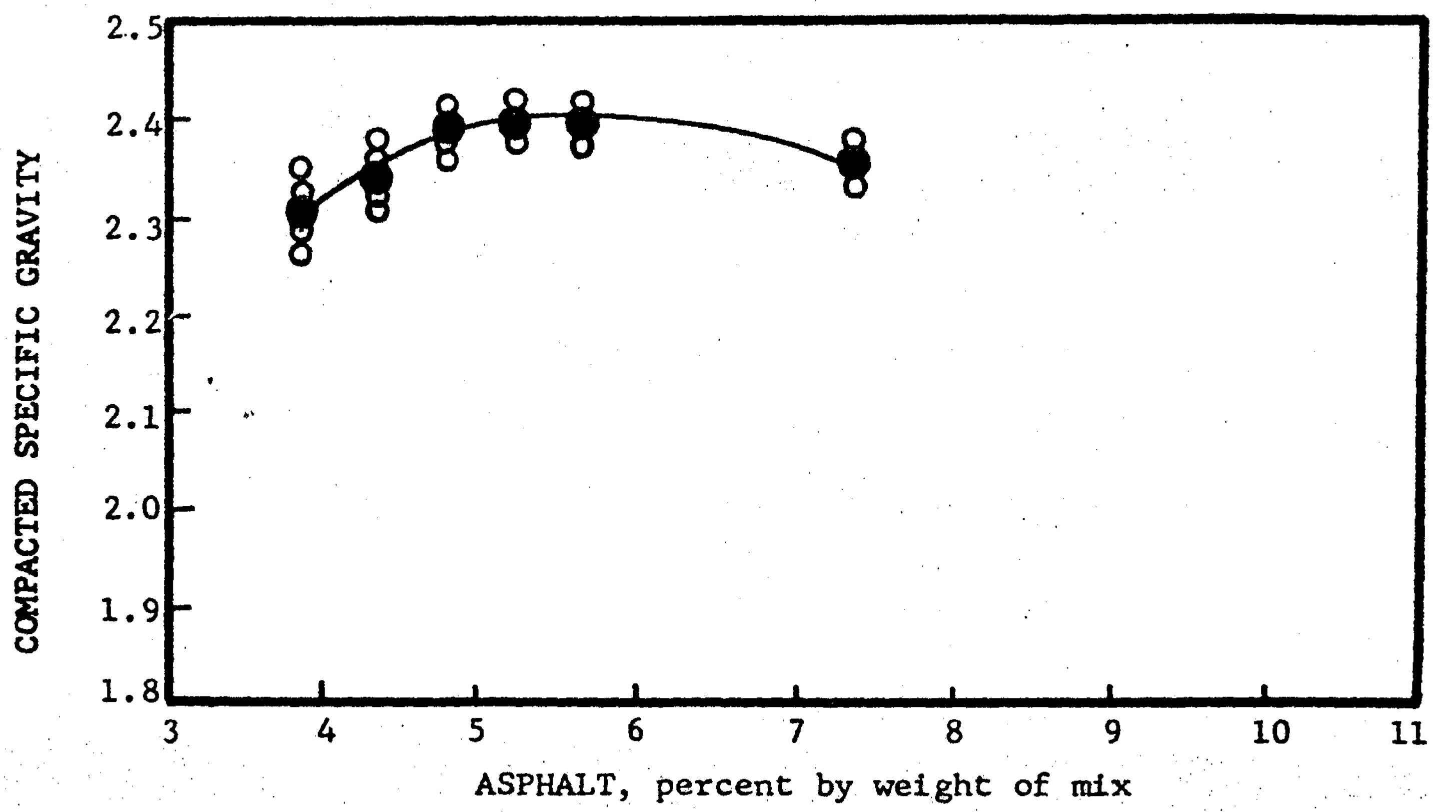

Figure 14. Compacted specific gravity versus asphalt content for limestone mixtures.

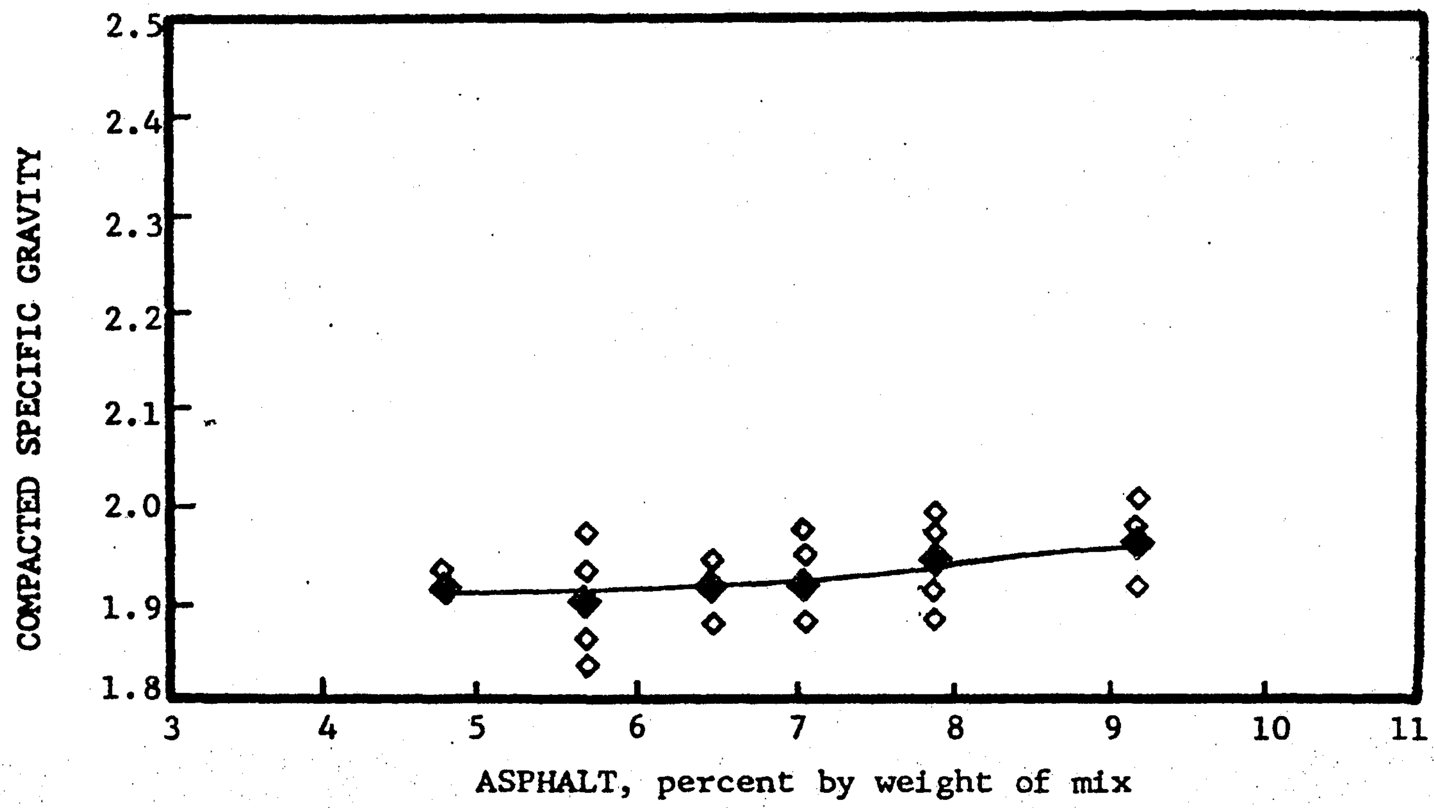

Figure 15. Compacted specific gravity versus asphalt content for crushed sandstone mixtures.

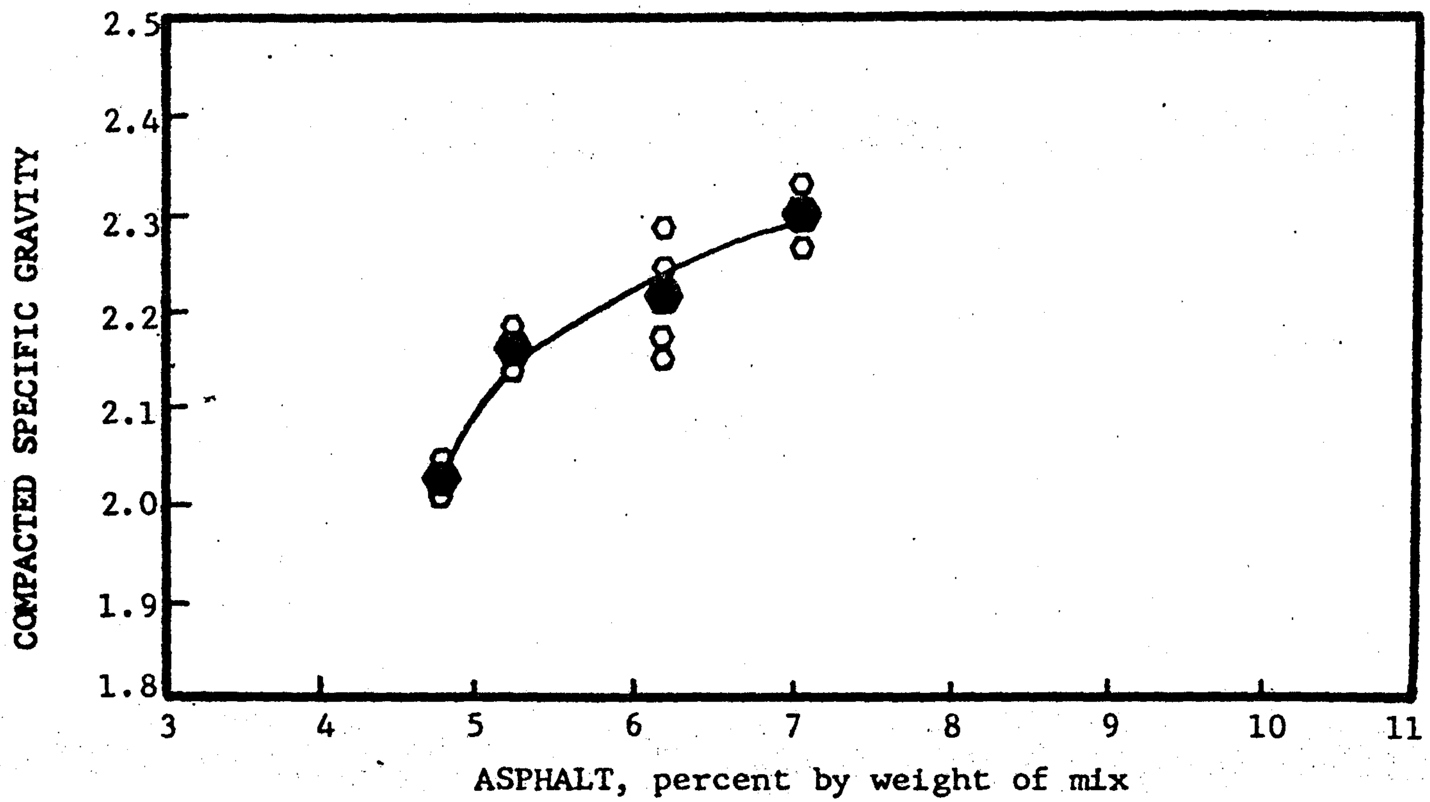

Figure 16. Compacted specific gravity versus asphalt content for crushed caliche gravel mixtures.

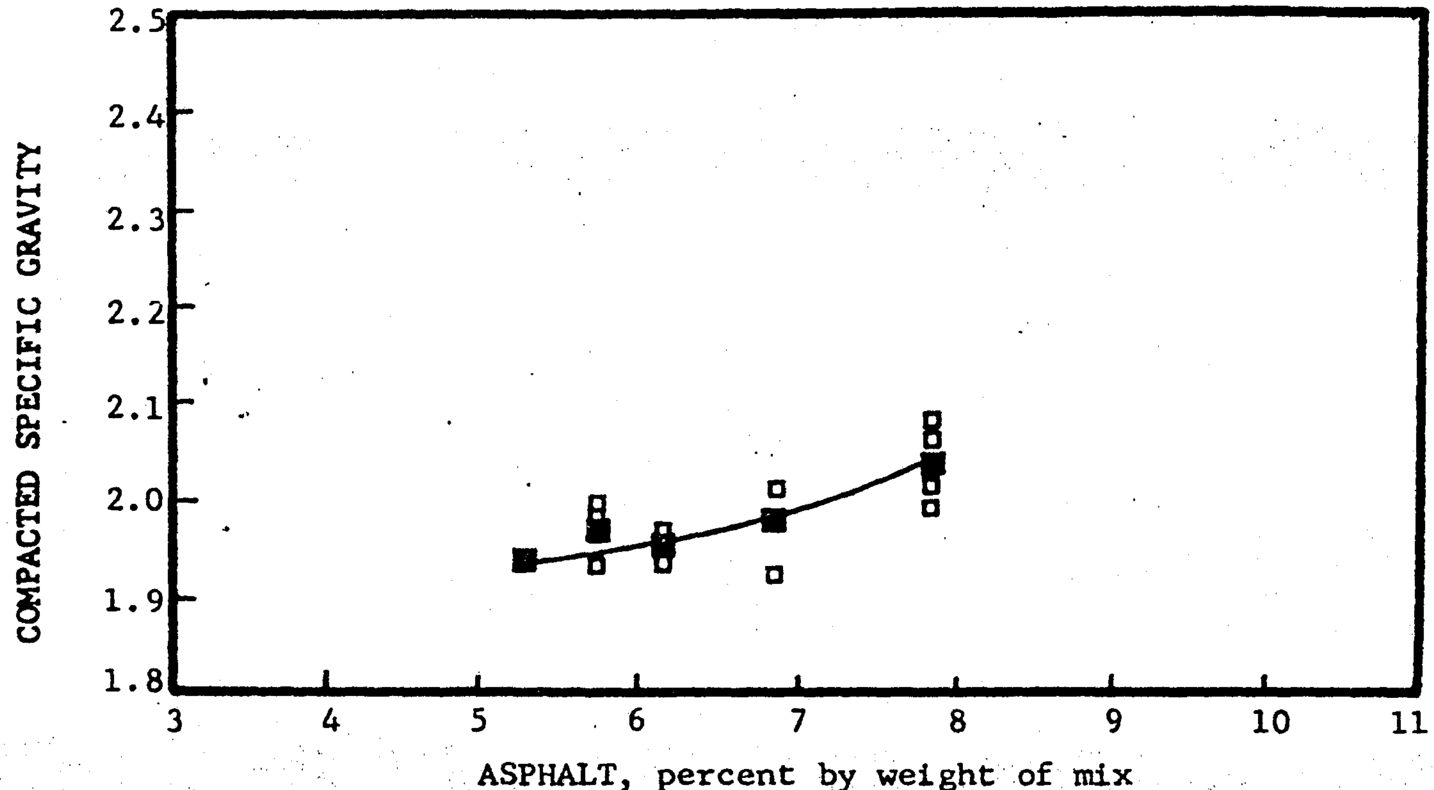

Figure 17. Compacted specific gravity versus asphalt content for Austin chalk mixtures.

Investigating the laboratory mixtures it appears that the amount of asphalt required to produce a sandstone mix with between three and eight percent air voids is in the range of 7.0 to 8.5 percent (Figure 18). The recommended asphalt content according to the Texas method of test would be 9.0 percent. Simarlarly, Figure 19 indicates that a limestone mixture with 3.5 to 5.0 percent asphalt will provide the desired air voids content. The Texas procedure also suggests an asphalt content of 5.0 for the limestone mixture. The required asphalt content for the crushed caliche gravel mixture is between 5.5 and 7.0 percent (Figure 21). The Texas procedure recommends an asphalt content of at least 7.0 percent for the crushed caliche gravel mixture. Figure 21 illustrates that crushed sandstone samples were not molded at an asphalt content high enough to produce a mix with 3.0 percent air voids. The curve indicated, however, that the optimum range would be upwards of 7.5 percent asphalt. Figure 21 tends to illustrate a reverse curve relationship which is not typical of bituminous mixtures. Apparently, the optimum asphalt content as recommended by the Texas procedure is approximately 9.0 percent. Likewise, Figure 22 illustrates that again samples were not molded at a high enough asphalt content. It appears that the chalk mixtures must have more than 7.0 percent asphalt to produce the desired air voids content. The reverse curve indicates that the Texas procedure would recommend an asphalt content above 8.0 percent as being optimum.

The Effect of Coring and Vacuum Saturation of the Resilient Modulus

The resilient modulus is generally defined as the ratio of the applied stress to the recovered strain under short-duration, dynamic loads. As explained

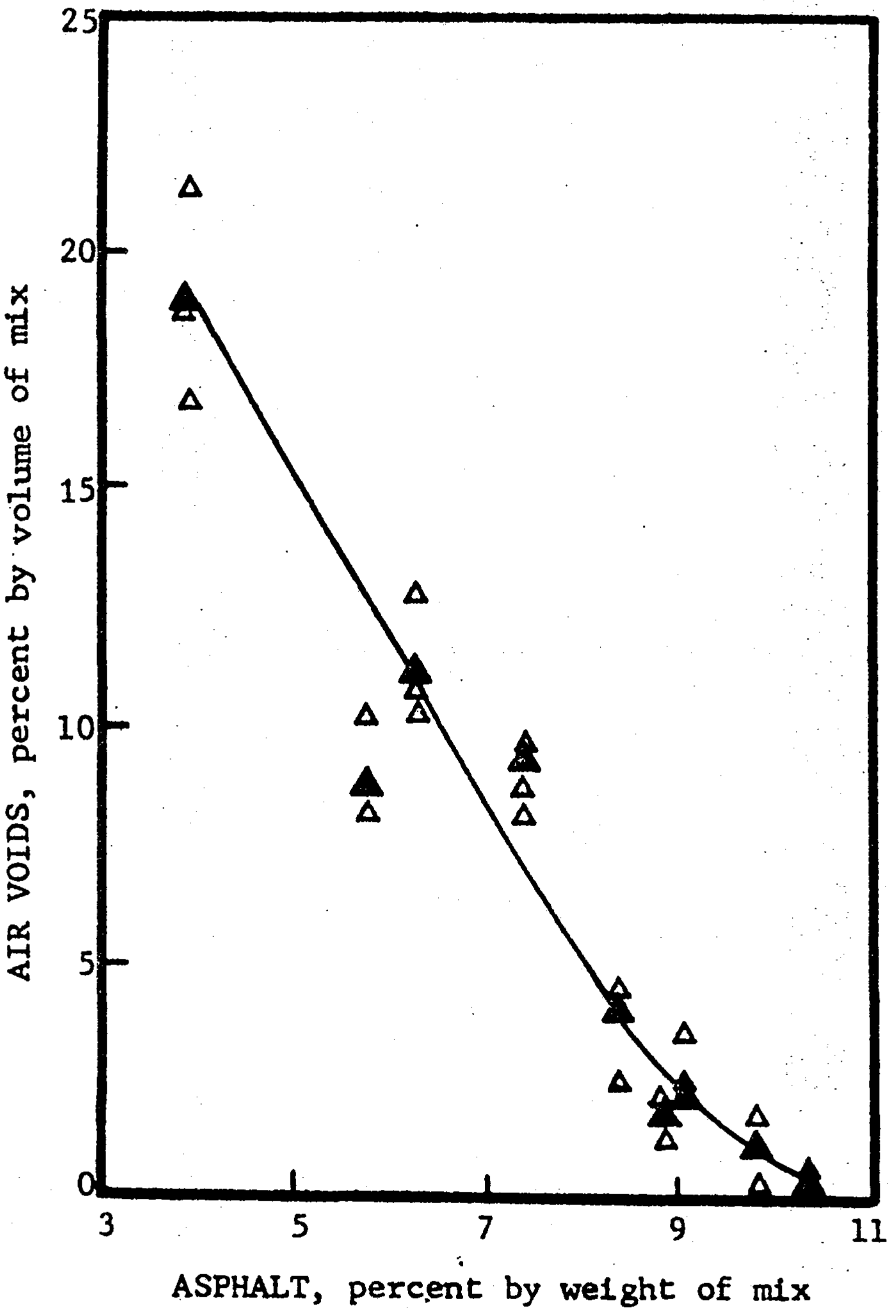

Figure 18. **Percent air voids in mix versus asphalt content for sandstone mixtures.**

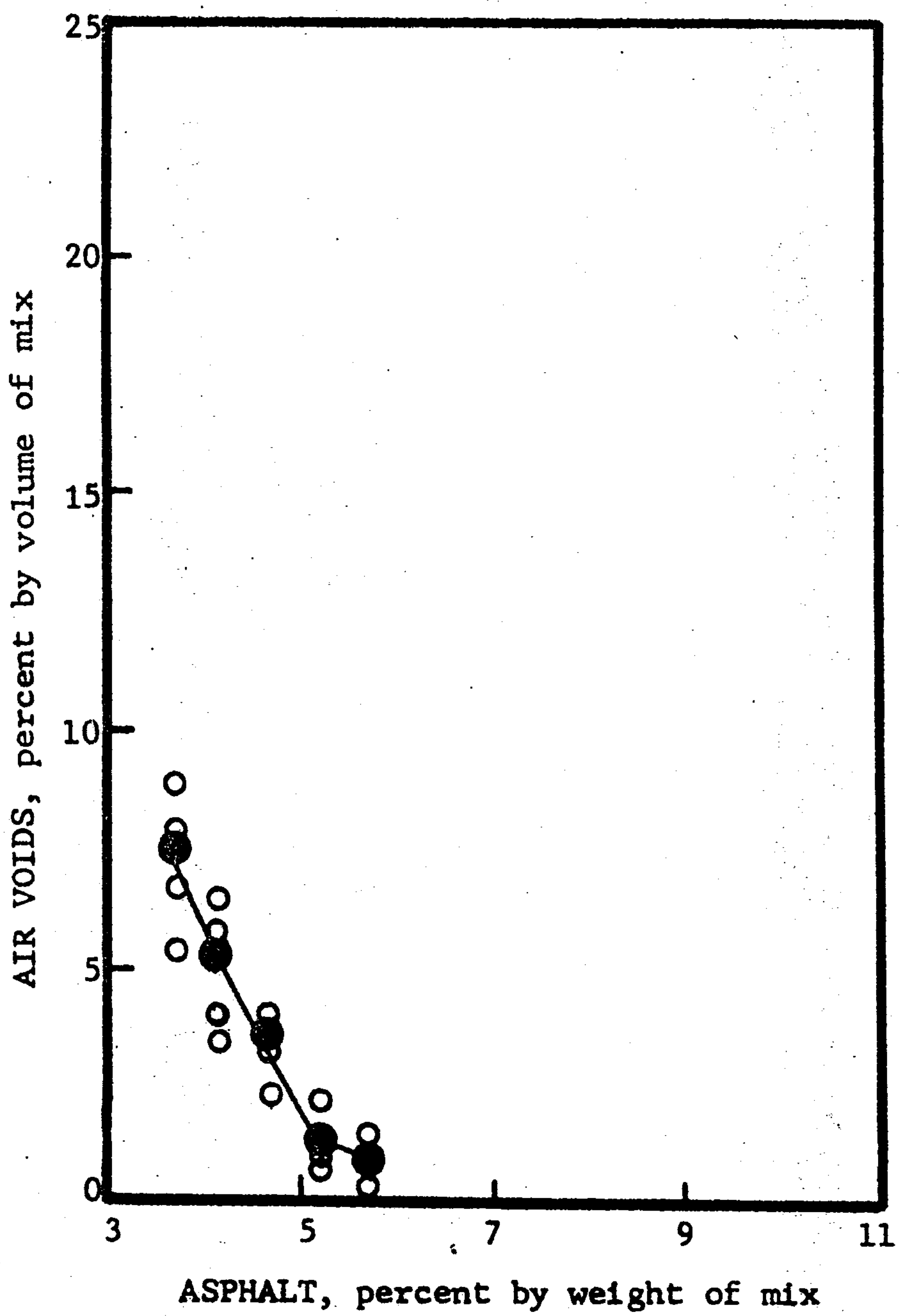

Figure 19. **Percent air voids in mix versus asphalt content for limestone mixtures.**

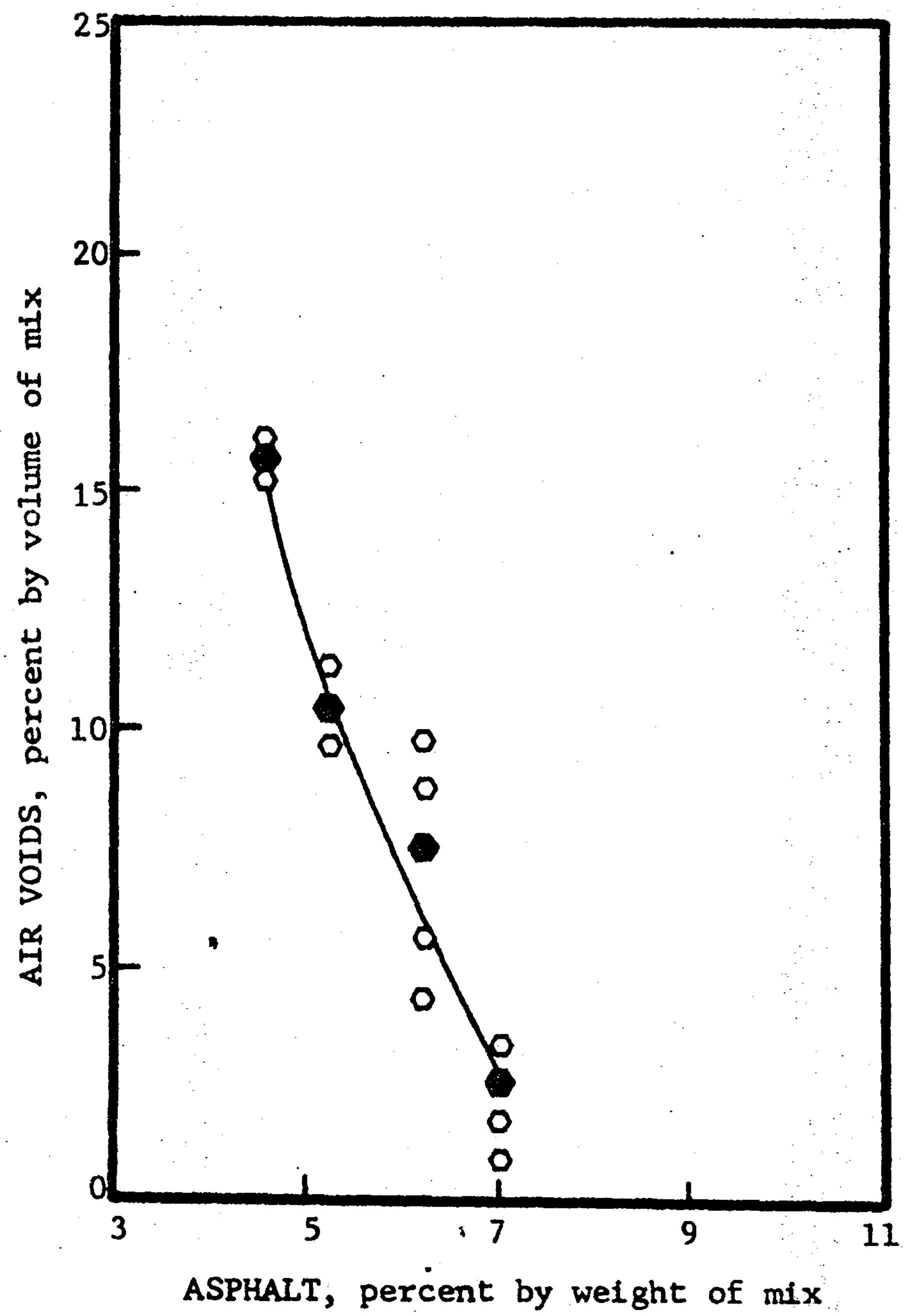

Figure 20. **Percent air voids in mix versus asphalt content for crushed caliche gravel mixtures.**

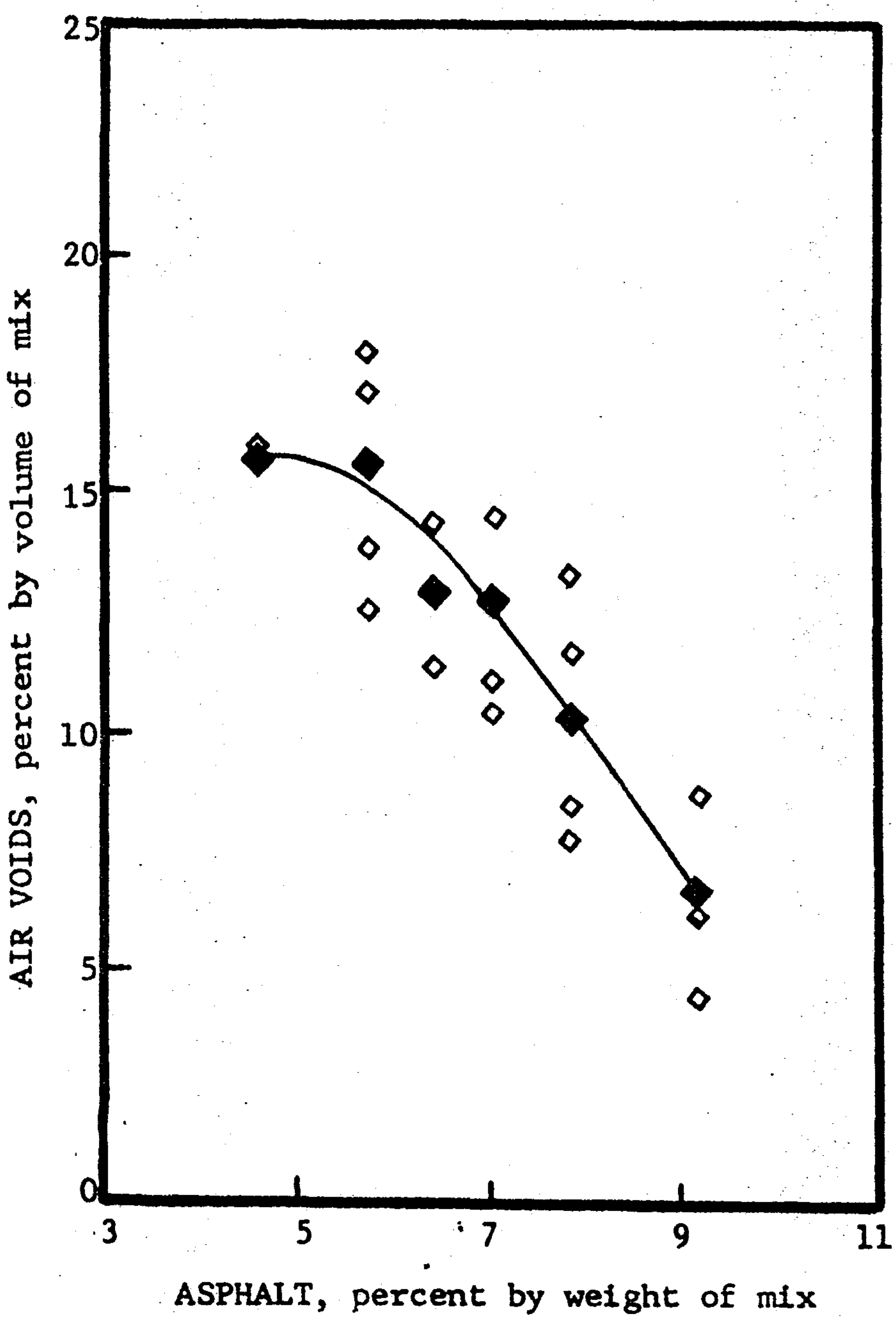

Figure 21. Percent air voids in mix versus asphalt content for crushed sandstone mixtures.

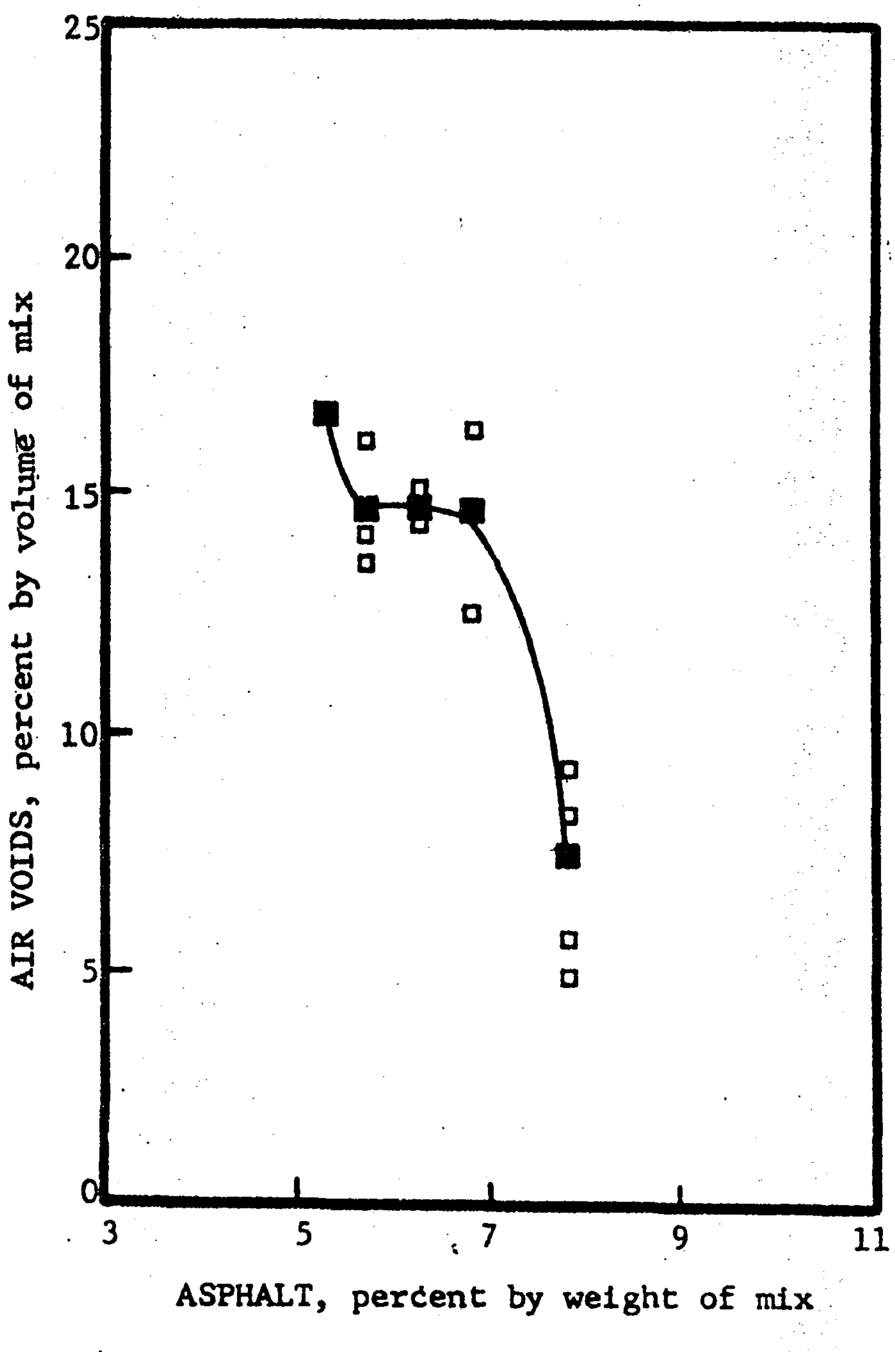

Figure 22. **Percent air voids in mix versus asphalt content for Austin chalk mixtures.**

previously all laboratory samples were subjected to the Schmidt test (27) after being sliced to 6 inches in diameter by 2 inches in height. The resulting resilient modulus values are plotted in Figures 23 to 26. The smaller open triangles (Δ) represent the individual data points at each asphalt content while the larger darkened triangles (Δ) represent the average value. Unless otherwise noted, this format is consistent throughout the paper. Resilient moduli, M_R, for these samples were obtained at 73°F. Figure 23 illustrates the resilient modulus of the 6-inch by 2-inch sandstone samples versus asphalt content while Figure 24 illustrates the resilient modulus values of the same samples after being cored to four inches in diameter by two inches in height. The sandstone samples were subjected to a vacuum saturation procedure for a period of two hours and a soaking period of seven days. After seven days and while still saturated, the samples were subjected to the Schmidt test; these values are plotted in Figure 25. Combining Figures 23, 24, and 25 Figure 26 illustrates the effect of coring and vacuum saturation on the sandstone mixtures. These data indicate that at asphalt contents less than 8.5 percent, the effect is quite severe. At 7.0 percent asphalt, for example, a loss of some 510,000 psi, or 85 percent, occurs when a comparison is made between extreme cases. Interestingly, it appears that at asphalt contents above 9.0 percent, the resilient modulus may increase with the intrusion of water.

Figures 27, 28, 29, and 30 indicate that the effect of coring and vacuum saturation on the laboratory molded limestone mixtures is minimal as measured by resilient modulus. Again, the resilient modulus at higher asphalt contents increased, somewhat, after vacuum saturation.

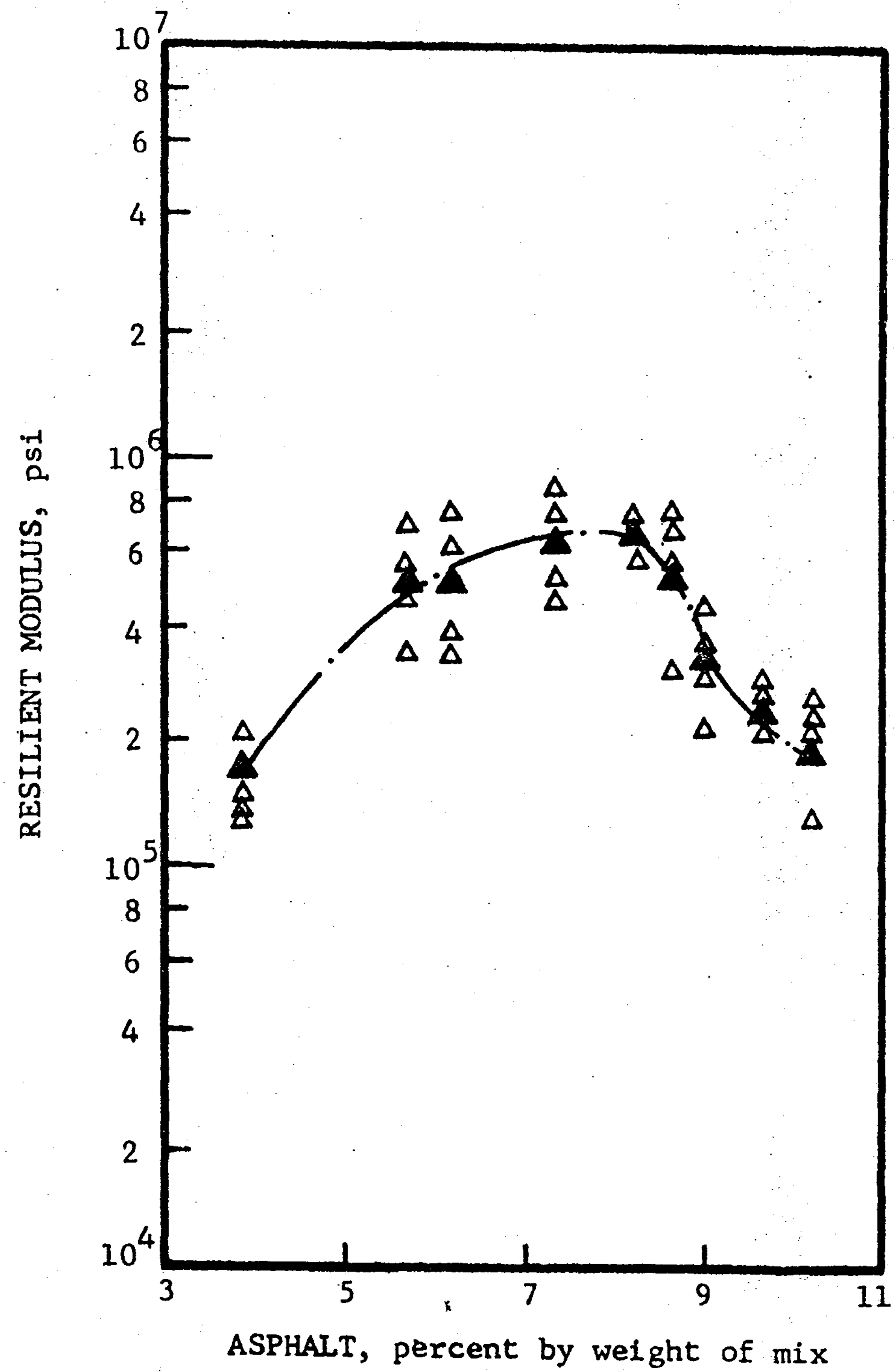

Figure 23. **Resilient modulus of sandstone mixtures at 73°F versus asphalt content of 6" ϕ x 2" ht. samples (before vacuum saturation).**

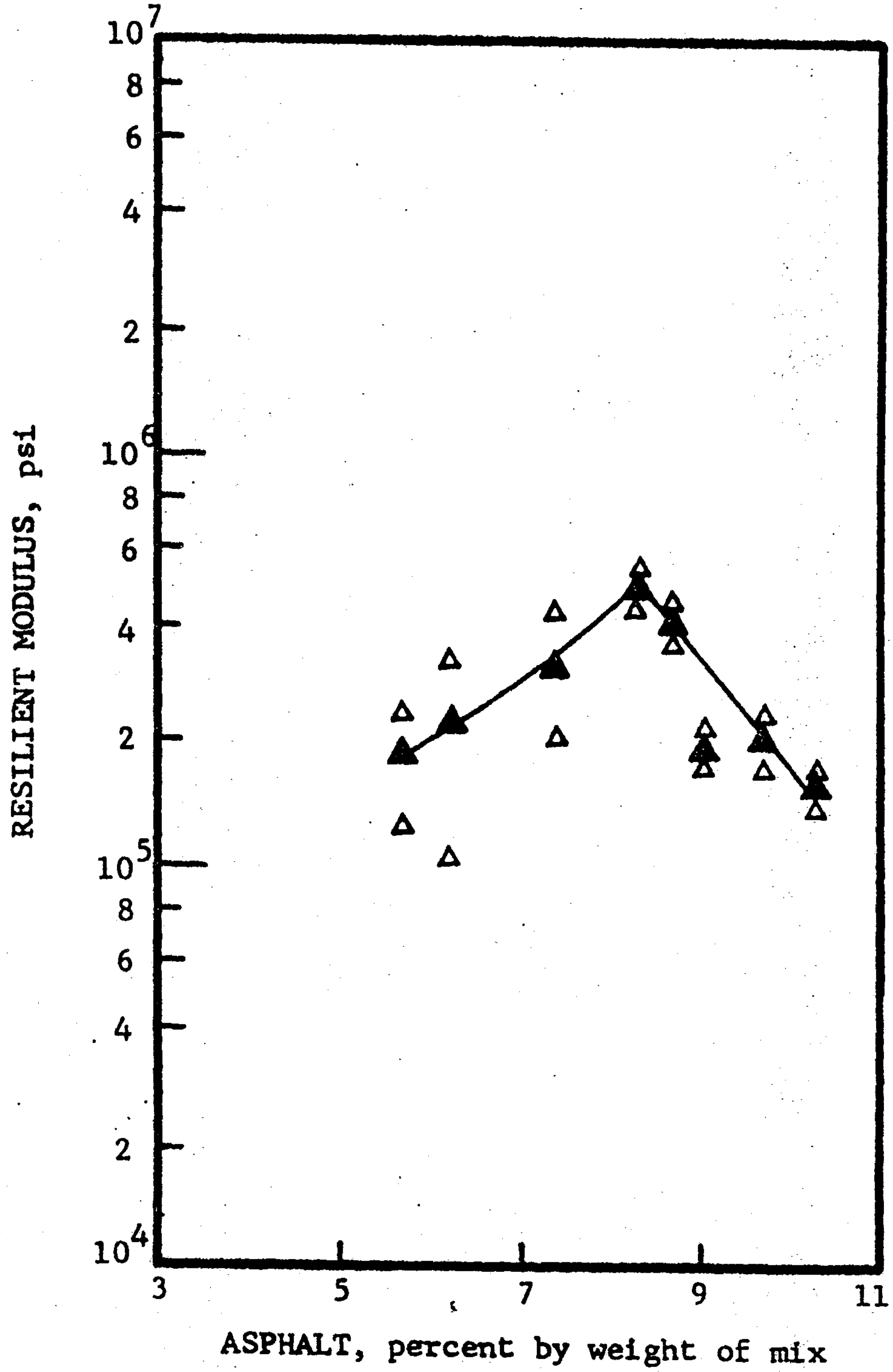

Figure 24. Resilient modulus of sandstone mixtures at 73°F versus asphalt content (before vacuum saturation).

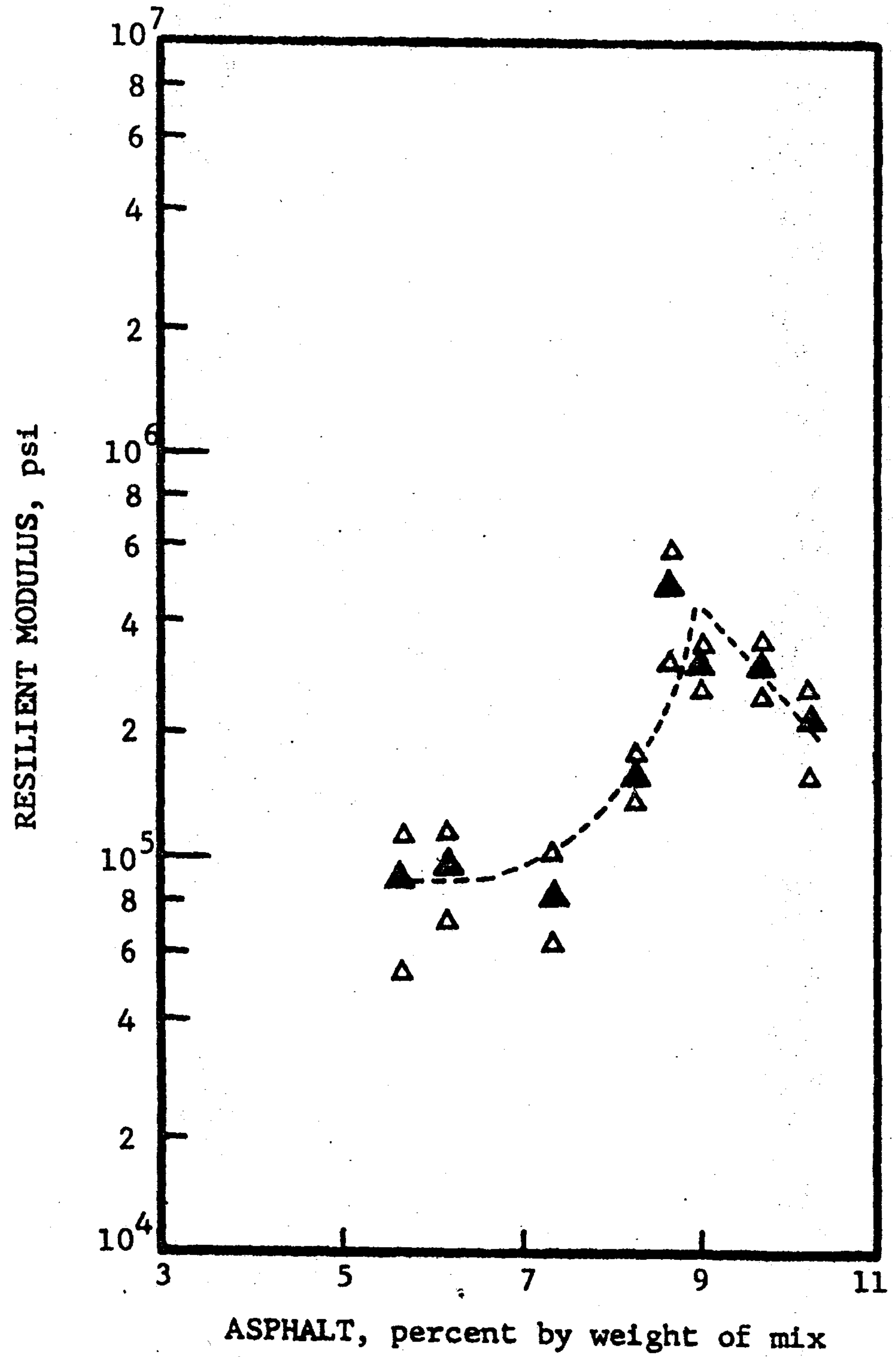

Figure 25. Resilient modulus of sandstone mixtures at 73°F versus asphalt content (after vacuum saturation).

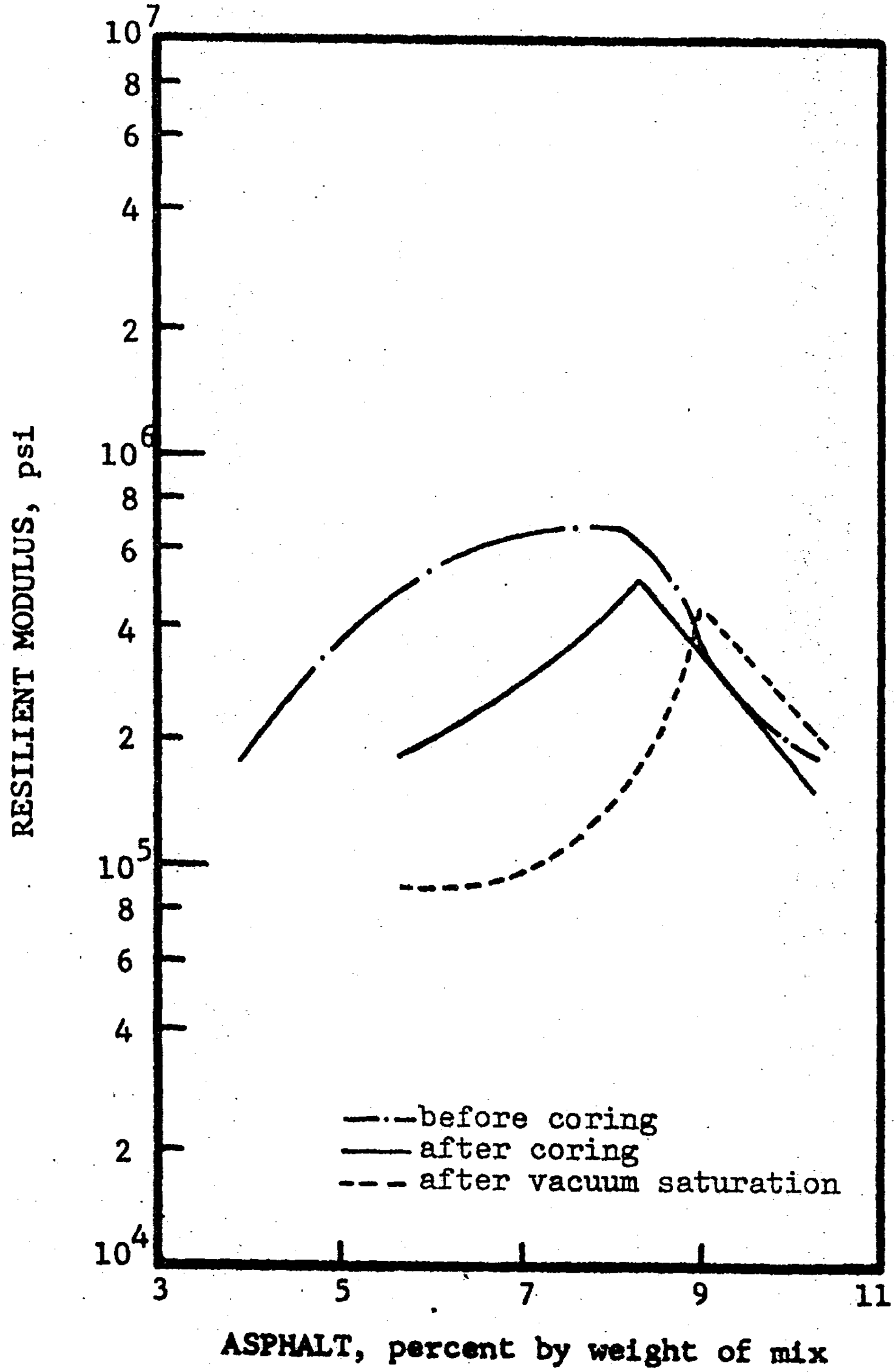

Figure 26. Effect of vacuum saturation and coring on resilent modulus of sandstone mixtures at 73°F versus asphalt content.

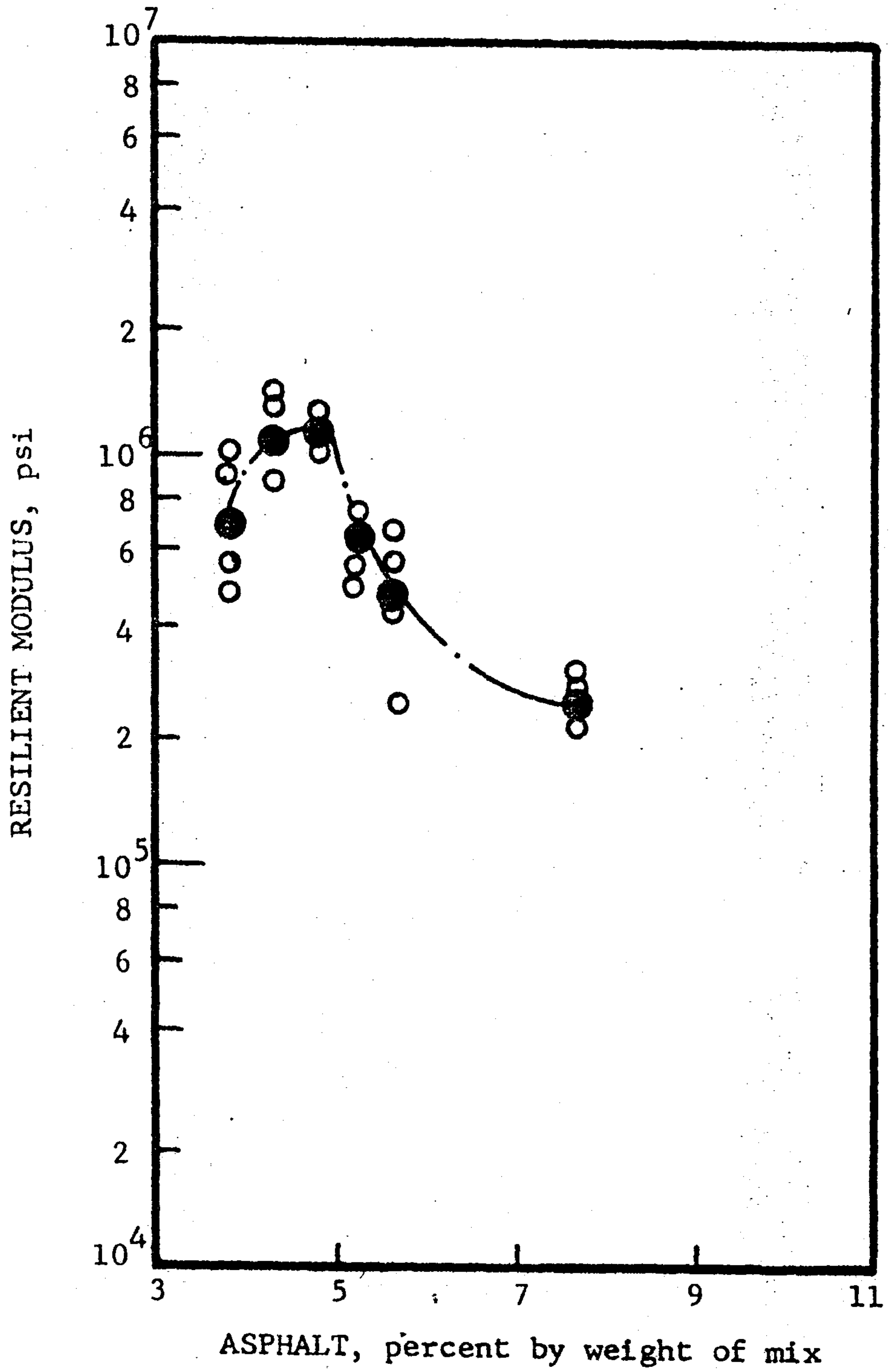

Figure 27. Resilient modulus of limestone mixtures at 73°F versus asphalt content of 6" ϕ x 2" ht. samples (before vacuum saturation).

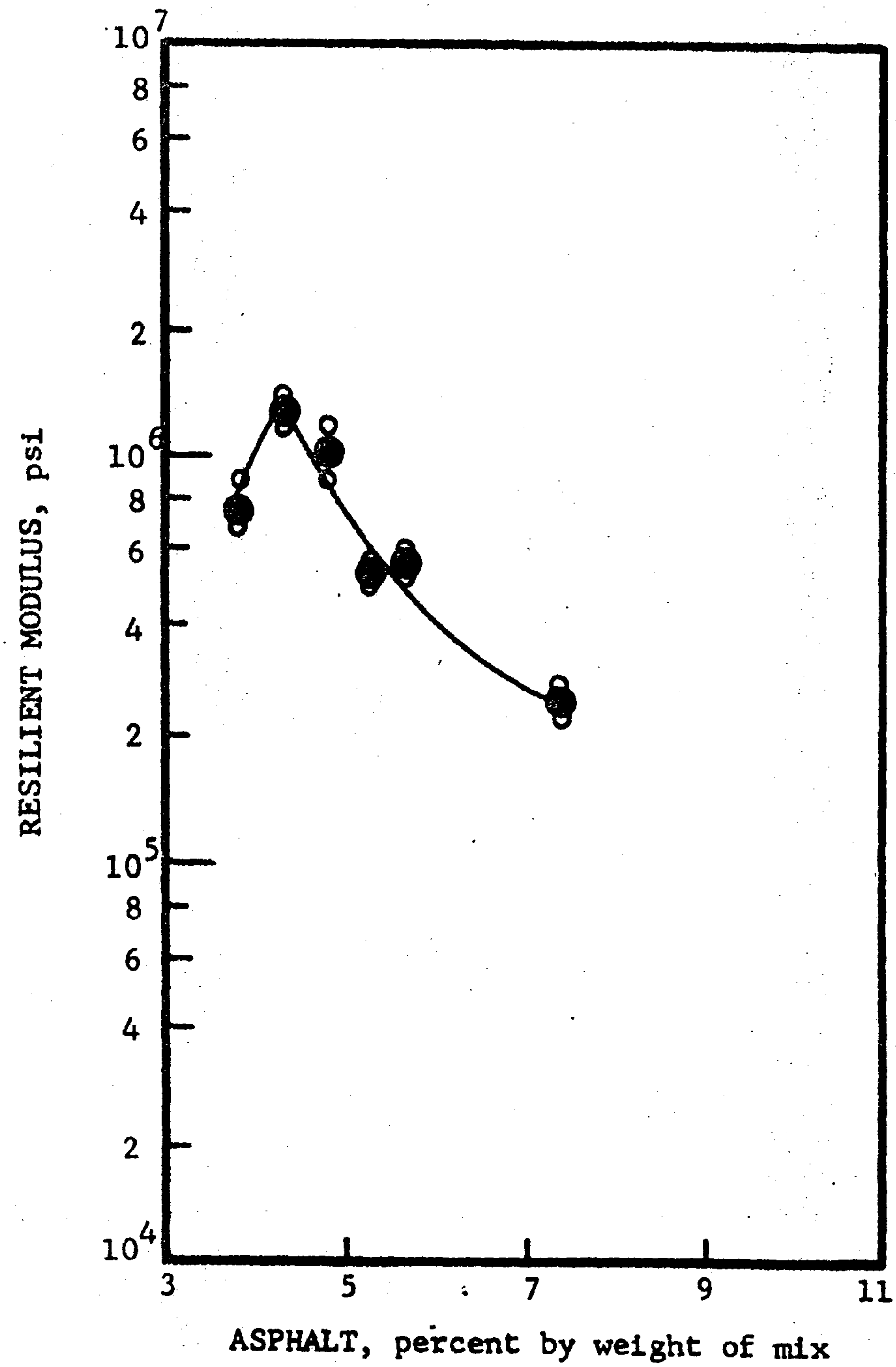

Figure 28. Resilient modulus of limestone mixtures at 73°F versus asphalt content (before vacuum saturation).

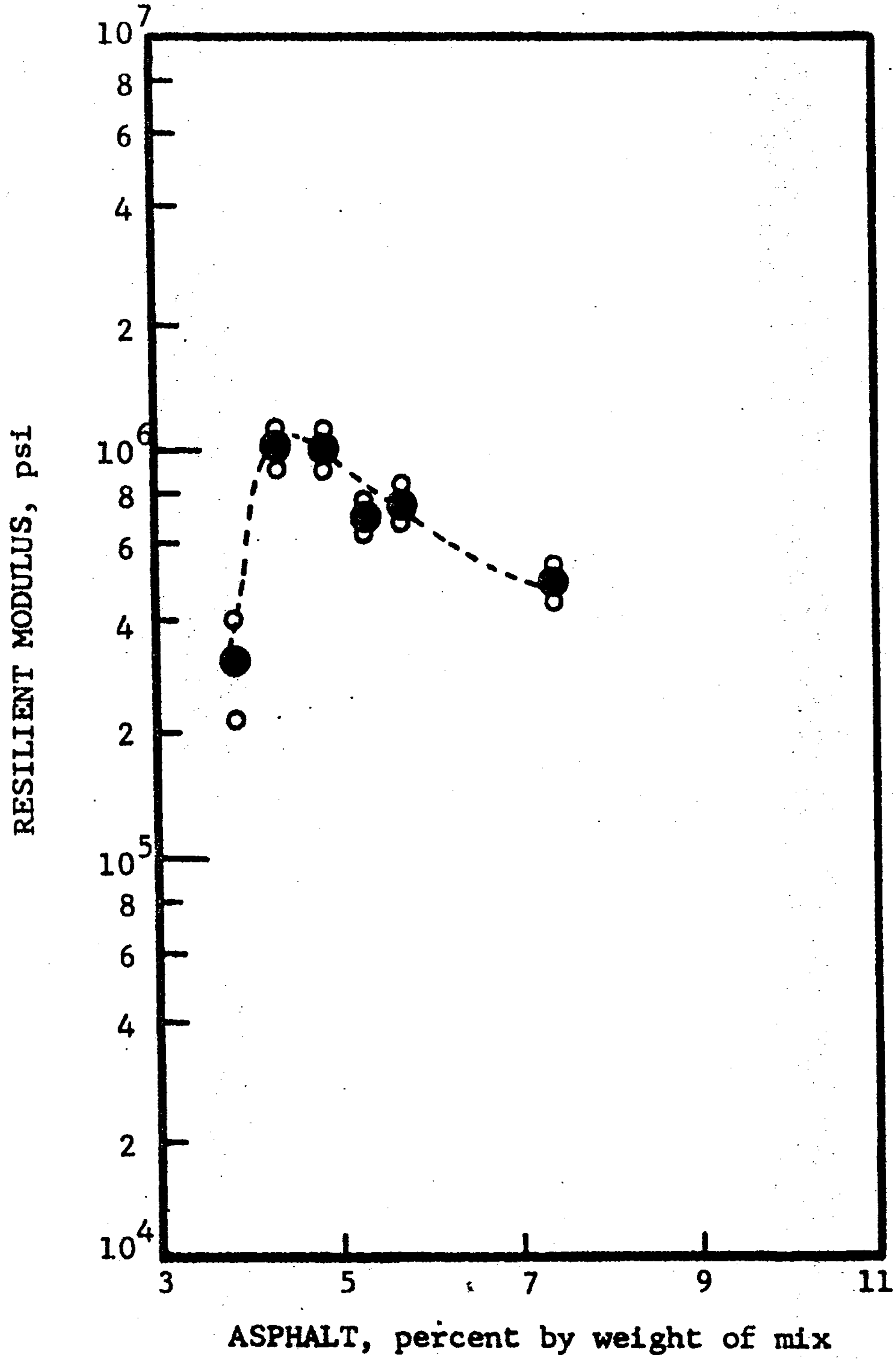

Figure 29. **Resilient modulus of limestone mixtures at 73°F versus asphalt content (after vacuum saturation).**

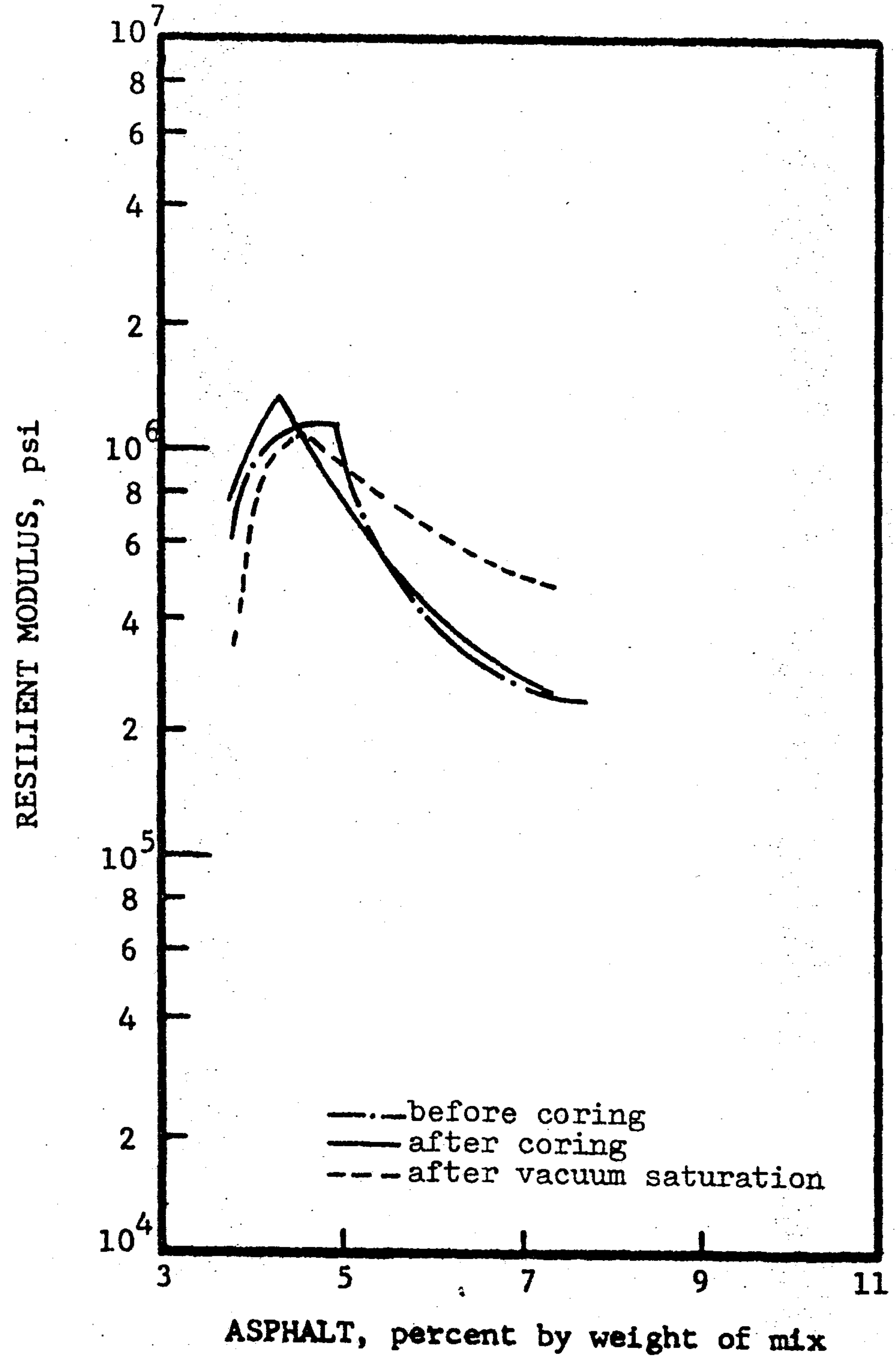

Figure 30. Effect of vacuum saturation and coring on resilient modulus of limestone mixtures at 73°F versus asphalt content.

Figures 31, 32, 33, and 34 illustrate the effect of coring and vacuum saturation on the crushed caliche gravel mixtures. While the coring operation had only a slight effect on the resilient moduli, vacuum saturation literally destroyed many of the samples. Two samples at 4.8 percent asphalt, one sample at 5.2 percent asphalt and one sample at 5.7 percent asphalt crumbled either during or immediately after vacuum saturation and were not suitable for further testing. From the crushed caliche gravel samples tested it is difficult to ascertain at what asphalt content the mixture will confidently withstand coring and vacuum saturation. It seems safe to assume, however, that this asphalt content is above 7.0 percent.

Figures 35, 36, 37, and 38 illustrate the effect of coring and vacuum saturation on the crushed sandstone mixtures. Figure 36 demonstrates a significant amount of data scatter after coring (Figure 35). Two samples at 4.8 percent asphalt and two samples at 5.7 percent asphalt failed prior to being tested in the saturated condition. Also, the crushed sandstone samples (6-inch diameter by 2-inch high) at 9.1 percent (CSS-6) were not tested for resilient modulus. Reviewing Figure 38 it appears that at asphalt contents approaching 9.0 percent, the effect of water on the crushed sandstone samples becomes less severe.

Figures 39, 40, and 41 illustrate the effect of coring and vacuum saturation on the laboratory molded Austin chalk mixtures. As with crushed sandstone, the coring operation resulted in an increase of data scatter. Prior to coring, the chalk mixtures appeared to possess promising stiffness. After coring, and especially after vacuum saturation, however, it became obvious that the intrusion of water damaged the samples

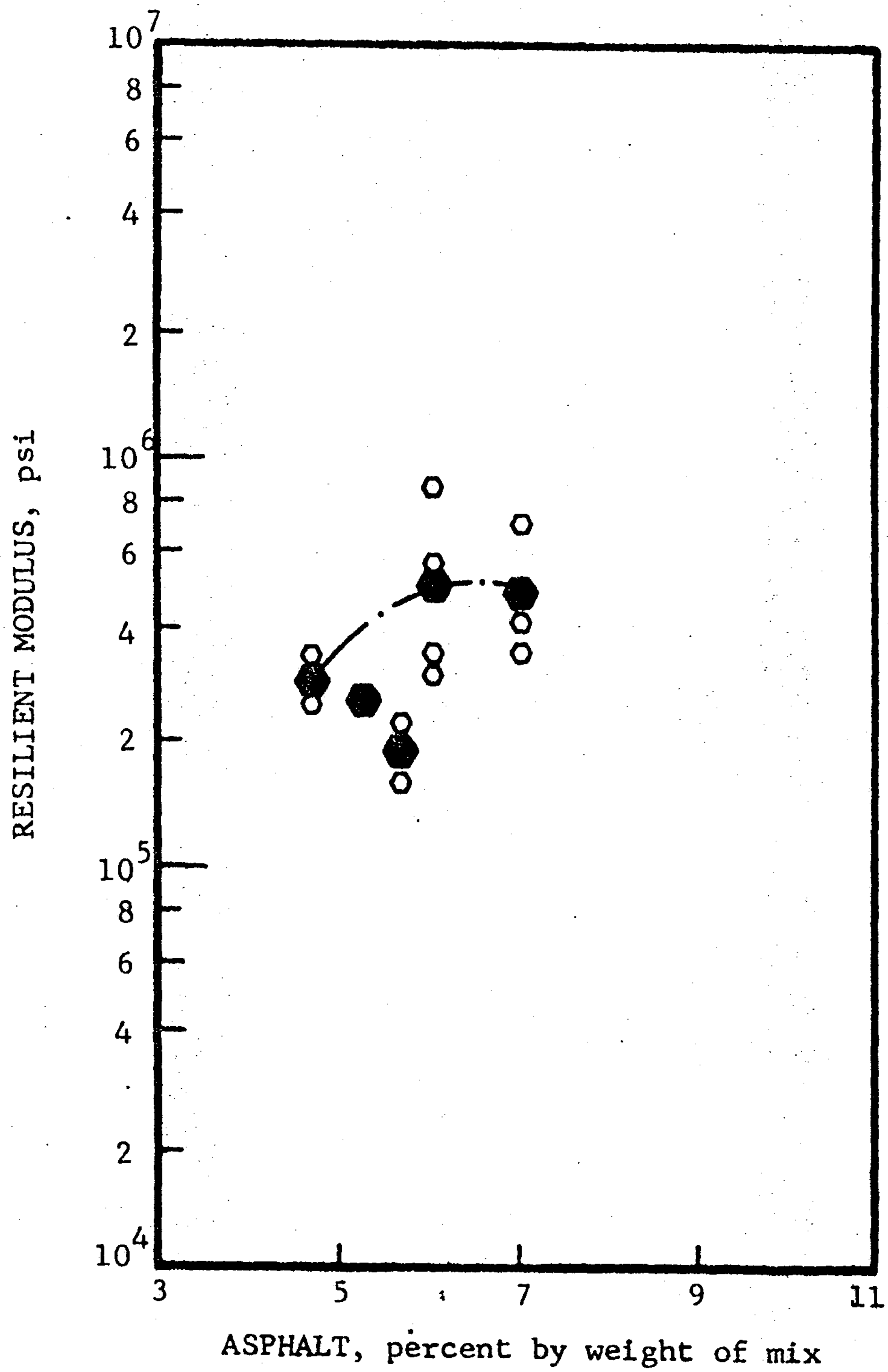

Figure 31. Resilient modulus of crushed caliche gravel mixtures at 73°F versus asphalt content of 6" ϕ x 2" ht. samples (before vacuum saturation).

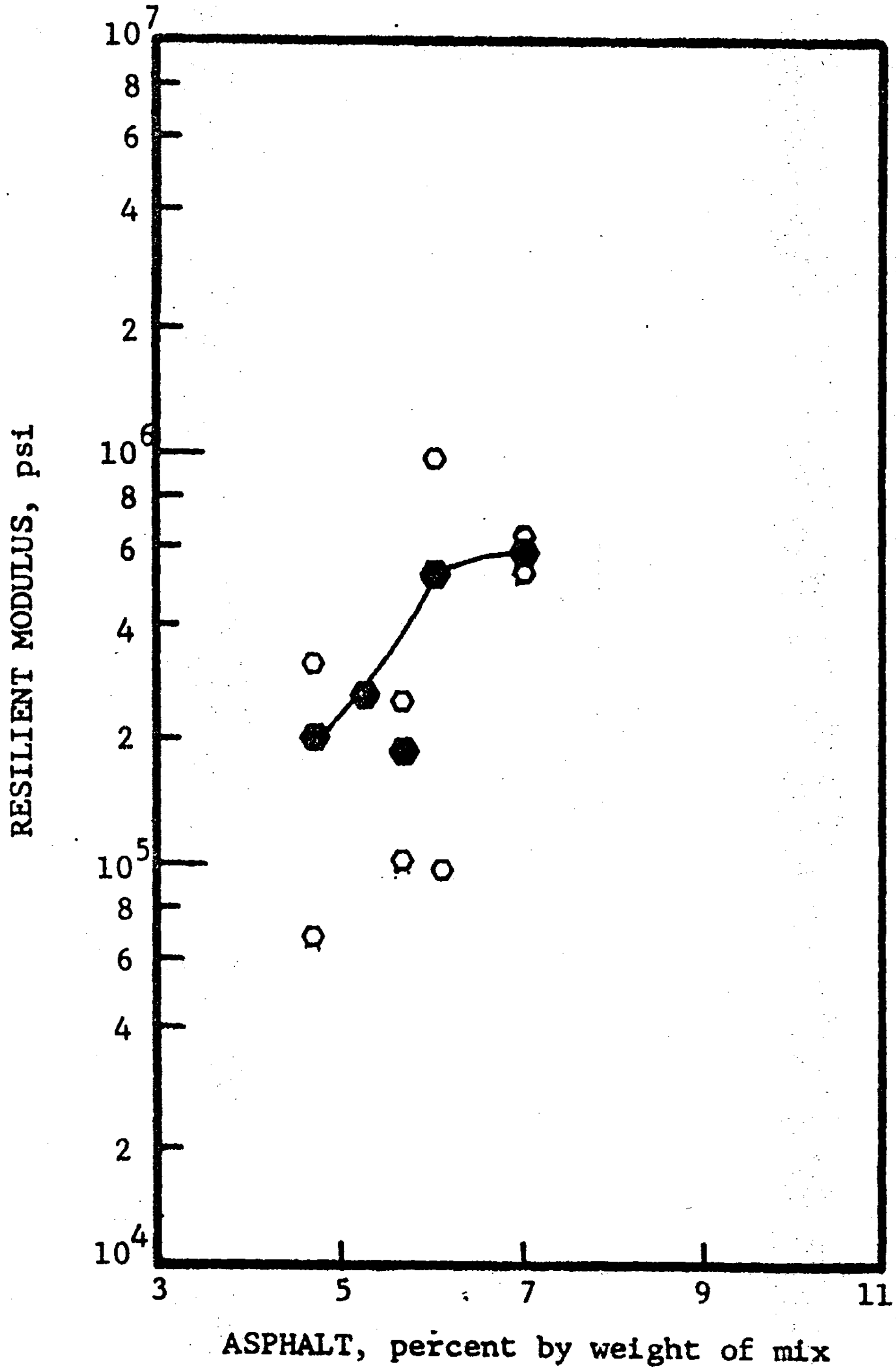

Figure 32. Resilient modulus of crushed caliche gravel mixtures at 73°F versus asphalt content (before vacuum saturation).

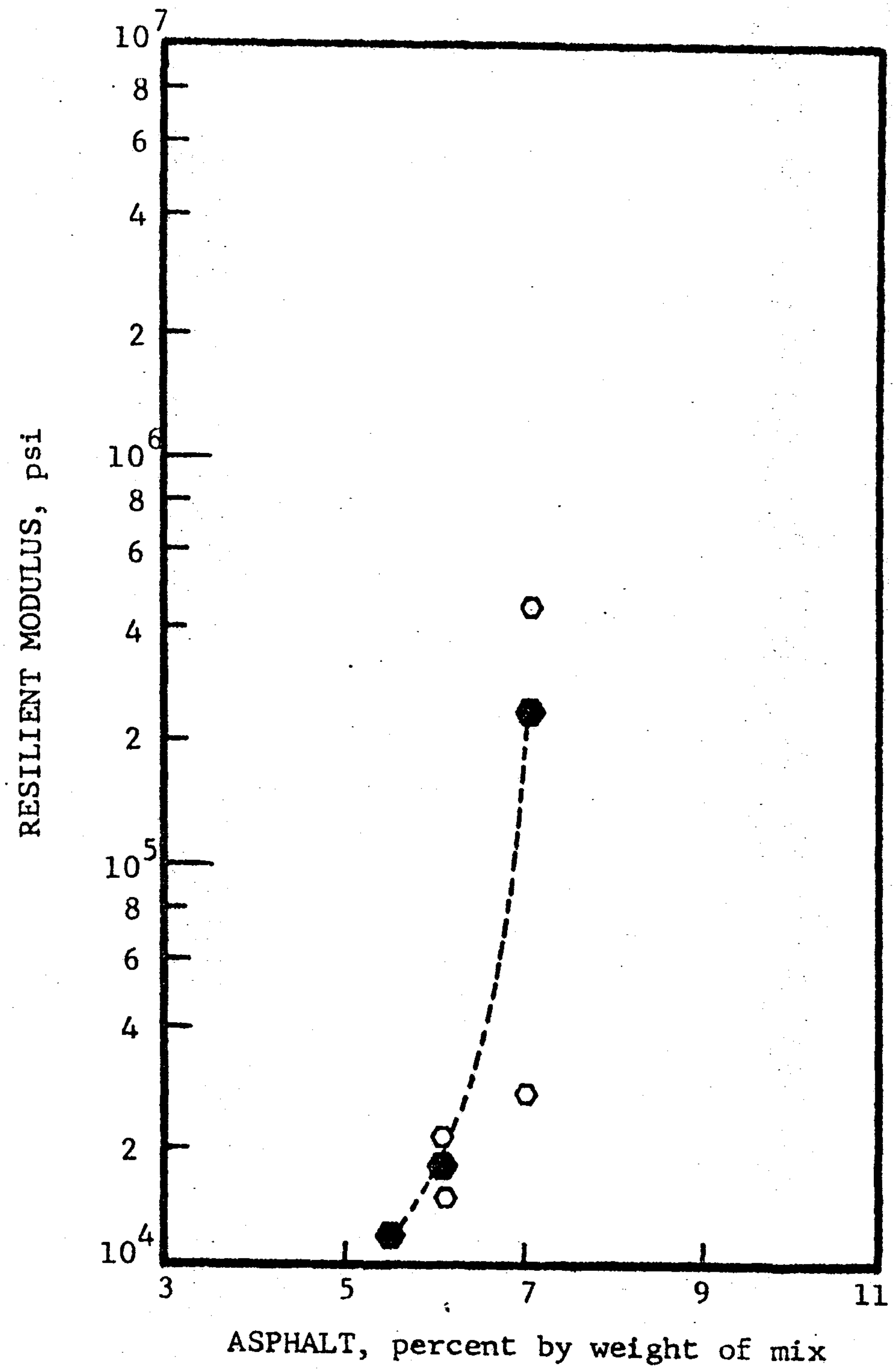

Figure 33. Resilient modulus of crushed caliche gravel mixtures at 73°F versus asphalt content (after vacuum saturation).

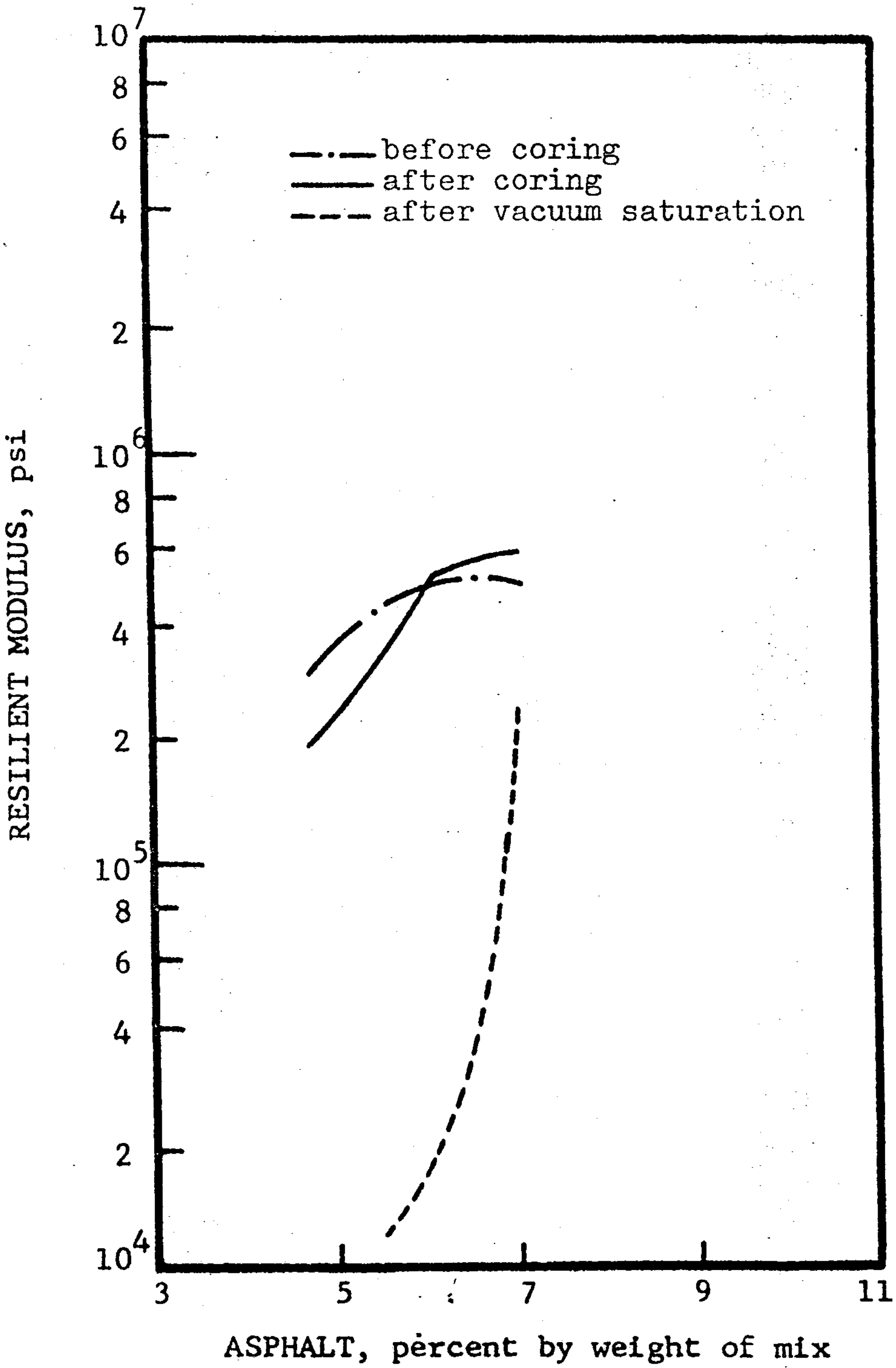

Figure 34. Effect of vacuum saturation and coring on resilient modulus of crushed caliche gravel mixtures at 73°F versus asphalt content.

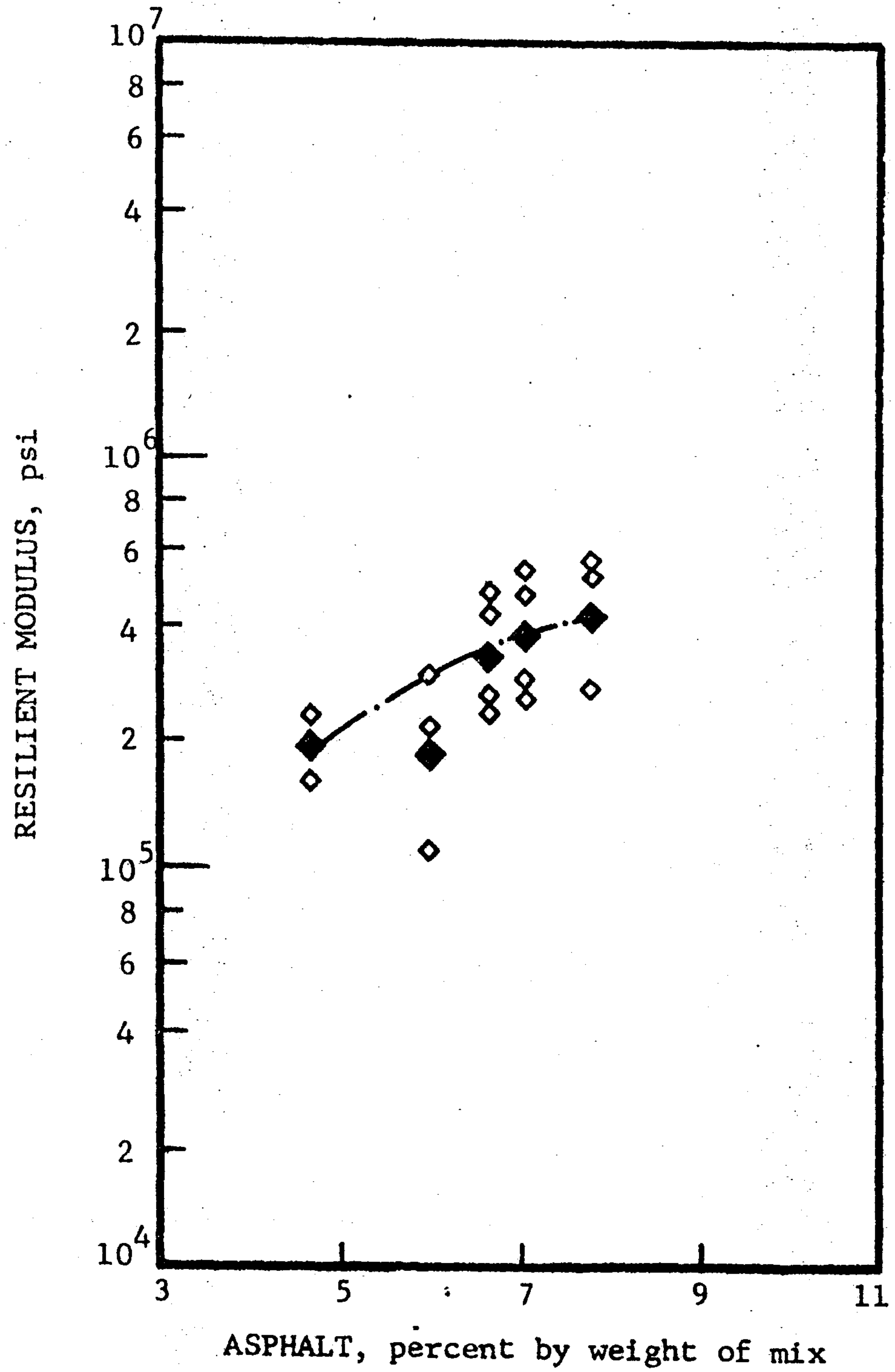

Figure 35. **Resilient modulus of crushed sandstone mixtures at 73°F versus asphalt content of 6" ϕ x 2" ht. samples (before vacuum saturation).**

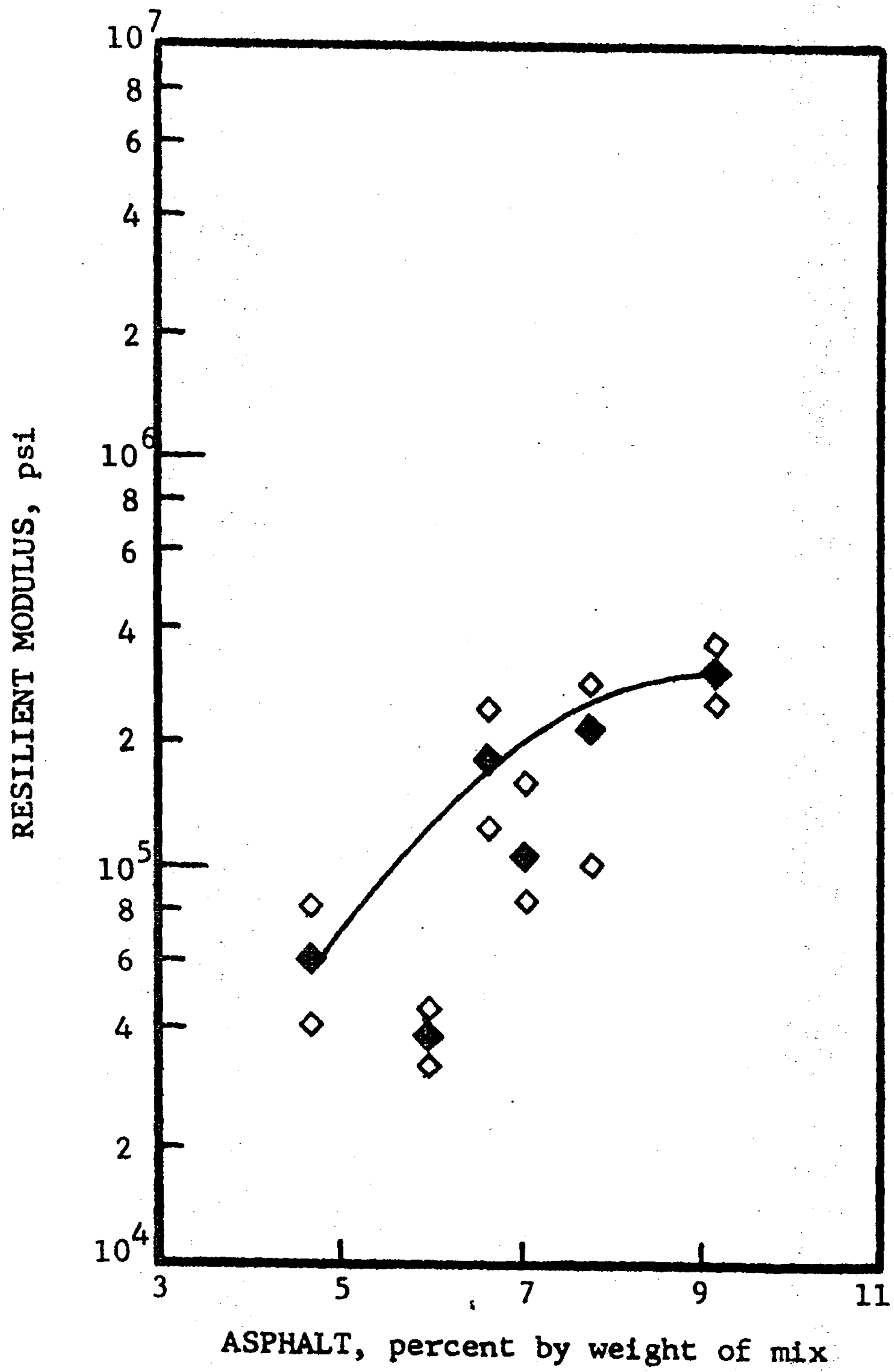

Figure 36. Resilient modulus of crushed sandstone mixtures at 73°F versus asphalt content (before vacuum saturation).

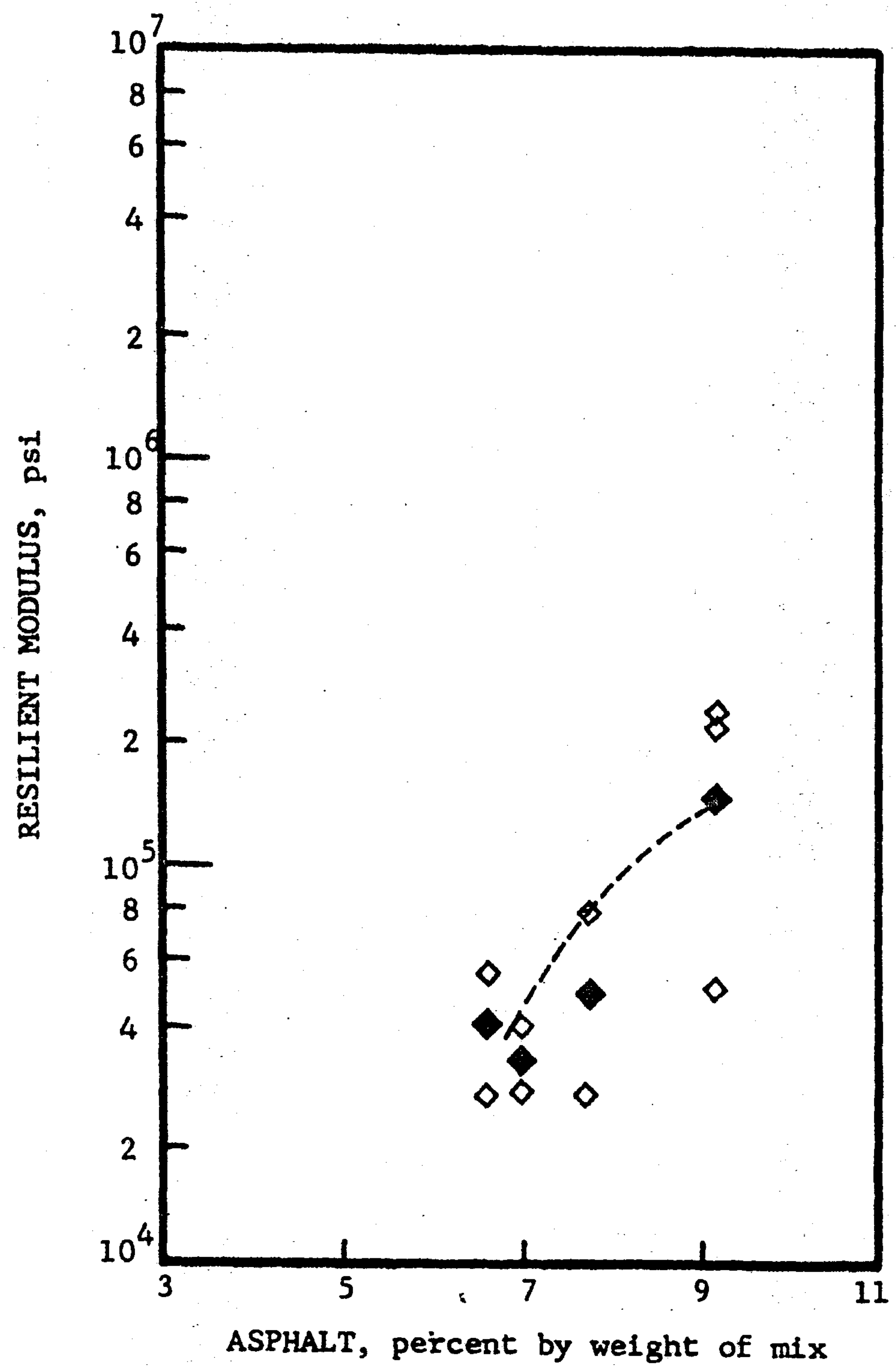

Figure 37. Resilient modulus of crushed sandstone mixtures at 73°F versus asphalt content (after vacuum saturation).

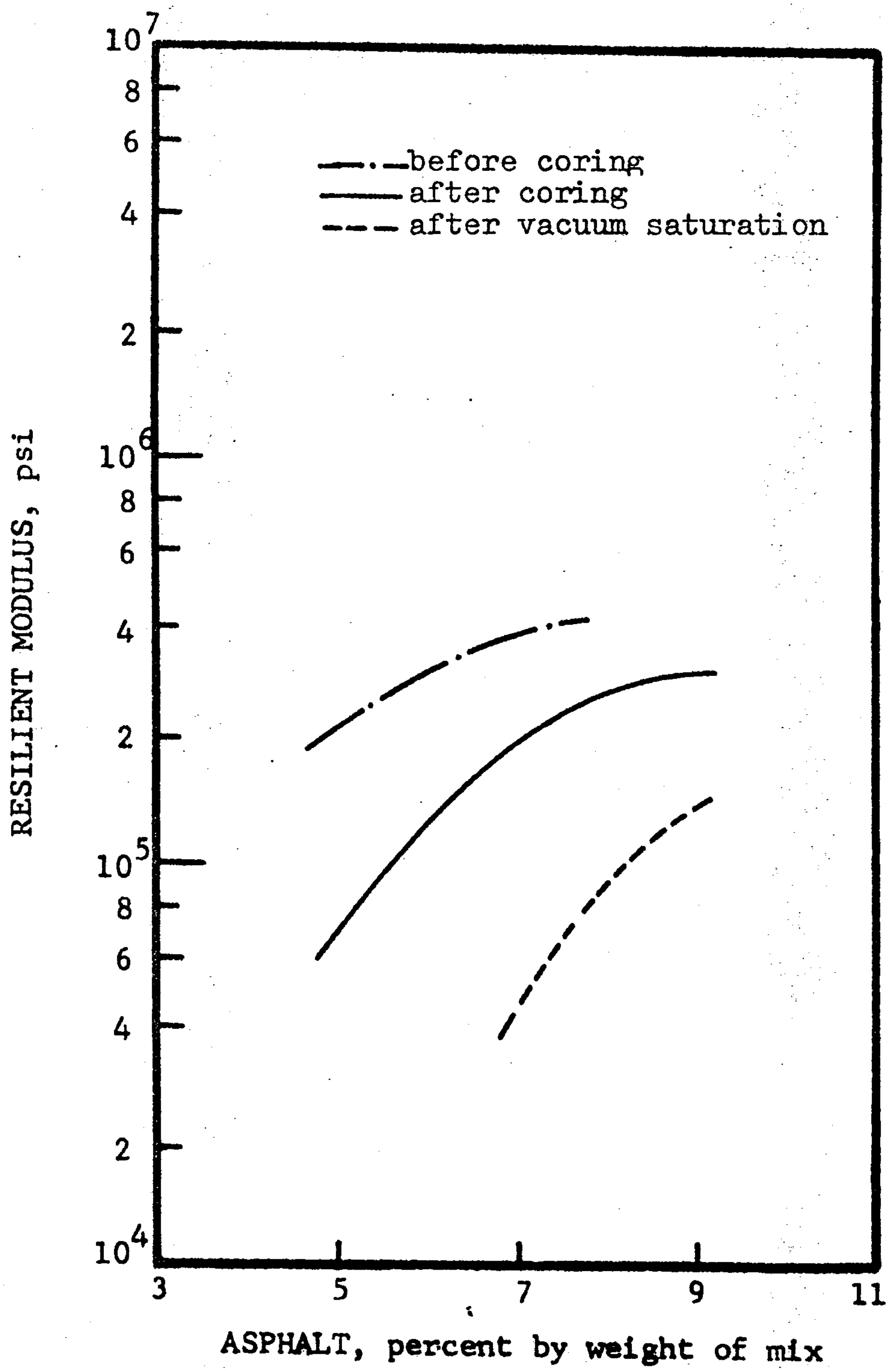

Figure 38. Effect of vacuum saturation and coring on resilient modulus of crushed sandstone mixtures at 73°F versus asphalt content.

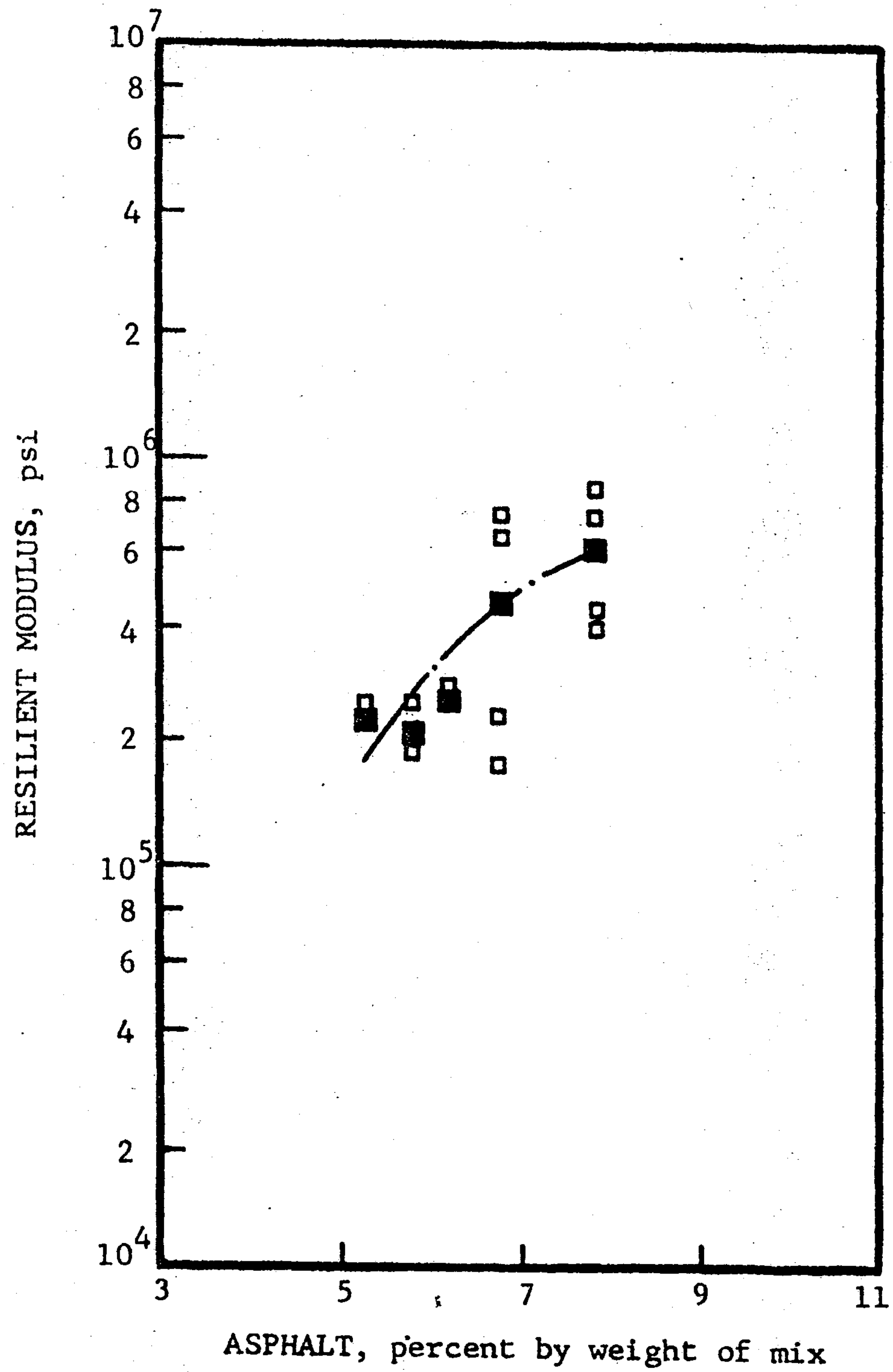

Figure 39. **Resilient modulus of Austin chalk mixtures at 73°F versus asphalt content of 6" ϕ x 2" ht. samples (before vacuum saturation).**

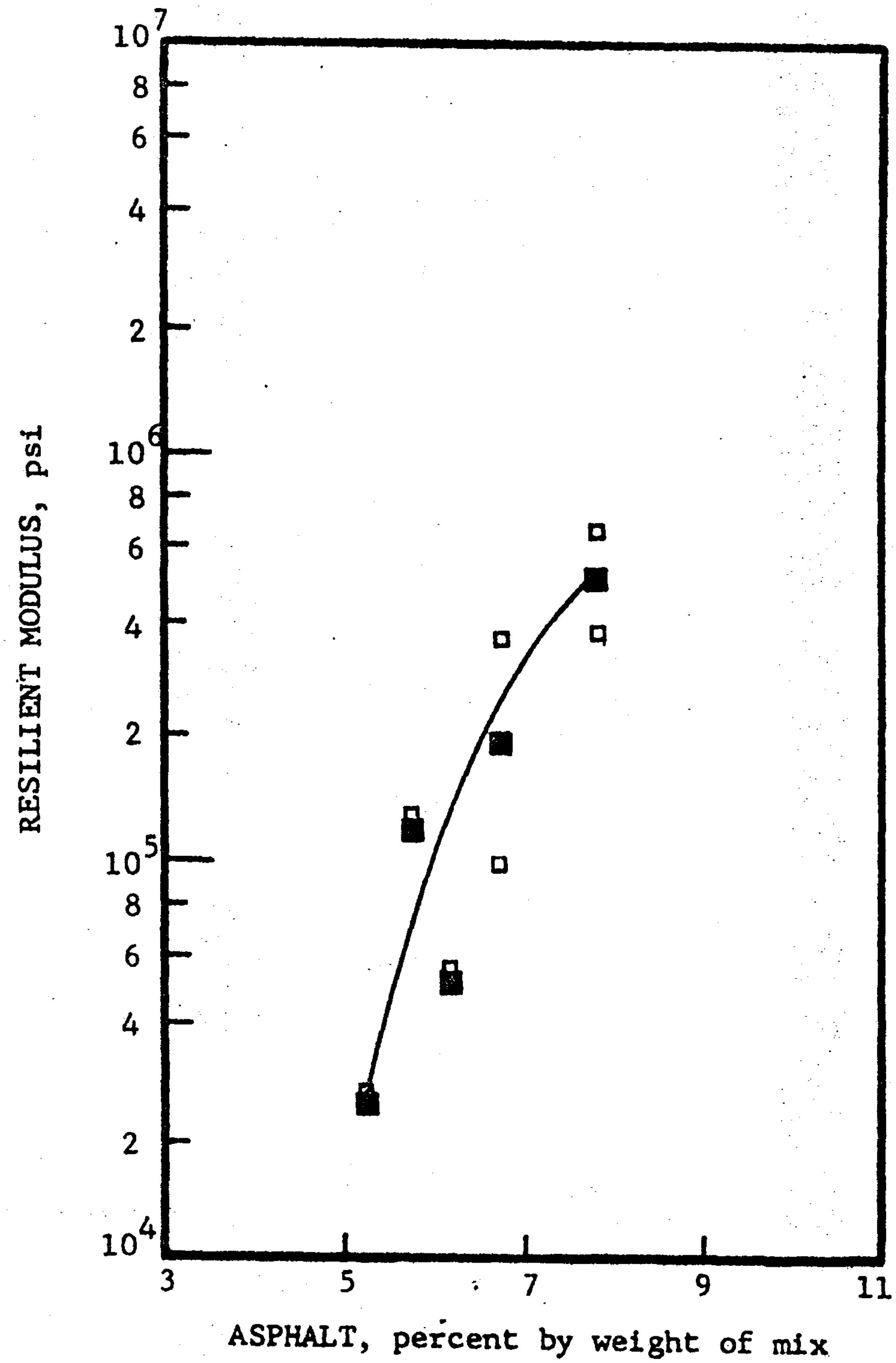

Figure 40. Resilient modulus of Austin chalk mixtures at 73°F versus asphalt content (before vacuum saturation).

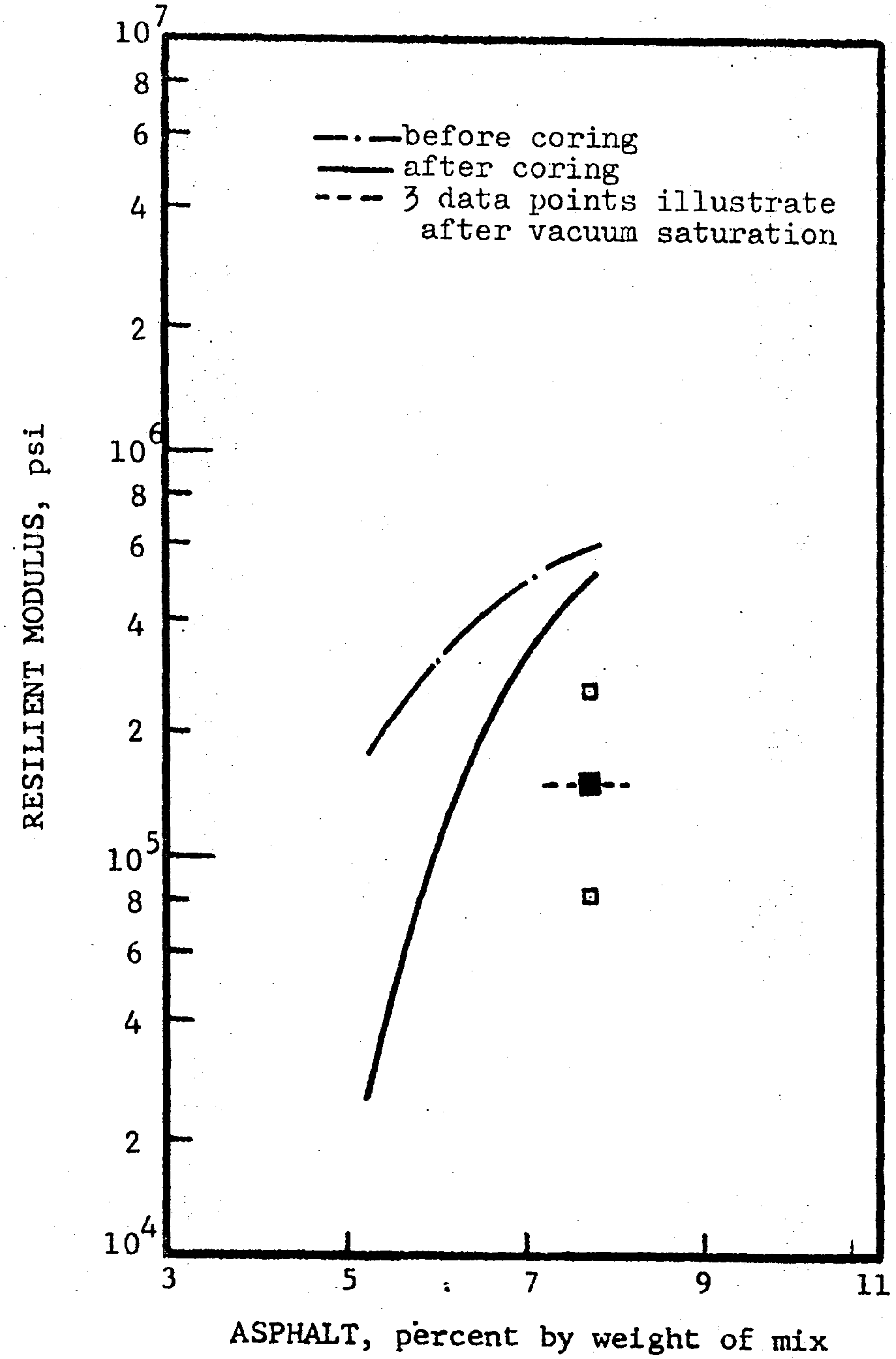

Figure 41. Effect of vacuum saturation and coring on resilient modulus of Austin chalk mixtures at 73°F versus asphalt content.

substantially. Of the forty Austin chalk samples molded, only two samples were suitable for testing after vacuum saturation; these two samples contained 7.8 percent asphalt and data are plotted in Figure 41. Figure 40, in particular, illustrates the fact that Austin chalk samples were not molded at a high enough asphalt content to reach a peak in the resilient modulus versus asphalt content curve. Of the five aggregates investigated, the Austin chalk mixtures were the most severely damaged by coring and vacuum saturation.

Although several investigations on the effect of moist environments on asphalt concrete pavements have been conducted (40, 41, 42), the work by Schmidt (43) is probably the most pertinent to this study. Schmidt investigated the effect of water on the resilient modulus of several asphalt mixtures. The aggregates investigated include gravel, granite, limestone, calcite and silica. As three aggregate gradations were investigated, one gradation in particular was similar to that used in the marginal aggregate bituminous base study. After measuring the resilient modulus dry, the specimens were tested to obtain the resilient modulus values at 73°F and 140°F and were returned to the water bath for soaking.

Schmidt concluded first that the concentration of water present in the specimens is proportional to the rate and extent of M_R drop. Secondly, the drop in resilient modulus due to the presence of water is less at higher asphalt contents. Thirdly, at very low asphalt contents the decrease becomes severe and at a higher asphalt content the mixes are almost completely water resistant. Interestingly, Schmidt found that these reductions in M_R are reversible; that is, the resilient modului of vacuum saturated specimens return to their original value after drying.

The results reported by Schmidt are quite consistent with those previously presented for the marginal aggregate bituminous base mixtures. Unfortunately, the resilient modulus of the marginal mixtures was not obtained for the dry specimens after vacuum saturation. Thus, it becomes impossible to determine whether or not the resilient moduli of the saturated specimens would have return to their original value after drying. Schmidt reported no samples being failed by the vacuum saturation procedure. By examining the two vacuum saturation procedures employed, it appears that the procedure used to saturate the marginal materials was significantly more severe than that used by Schmidt.

Effect of Temperature. Investigating the effect of temperature on bituminous mixtures may be approached several ways. One method involves the use of asphalt properties such as viscosities, penetration, ring and ball softening points, etc., to predict stiffness over a range of temperatures(25). A second approach and the method utilized in the investigation is by use of the Schmidt device where resilient modulus of the bituminous mixtures is measured at various temperatures.

The marginal aggregate, black base mixtures were tested at 34°F, 73°F, and 100°F by use of the Resilient Modulus Apparatus. At 34°F, many samples experienced deformations of less than 20 micro-inches and due to the fact that below 20 micro-inches the resilient modulus results become questionable, the samples were not tested at a lower temperature.

The effect of temperature on the resilient modulus of the sandstone mixtures is illustrated in Figures 42, 43, and 44. At 34°F it appears that an "optimum" asphalt content, or peak in the M_R versus asphalt content curve, occurs at about ten percent. At 73°F and 100°F the peak occurs

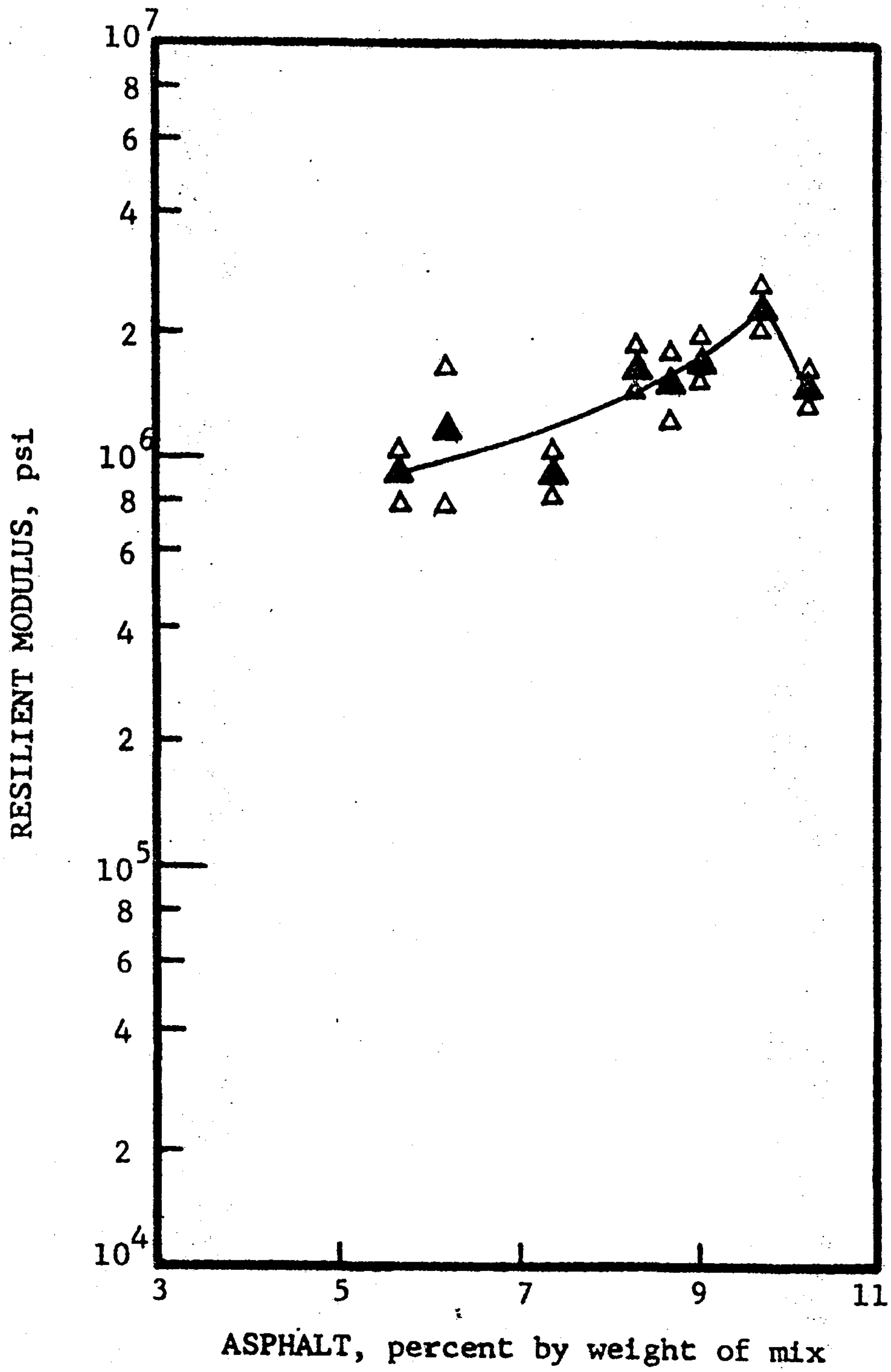

Figure 42. Resilient modulus of sandstone mixtures at 34°F versus asphalt content.

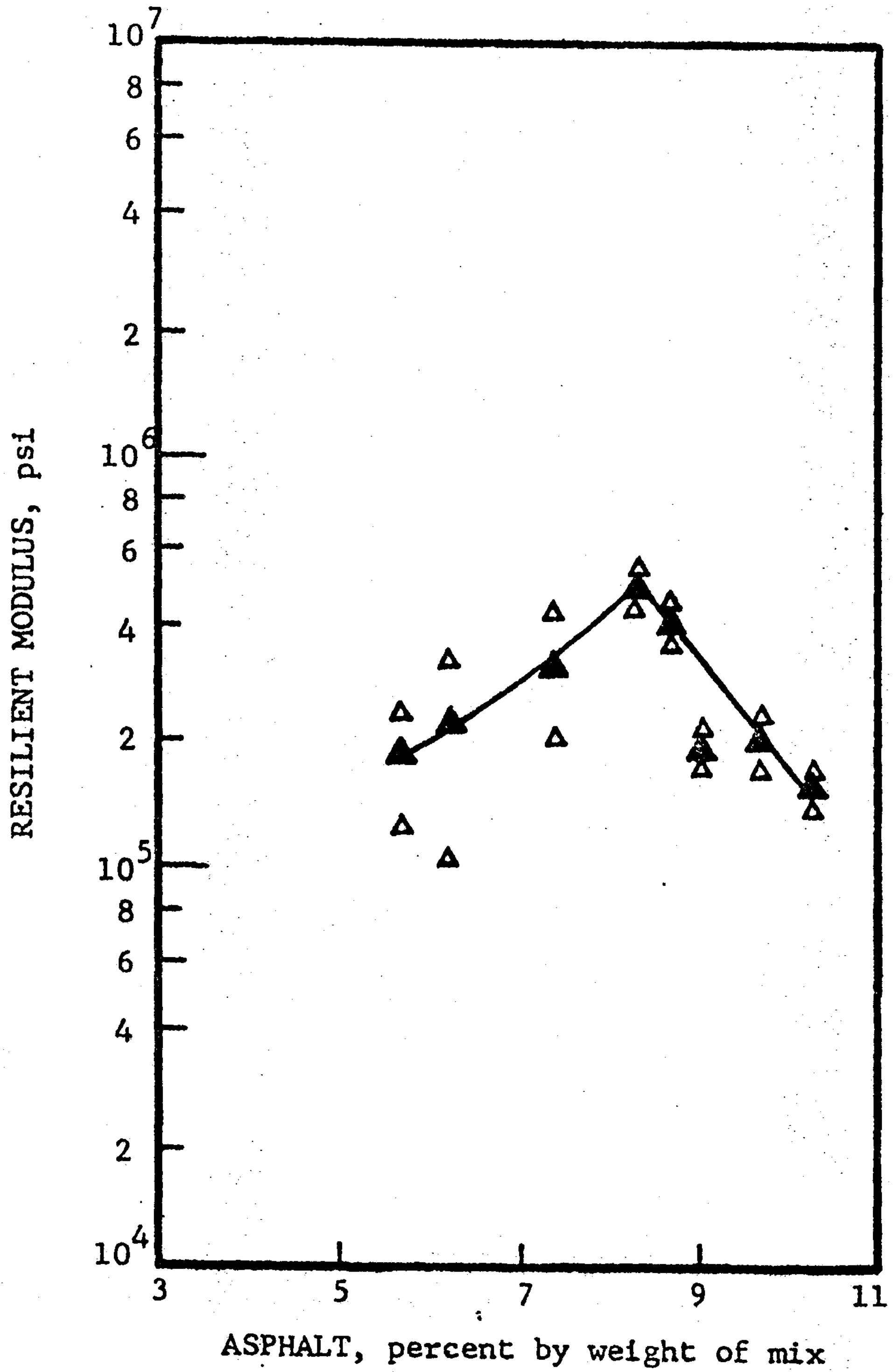

Figure 43. Resilient modulus of sandstone mixtures at 73°F versus asphalt content (before vacuum saturation).

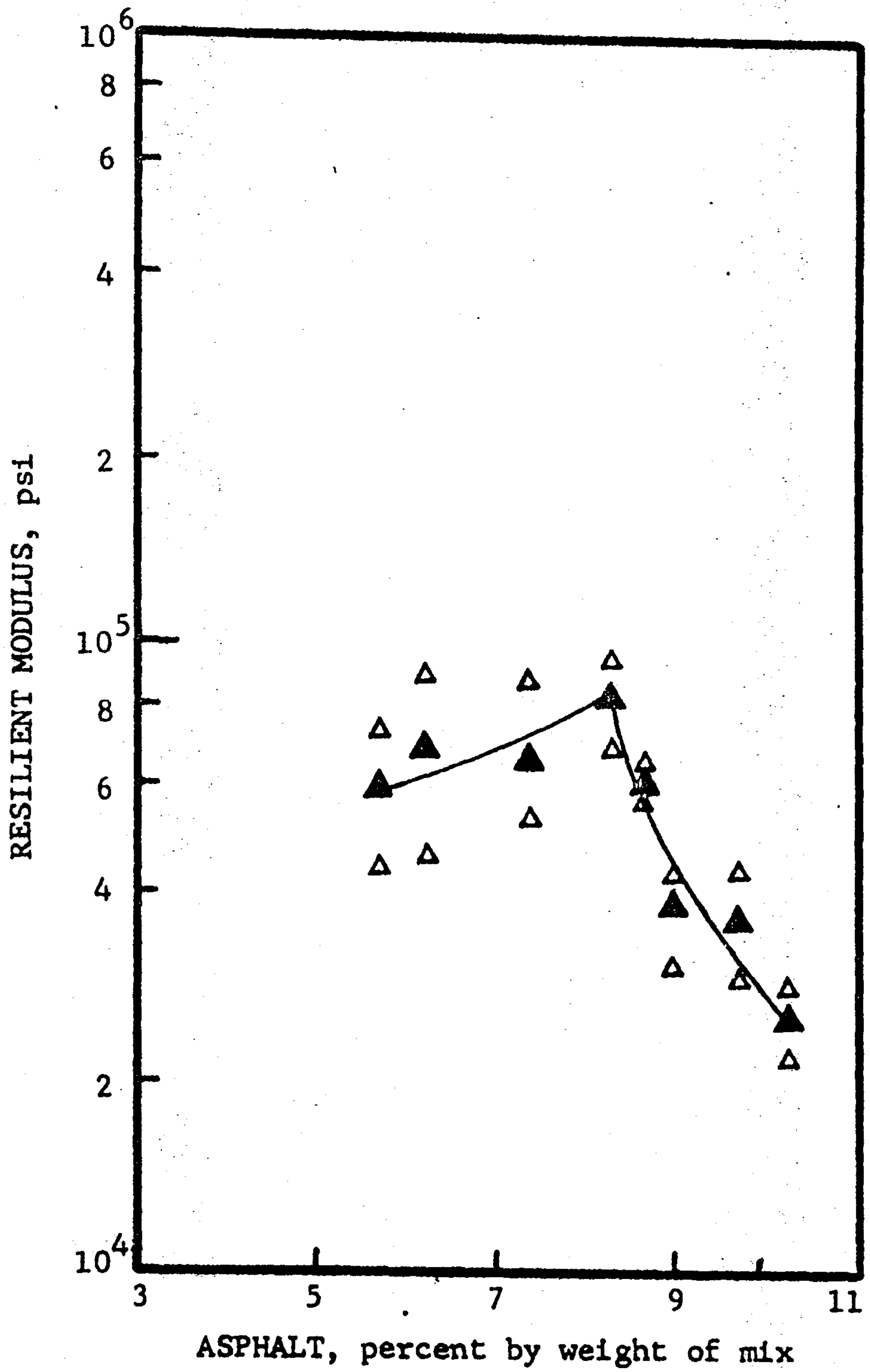

Figure 44. Resilient modulus of sandstone mixtures at 100°F versus asphalt content.

near eight percent asphalt. This can be explained by the relationship between the stiffness of the asphalt and the stiffness of the mix. At lower temperatures, the asphalt viscosity is increased and as a result the mixture stiffness, or resilient modulus, in increased. Figure 45 illustrates the temperature susceptibility of the sandstone mixtures. The temperature susceptibility of the mix could be defined as the slope of the M_R versus asphalt content curve. As the slope of the curve is increased (the curve becomes steeper), the drop in M_R per degree Farenheit is increased and thus the mix is more drastically affected by temperature. Figure 45 demonstrates that the asphalt content corresponding to the highest M_R values is near eight percent asphalt.

Figures 46, 47, 48, and 49 illustrate that as the test temperature is increased, the optimum asphalt content of the limestone mixes is decreased. The desired asphalt content for the limestone mixes is approximately five percent.

Figure 50 illustrates that at 34°F the optimum asphalt content for the crushed caliche gravel mixtures would occur at or above 7.0 percent. Figures 51 and 52 indicate that the optimum asphalt content is between 6.0 and 7.0 percent asphalt. Investigating Figure 53, however, it is impossible to conclude that 7.0 percent asphalt is optimum because a higher asphalt content curve could lie either above or below the 7.0 percent curve. The relative portions of the curves, leads one to conclude that the optimum asphalt content is near 7.0 percent.

Figures 54, 55, and 56 illustrate that a well defined, peak asphalt content was not obtained for the crushed sandstone mixtures. Although the curves are of increasing slope, there is a tendency to level off near 9.0

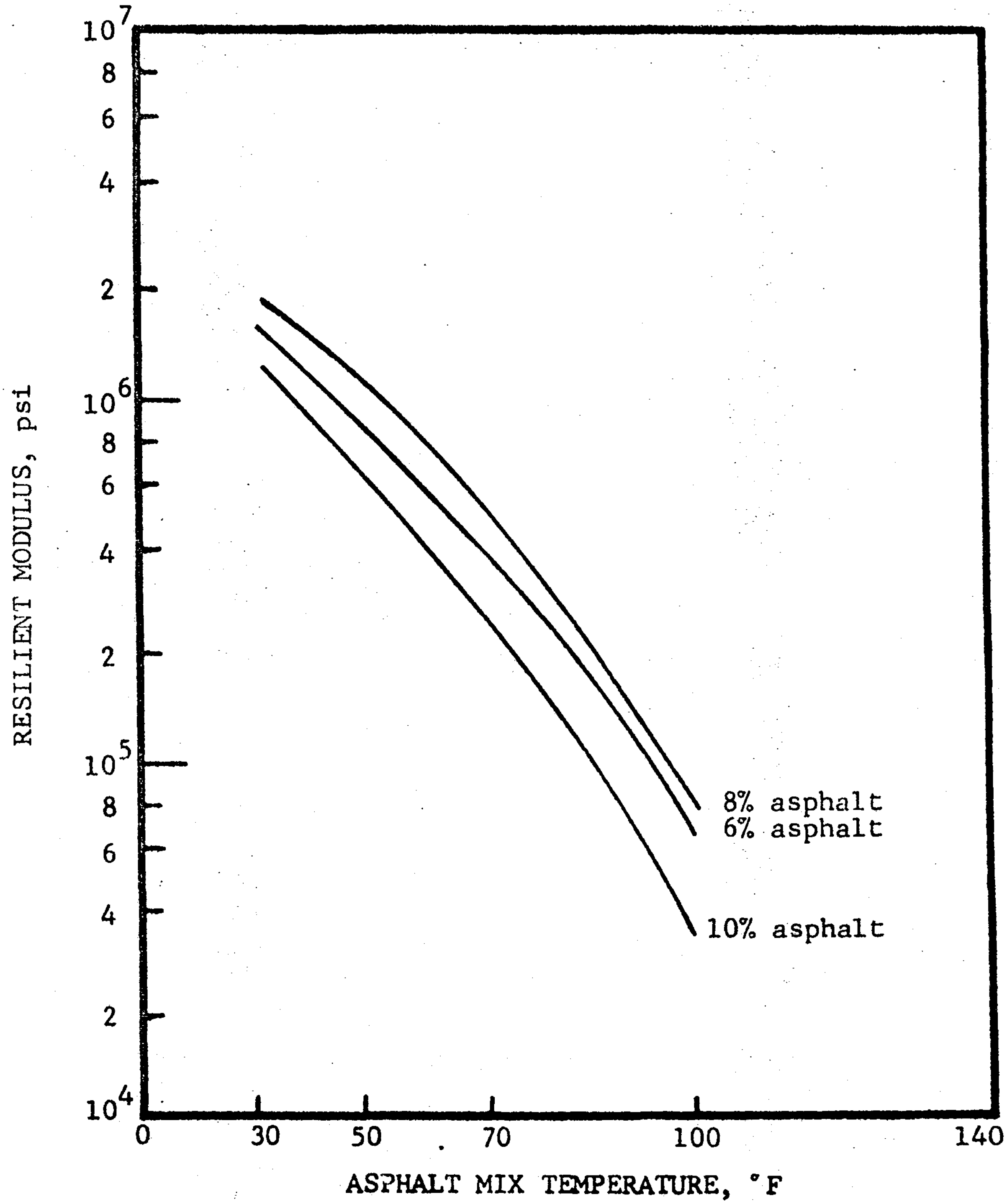

Figure 45. **Temperature susceptibility of sandstone mixes.**

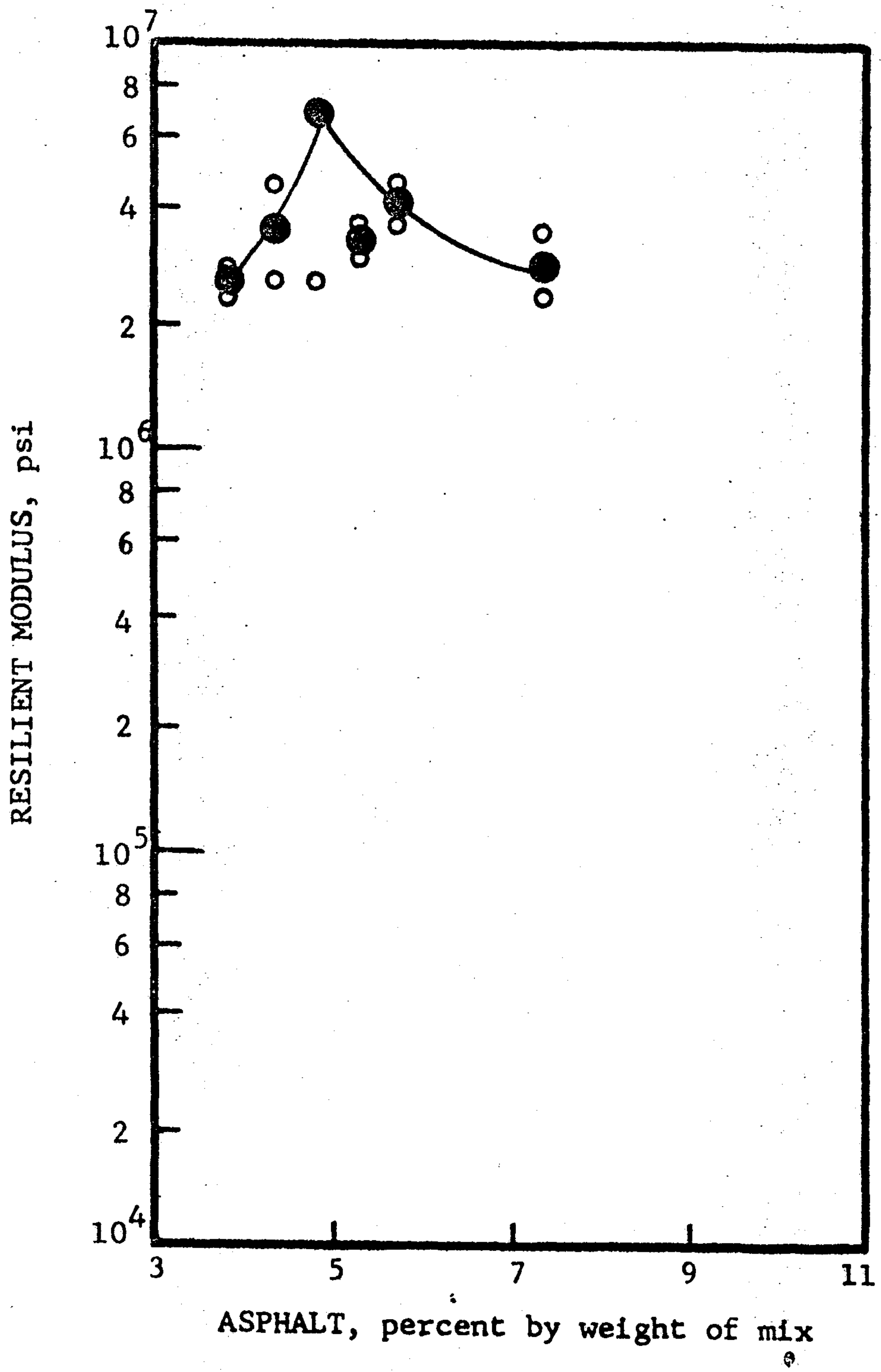

Figure 46. Resilient modulus of limestone mixtures at 34°F versus asphalt content.

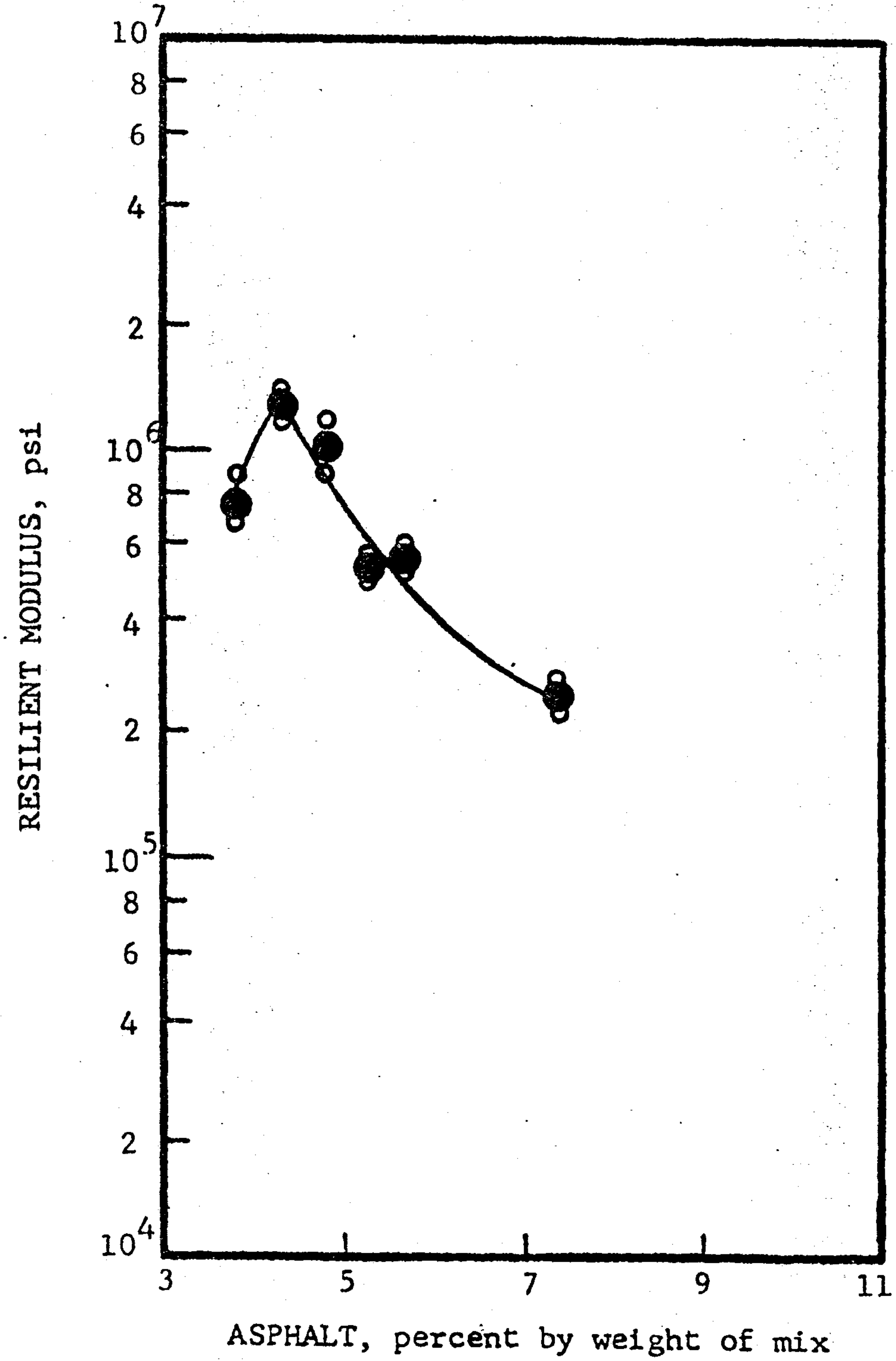

Figure 47. Resilient modulus of limestone mixtures at 73°F versus asphalt content (before vacuum saturation).

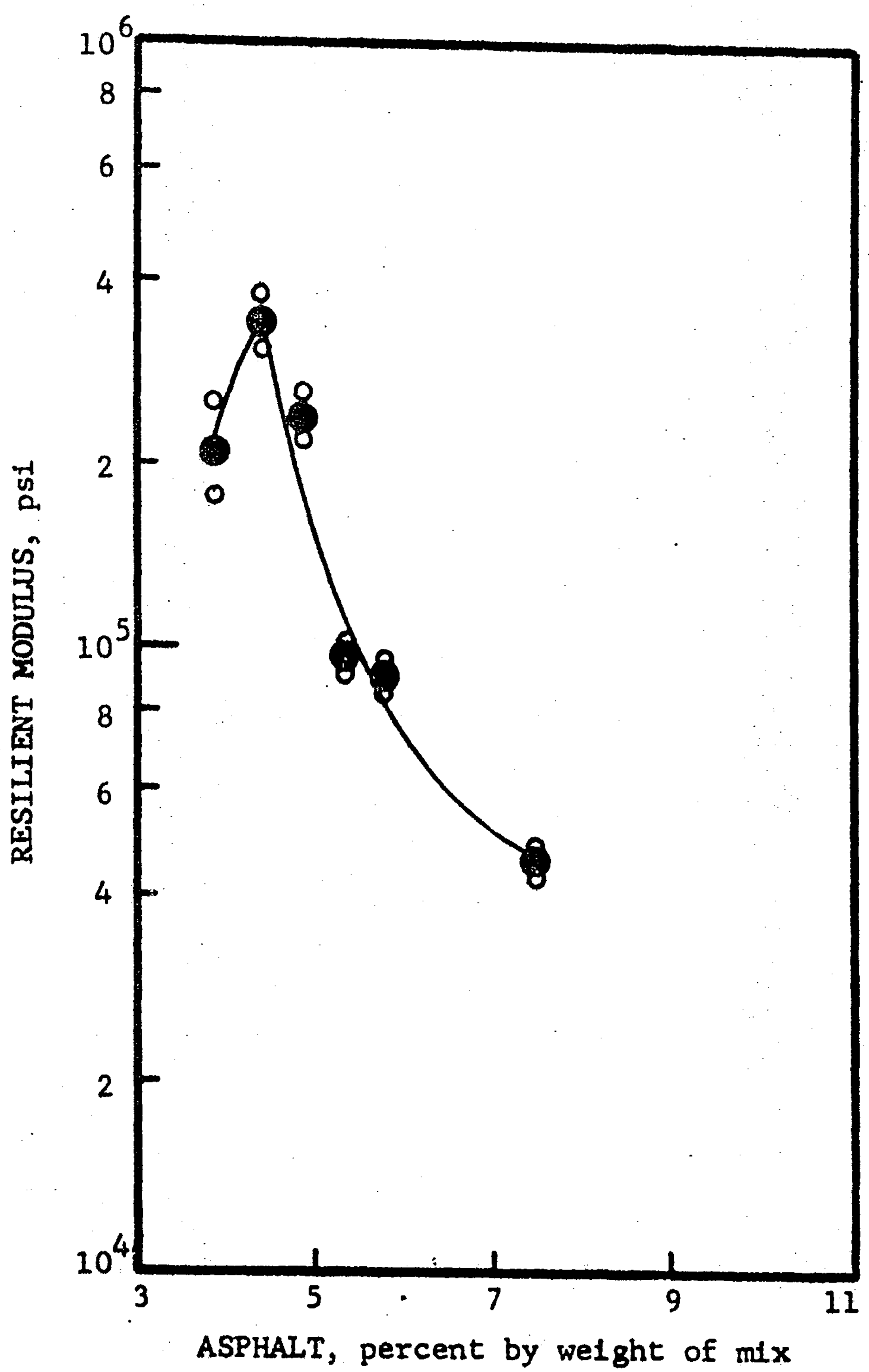

Figure 48. Resilient modulus of limestone mixtures at 100°F versus asphalt content.

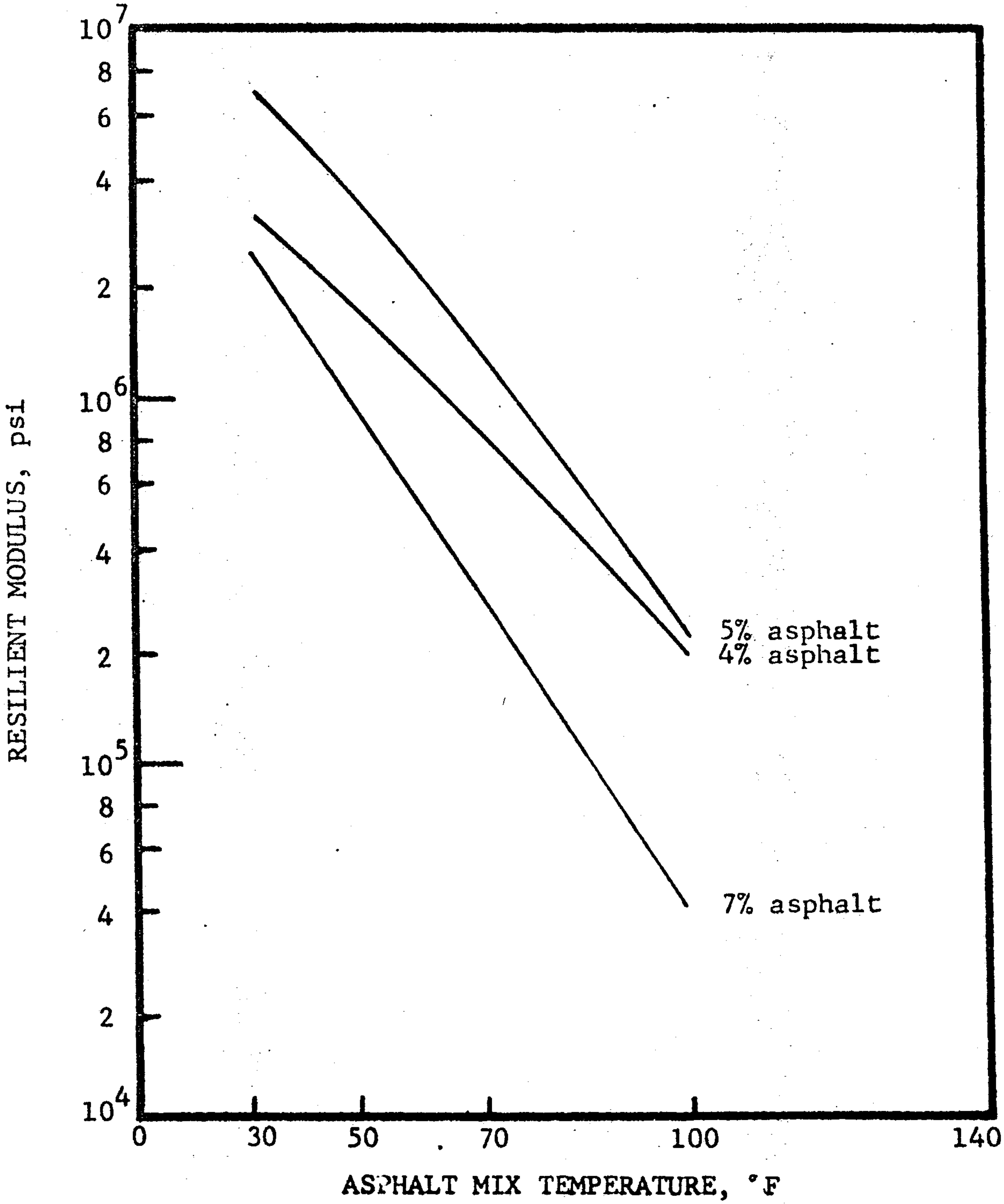

Figure 49. **Temperature susceptibility of limestone mixes.**

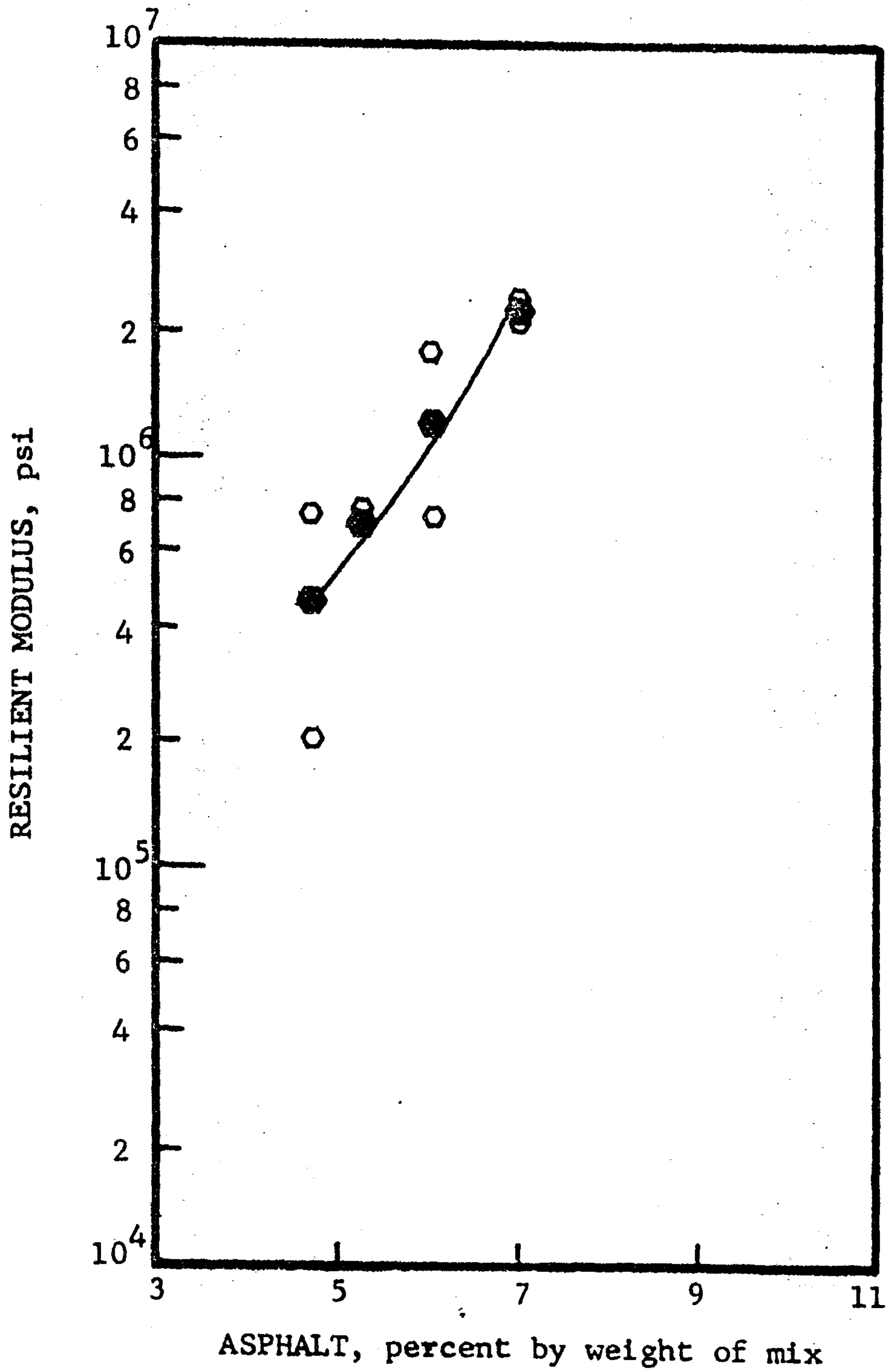

Figure 50. Resilient modulus of crushed caliche gravel mixtures at 34°F versus asphalt content.

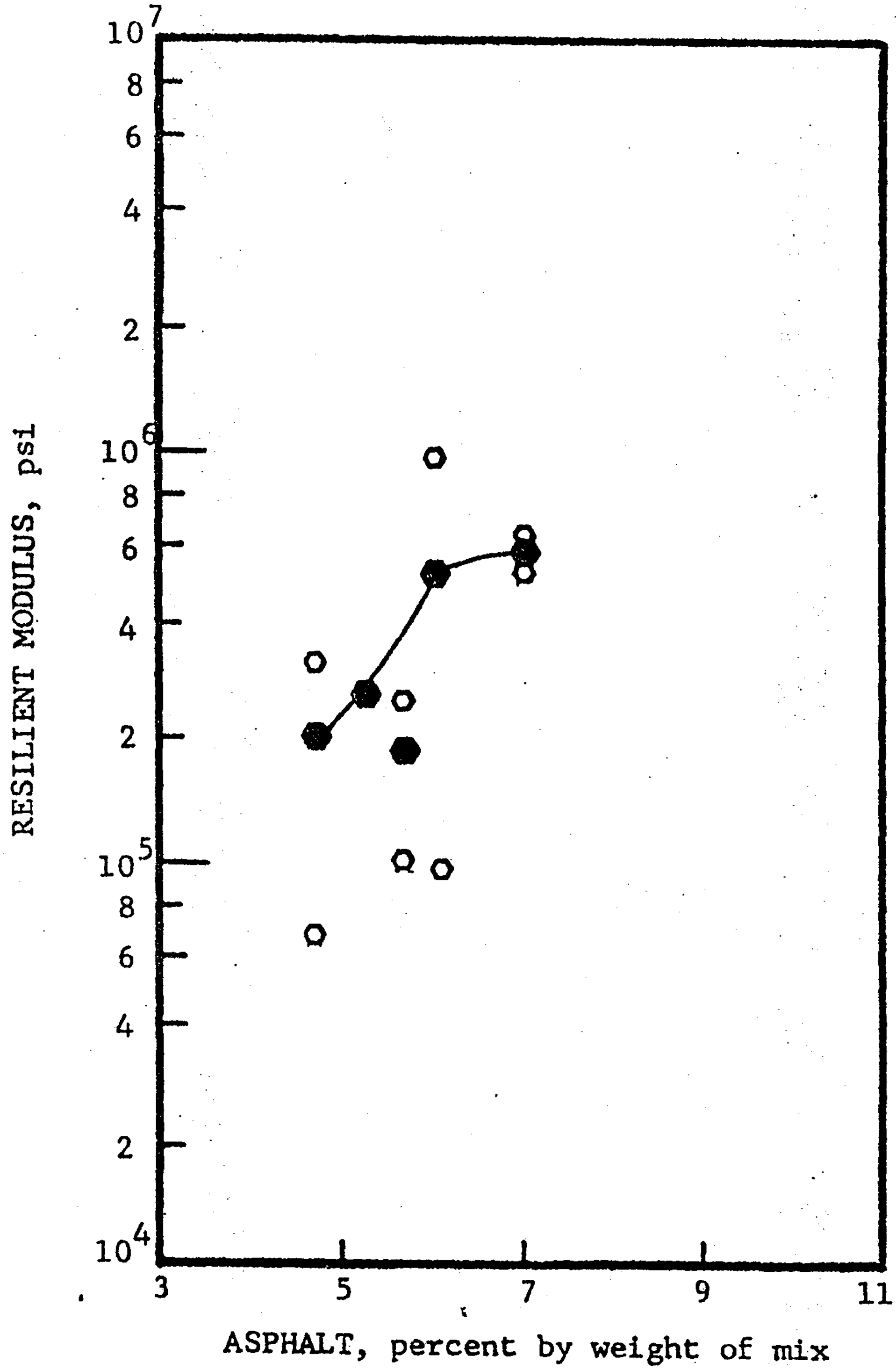

Figure 51. Resilient modulus of crushed caliche gravel mixtures at 73°F versus asphalt content (before vacuum saturation).

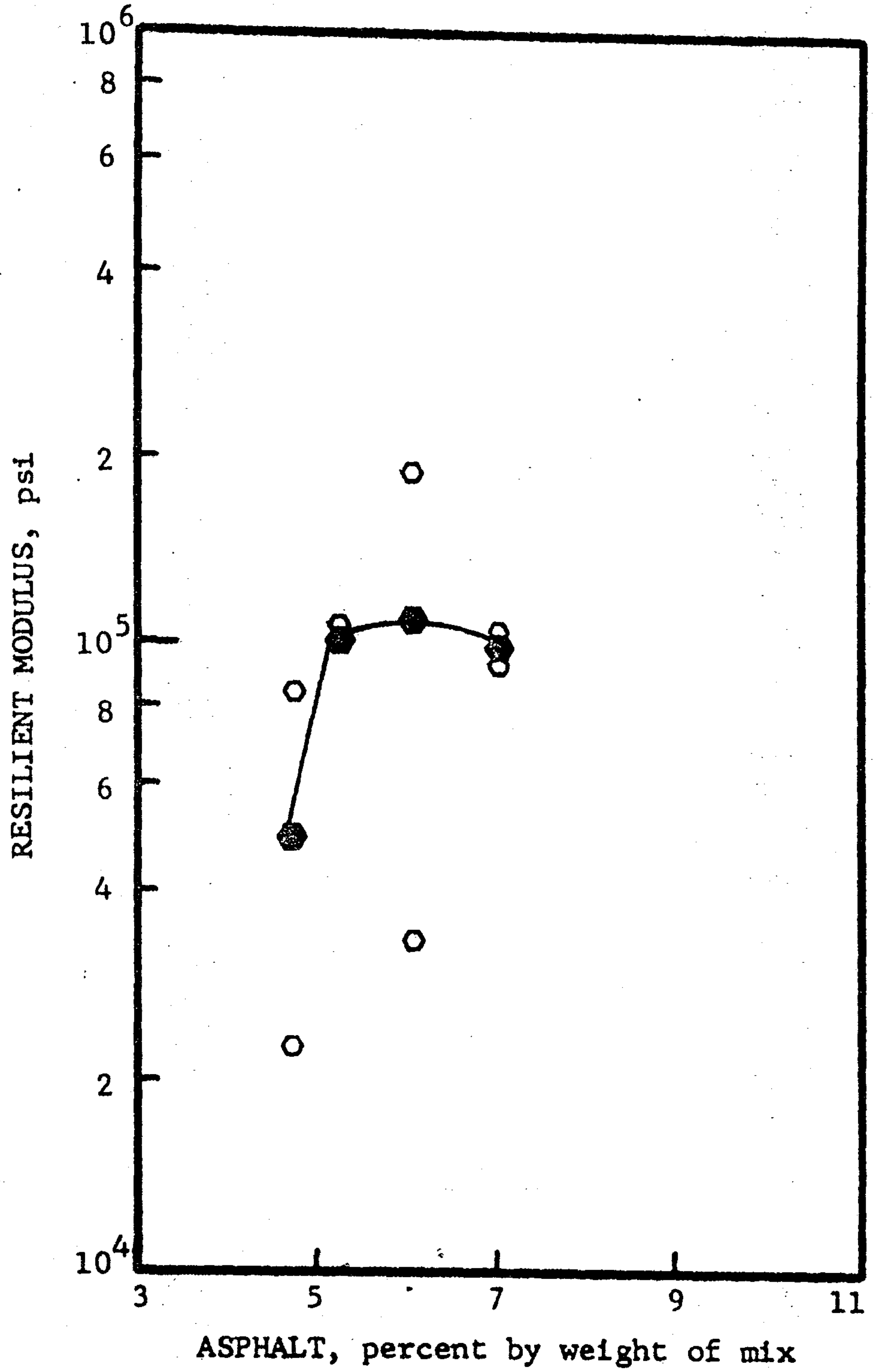

Figure 52. Resilient modulus of crushed caliche gravel mixtures at 100°F versus asphalt content.

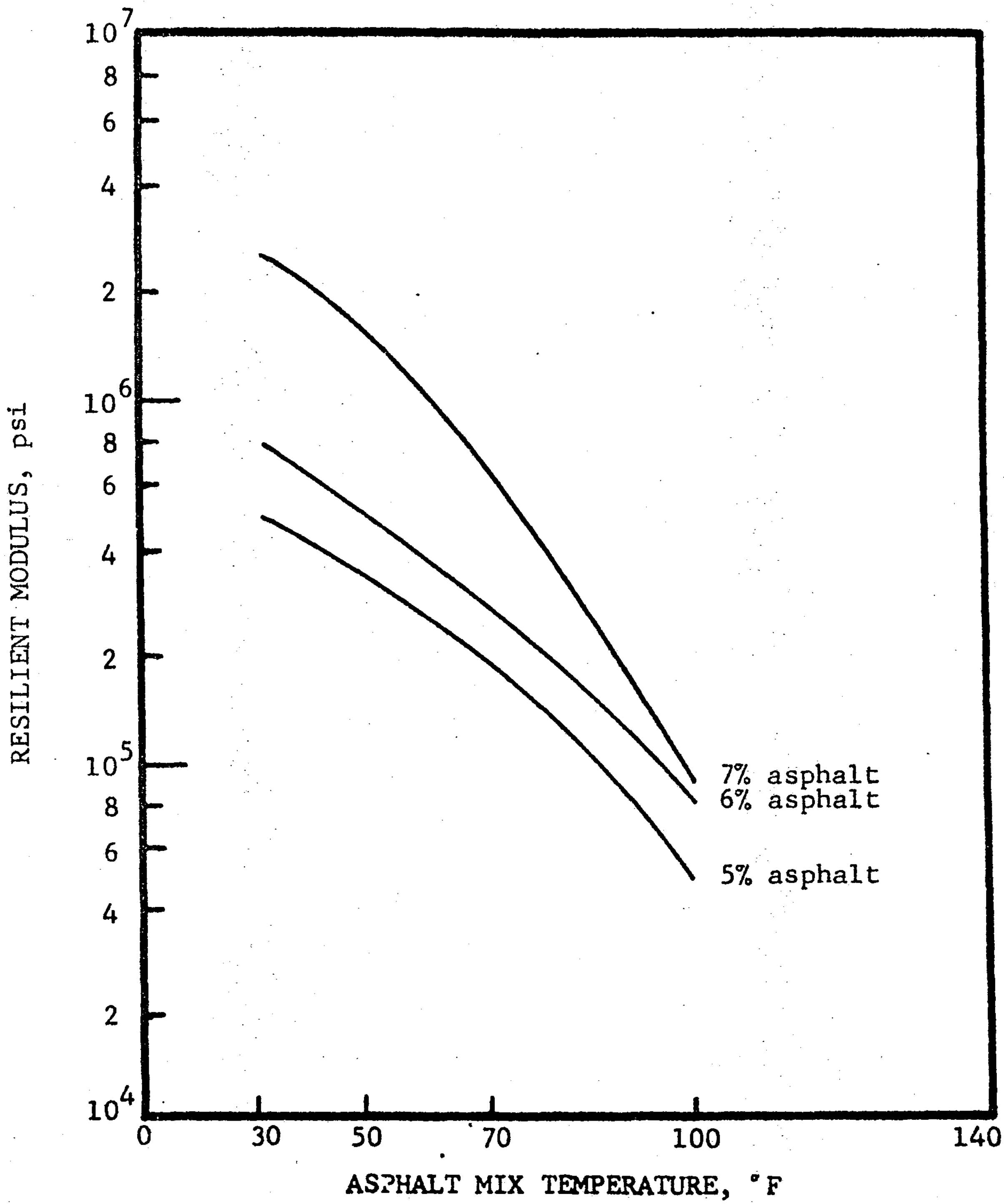

Figure 53. Temperature susceptibility of crushed caliche samples.

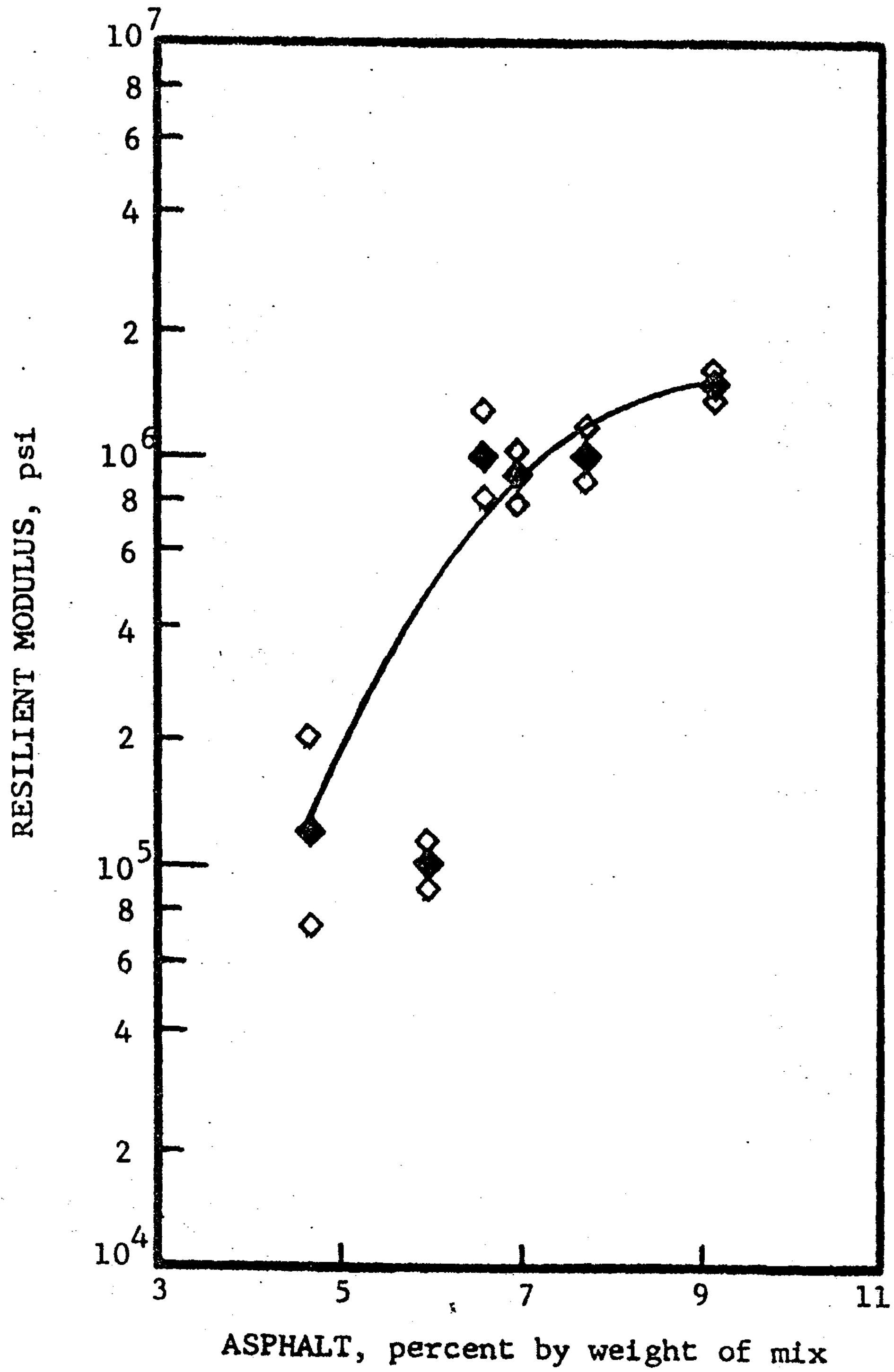

Figure 54. Resilient modulus of crushed sandstone mixtures at 34°F versus asphalt content.

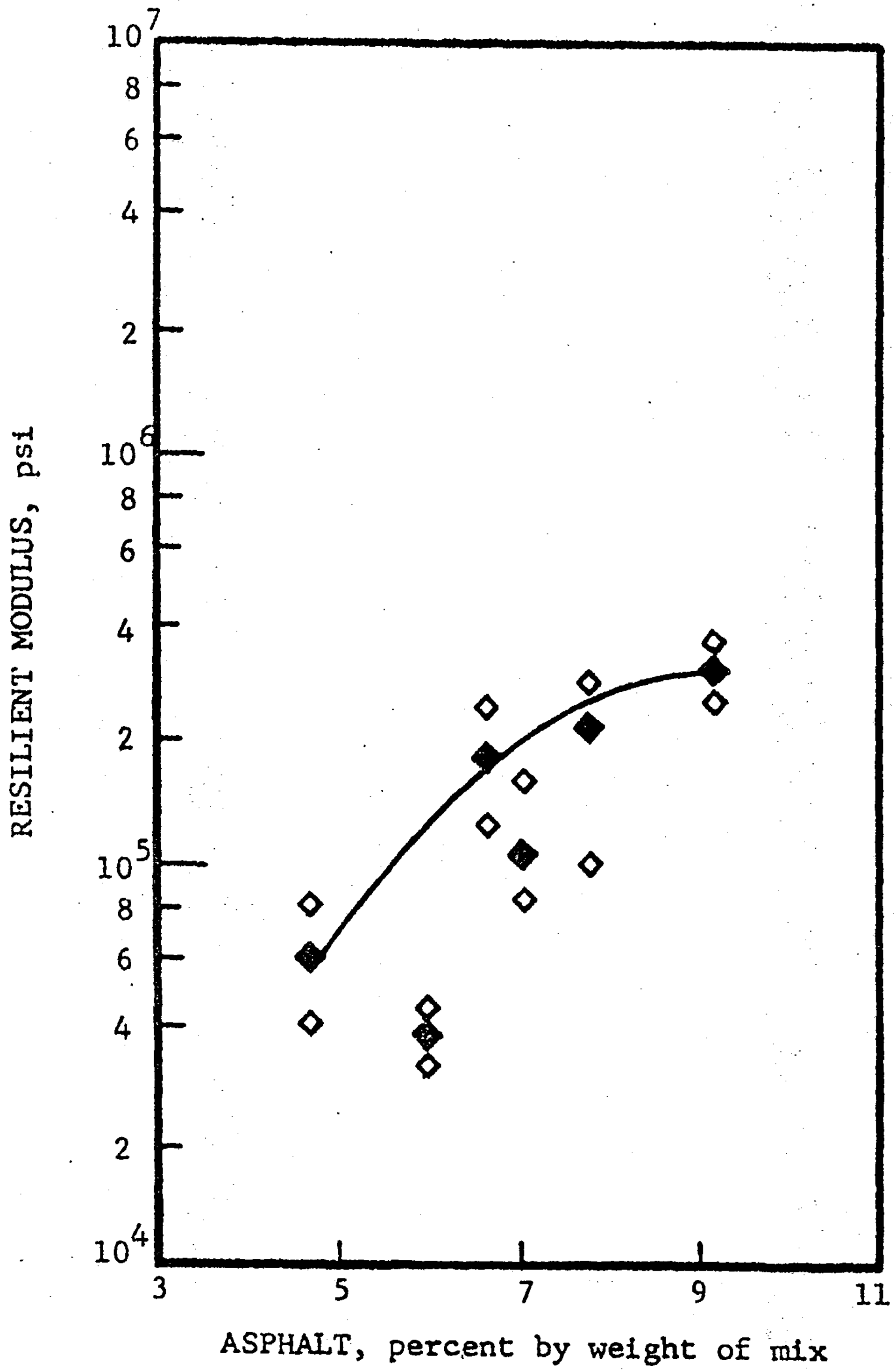

Figure 55. Resilient modulus of crushed sandstone mixtures at 73°F versus asphalt content (before vacuum saturation).

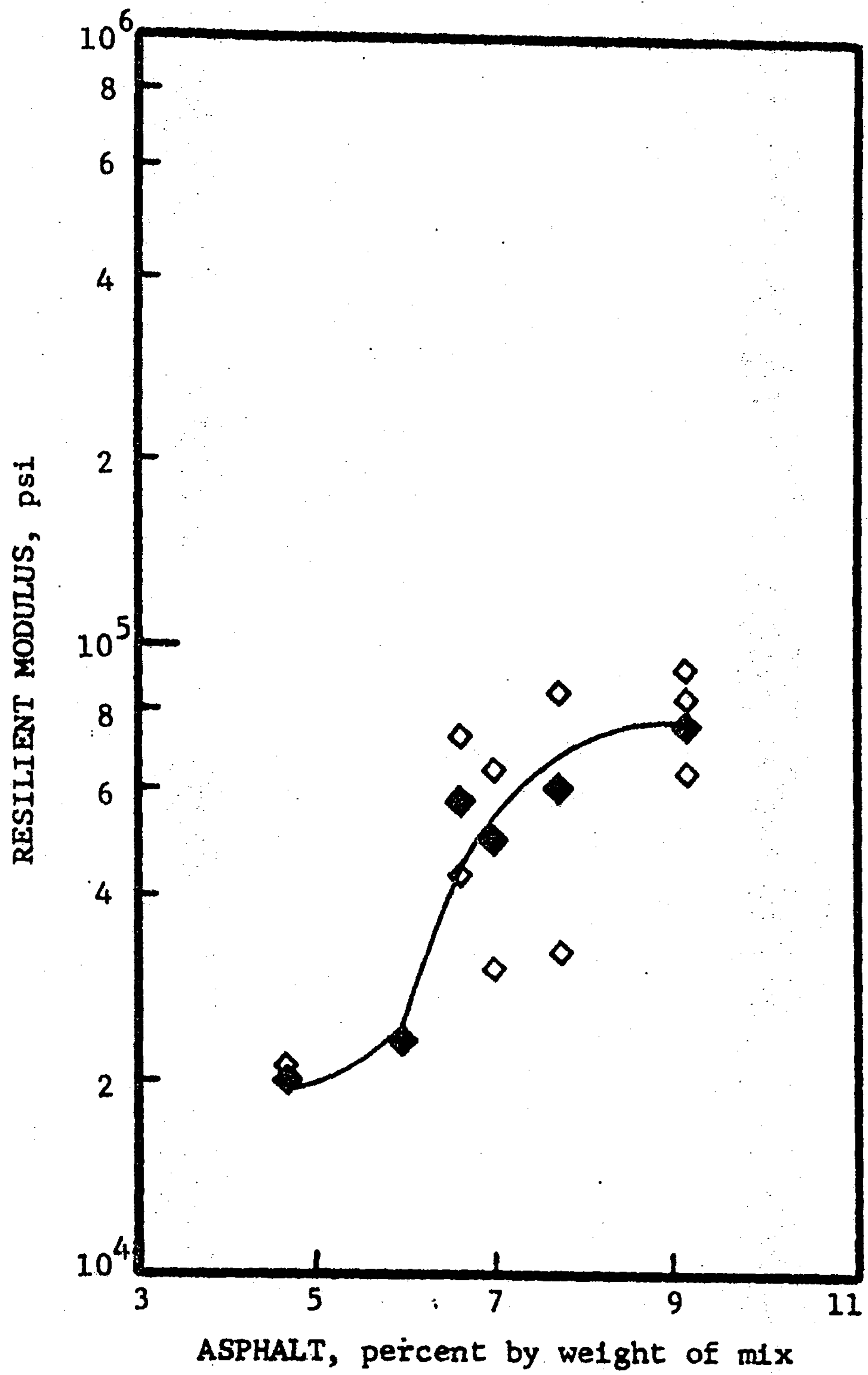

Figure 56. Resilient modulus of crushed sandstone mixtures at 100°F versus asphalt content.

percent asphalt. Basically, the three curves are of the same slope with only the magnitude of M_R as a variant. Figure 57 further exemplifies that samples were not molded at a high enough asphalt content to reach a peak in the curve. As with the crushed caliche gravel mixtures, the relative positions of the curves at seven, eight, and nine percent asphalt indicates that the optimum asphalt content is approximately nine percent.

The Austin chalk mixtures require significantly more than 8.0 percent asphalt (Figures 58, 59, and 60). The curves demonstrate little tendency to level off at at an optimum asphalt content. Figure 61 illustrates again that samples should be molded at considerably higher asphalt contents if an optimum be desired.

Stability of Mixes. The Hveem method has been used principally for the design of dense graded, hot asphalt paving mixtures. As developed by the California Division of Highways, the Hveem method is applicable to paving mixtures using both penetration grades and viscosity grades of asphalts and a maximum aggregate size of one inch. Through the years, the Hveem method of test has been further developed and improved by extensive research and correlation studies on laboratory design and field control of asphalt pavements. Hveem method test procedures have been standardized by the American Society for Testing and Materials (20) and are designated as ASTM D 1560, Resistance to Deformation and Cohesion of Bituminous Mixtures by Means of Hveem Apparatus and ASTM D 1561, Preparation of Test Specimens of Bituminous Mixtures by Means of California Kneading Compactor.

As explained above, the marginal aggregate black base mixtures were molded according to test method Tex-126-E (26). The samples were tested for stability according to test method Tex-208-F, Test for Stabilometer

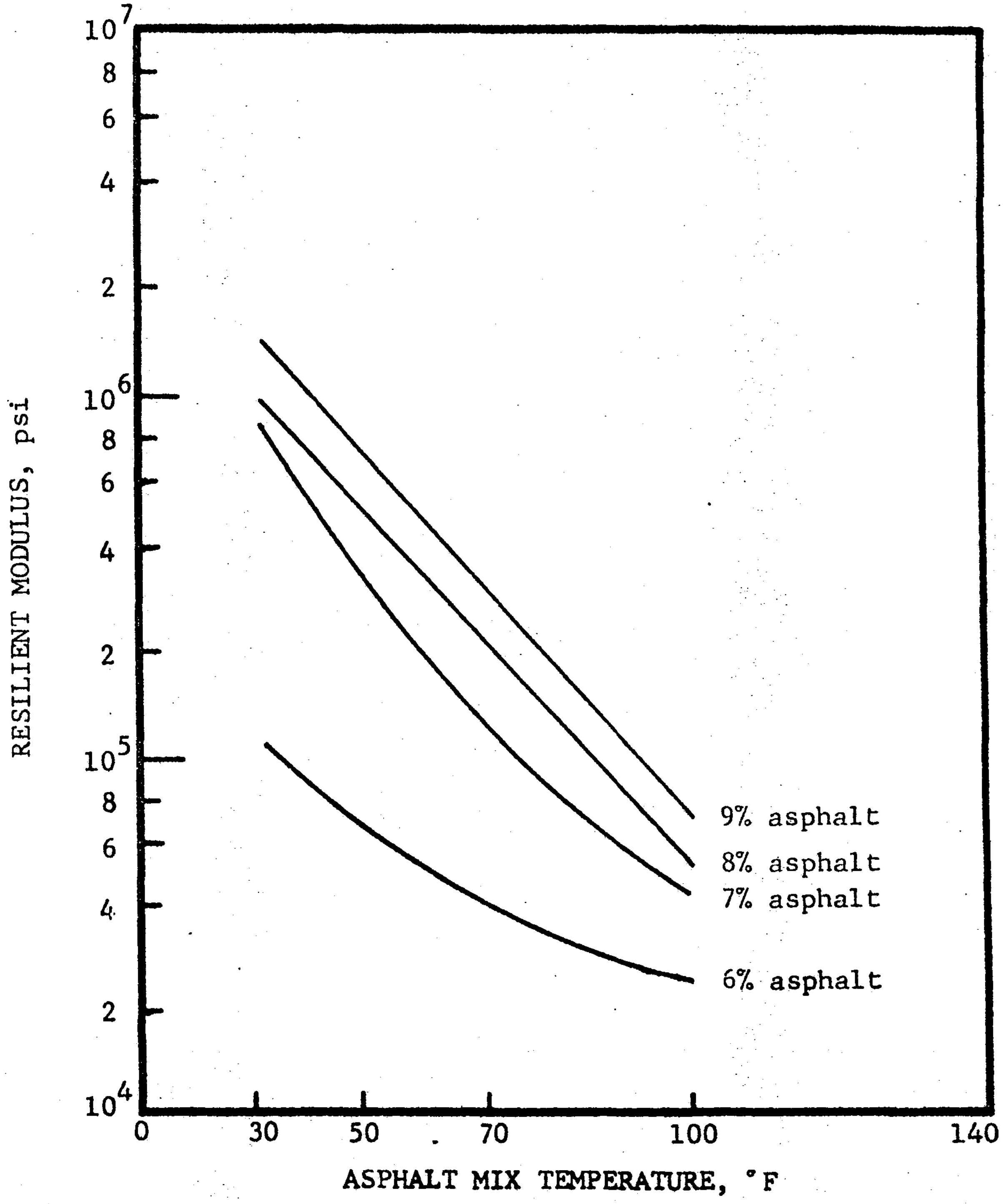

Figure 57. Temperature susceptibility of crushed sandstone samples.

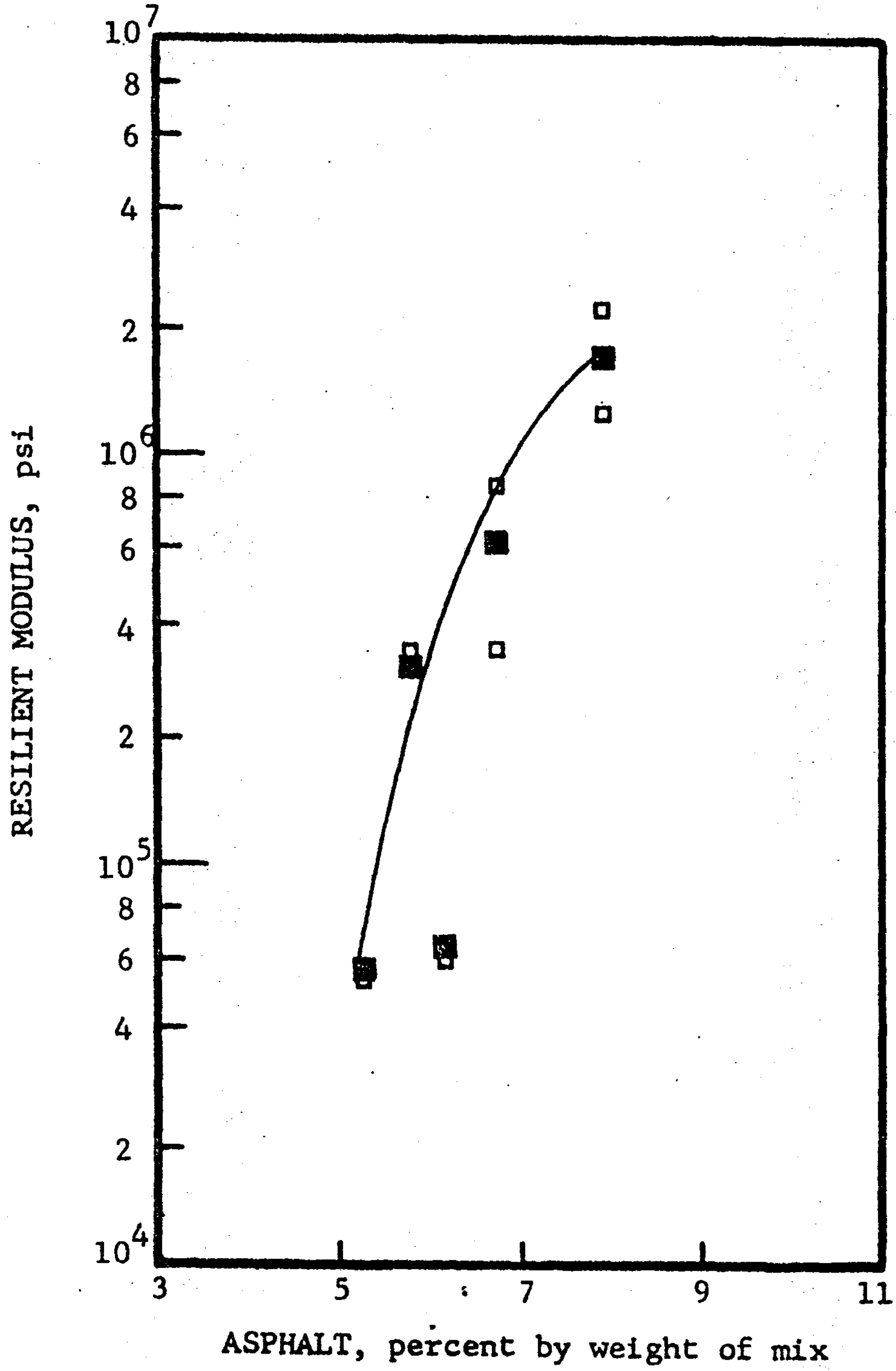

Figure 58. Resilient modulus of Austin chalk mixtures at 34°F versus asphalt content.

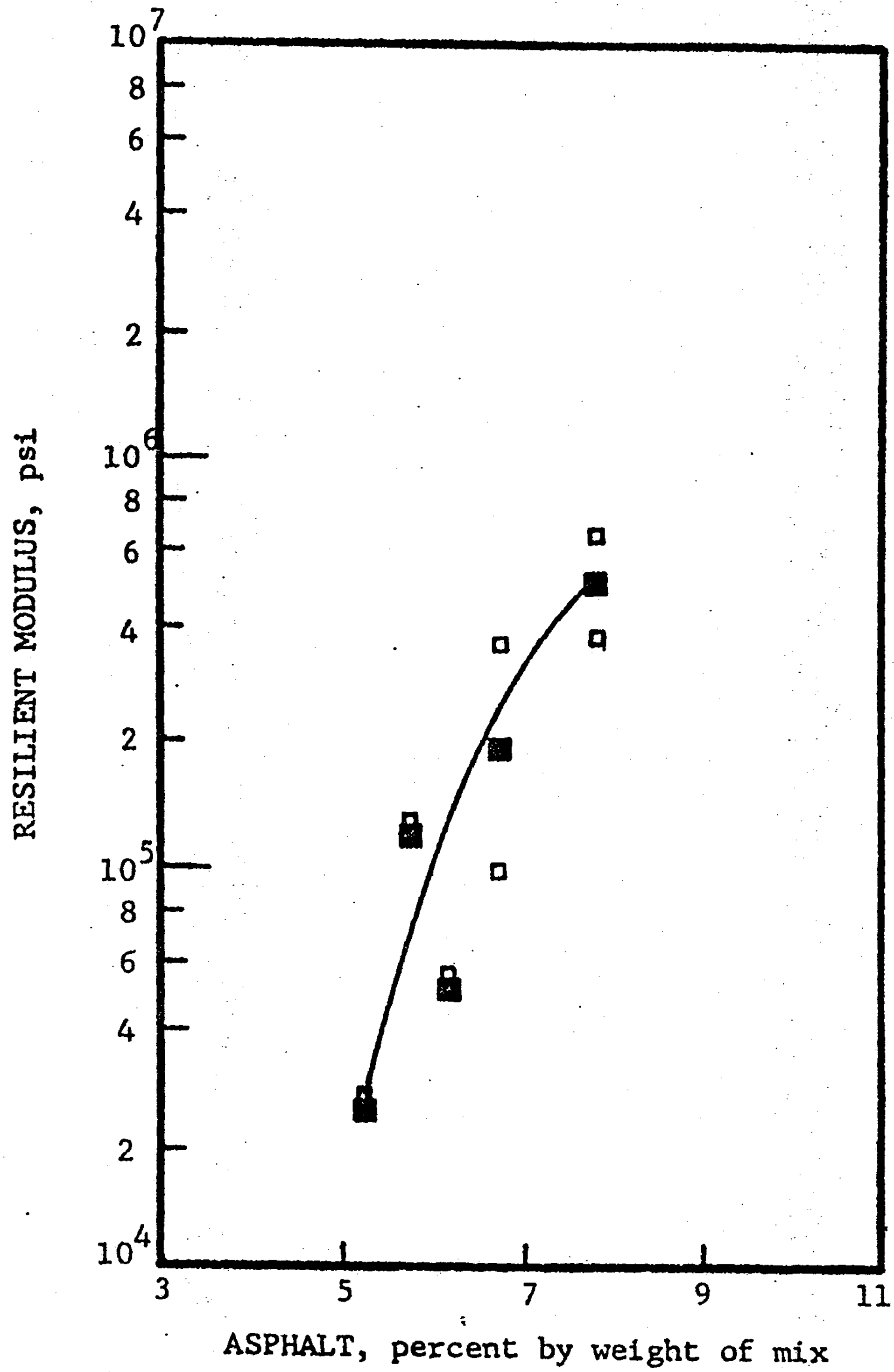

Figure 59. Resilient modulus of Austin chalk mixtures at 73°F versus asphalt content (before vacuum saturation).

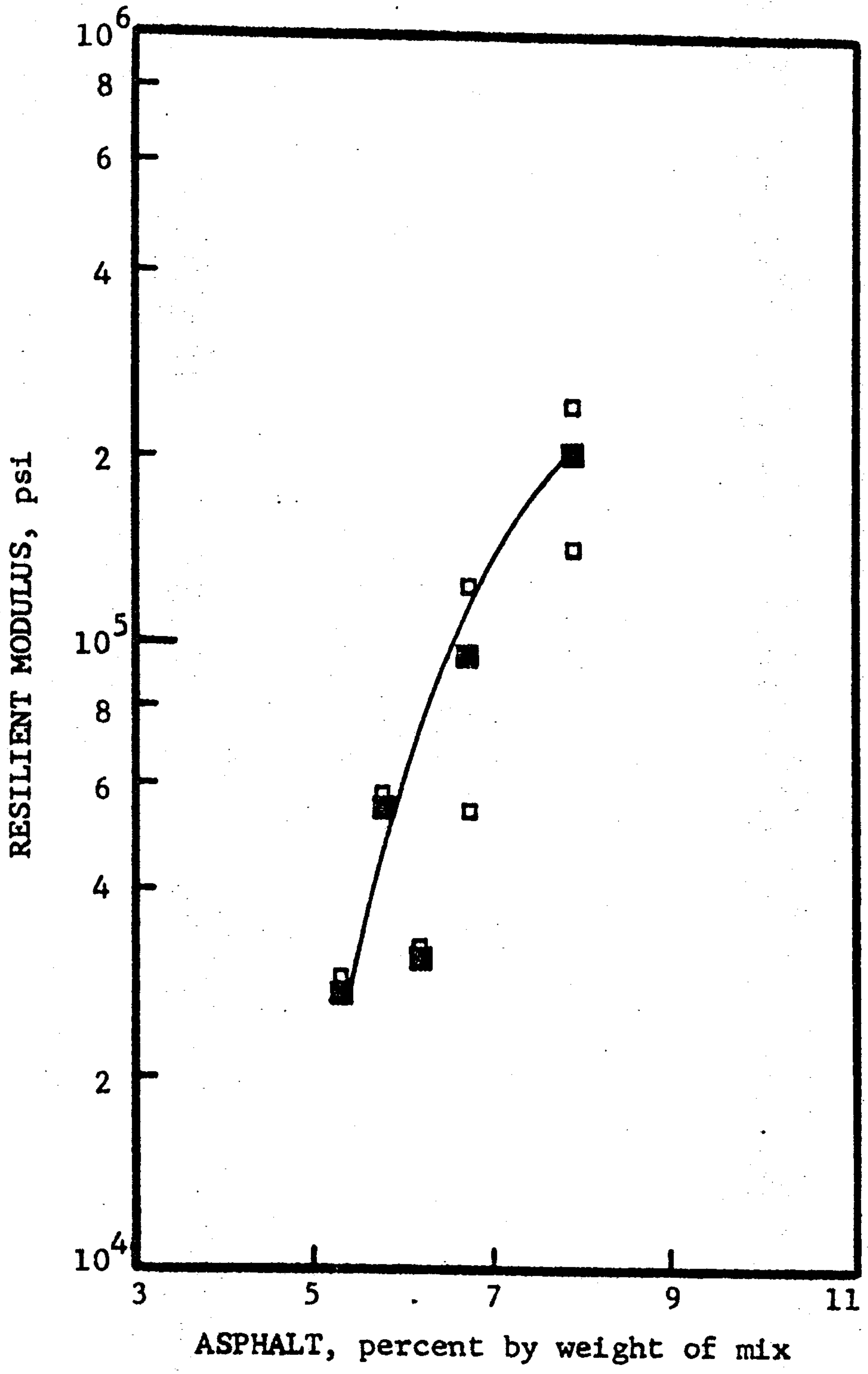

Figure 60. Resilient modulus of Austin chalk mixtures at 100°F versus asphalt content.

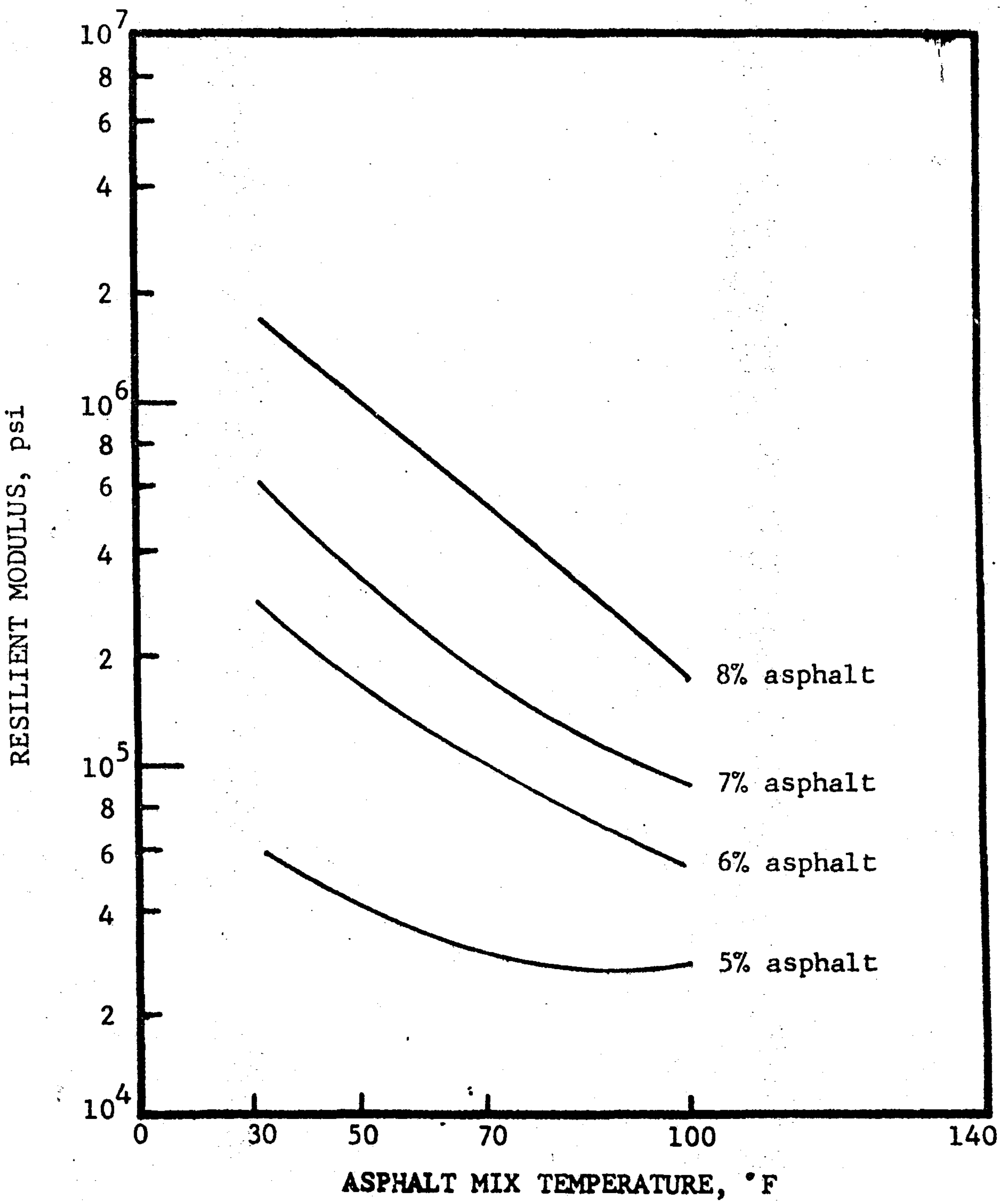

Figure 61. **Temperature susceptibility of Austin chalk samples.**

Value of Bituminous Mixtures (26) which is a modification of ASTM D 1560 (24). An additional modification to test method Tex-208-F was mandatory in the course of this study, that is, the maximum allowable nominal size aggregate, according to Tex-208-F, is 7/8-inch and the maximum size aggregate used in the black base samples was 1½-inches. The purpose of the test is to measure the shearing resistance of the material which results primarily from the internal friction of the aggregate and the effect of the larger aggregate particles on stabilometer values is difficult to determine. It is believed, however, since the samples were not molded as 4-inch by 2-inch specimens, the effect of the larger particles is minimal. Furthermore, the original 6-inch by 8-inch specimens contained only 20 percent aggregate (by volume) above the allowable 7/8-inch maximum size.

The Hveem method of mix design and stability criteria is quite popular in many of the western states. Most states which use the Hveem criteria incorporate percent air voids into the design specifications such that the optimum asphalt content occurs in the range of 3 to 7 percent air voids and the mixture has a Hveem stability above 35. Texas, however, has no specified provisions for air voids but rather that optimum asphalt content occurs slightly above the break in the Asphalt-Voids Ratio curve and requires a minimum stability of 30 for bituminous aggregate base courses (21).

The laboratory molded, black base mixtures investigated in this study reported consistently high Hveem stability values, even at lower asphalt contents. In fact, only at asphalt contents one and two percent above optimum did the mixtures report stabilities at or below 30. The following set of curves illustrate the effect of asphalt content on the Hveem stability of the various black base mixtures. Hveem results were obtained before and after vacuum saturation to investigate the effect of water on the stability of the mixtures.

Figure 62 illustrates the stability versus asphalt content relationship of sandstone mixtures prior to vacuum saturation. Maximum stability occurs between 6.0 and 7.0 percent asphalt. After vacuum saturation and drying, the relative stabilities increased (Figures 63 and 64). Many of the samples, particularly those at lower asphalt contents, either failed during vacuum saturation or became permanently swollen and were unsuitable for testing.

The limestone mixtures exhibit maximum stability between 4.0 and 5.0 percent asphalt (Figures 65 and 66). Again the stability values increased after vacuum saturation (Figure 67). The limestone mixtures experienced only slight amounts of swell due to the presence of water.

Figure 68 demonstrates the maximum stability for the crushed caliche gravel mixtures as occurring between 6.0 and 7.0 percent asphalt. Because many of the samples either failed during vacuum saturation or became swollen beyond acceptable test dimensions, stabilometer values after vacuum saturation were slight. Figures 69 and 70 illustrate the values which were obtained.

Stabilometer values indicate an optimum asphalt content for the crushed sandstone mixtures between 6.0 and 7.0 percent (Figure 71). Interestingly, however, all samples below 6.0 percent asphalt were unsuitable for testing after vacuum saturation (Figure 72). Furthermore, Figure 73 illustrates that the crushed sandstone mixtures were the only mixtures to experience

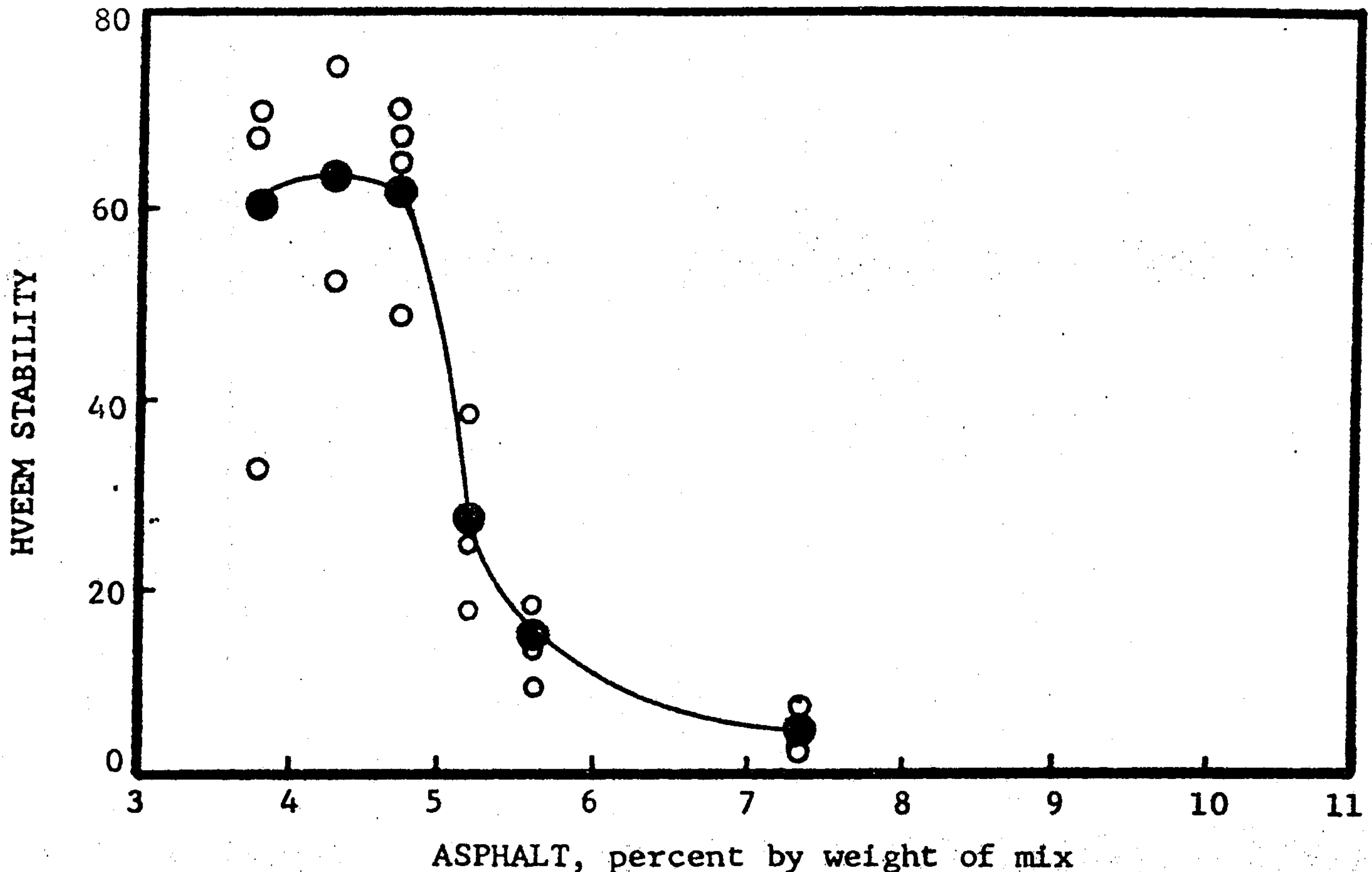

Figure 62. Hveem stability versus asphalt content for limestone mixtures (before vacuum saturation).

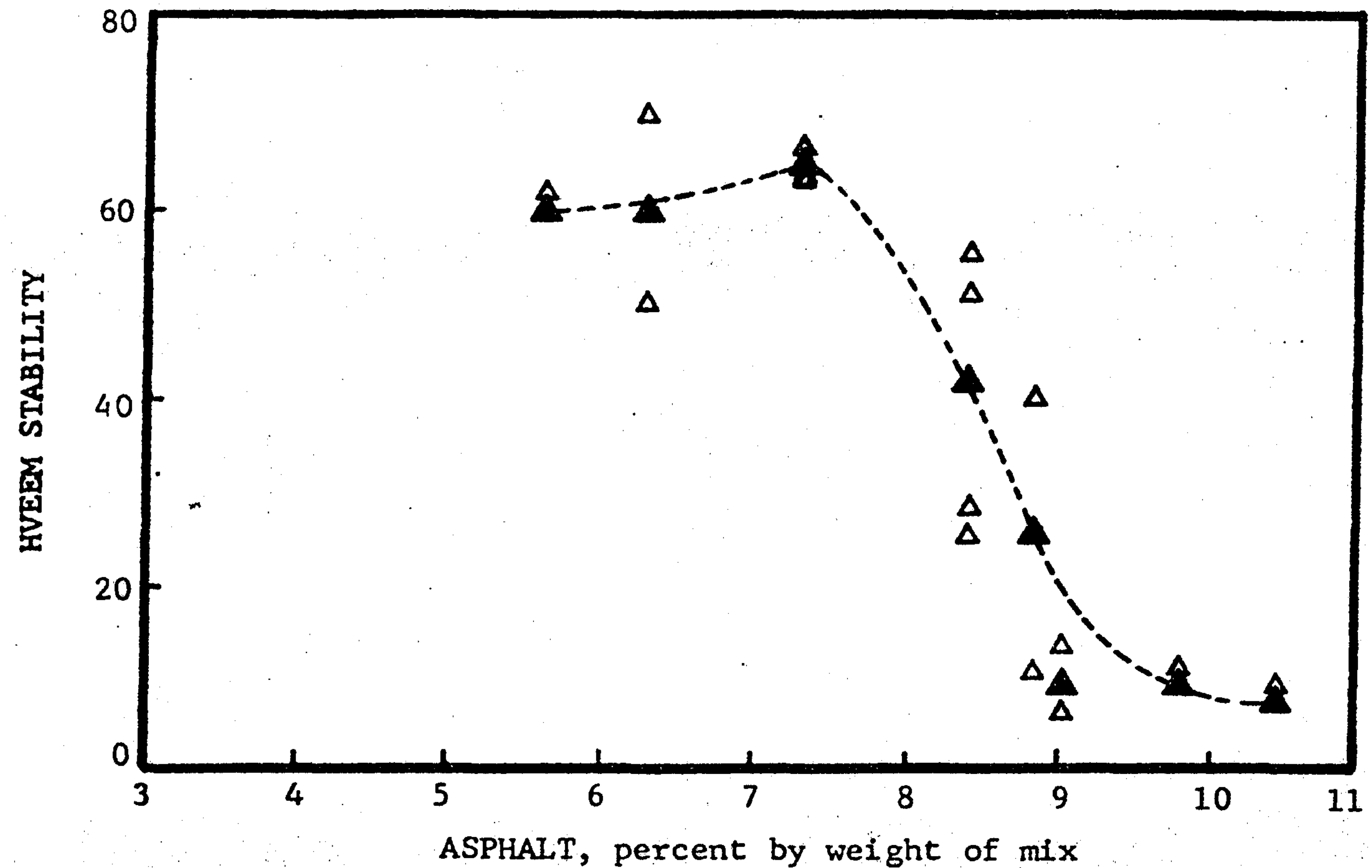

Figure 63. Hveem stability versus asphalt content for sandstone mixtures (after vacuum saturation).

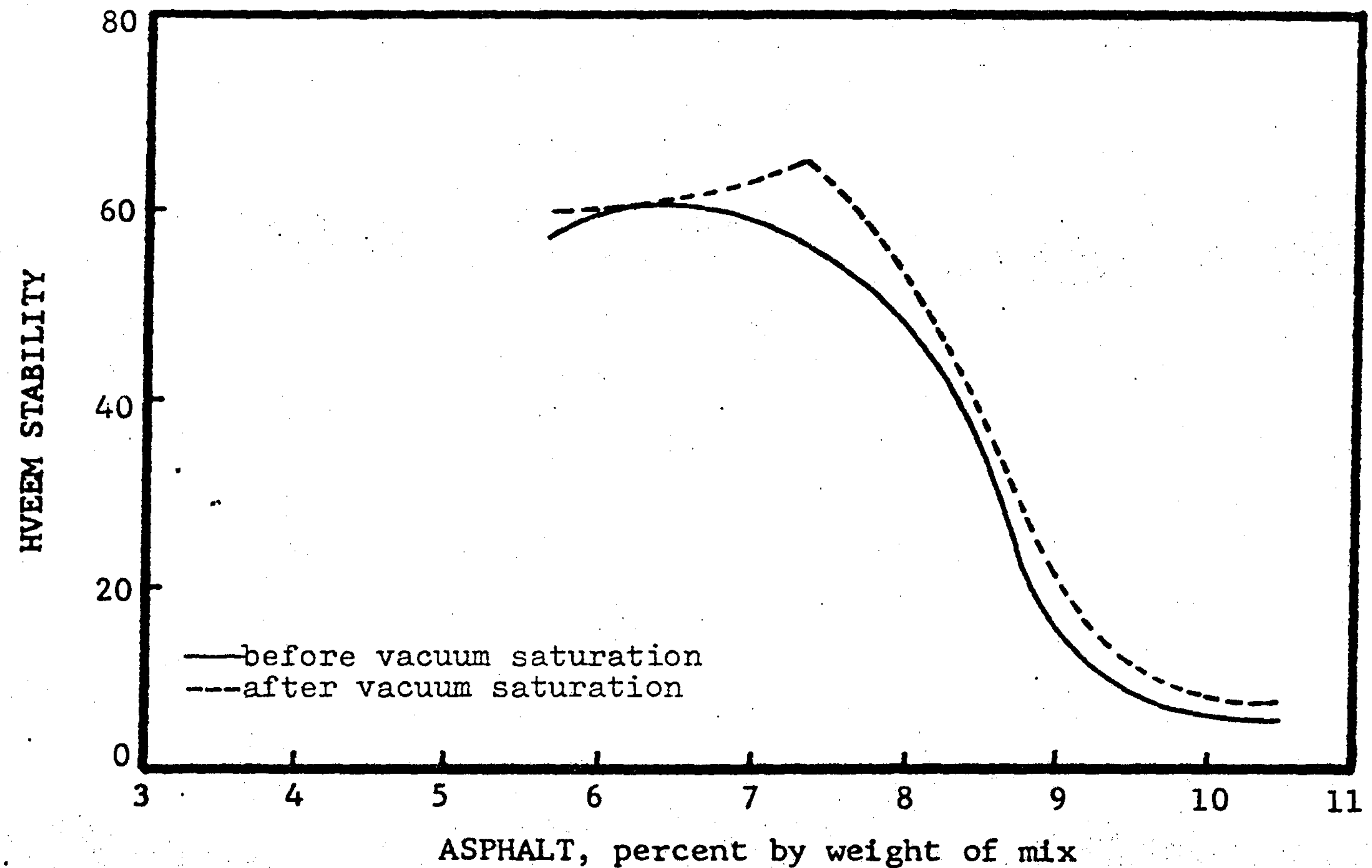

Figure 64. Effect of vacuum saturation on the Hveem stability of sandstone mixtures.

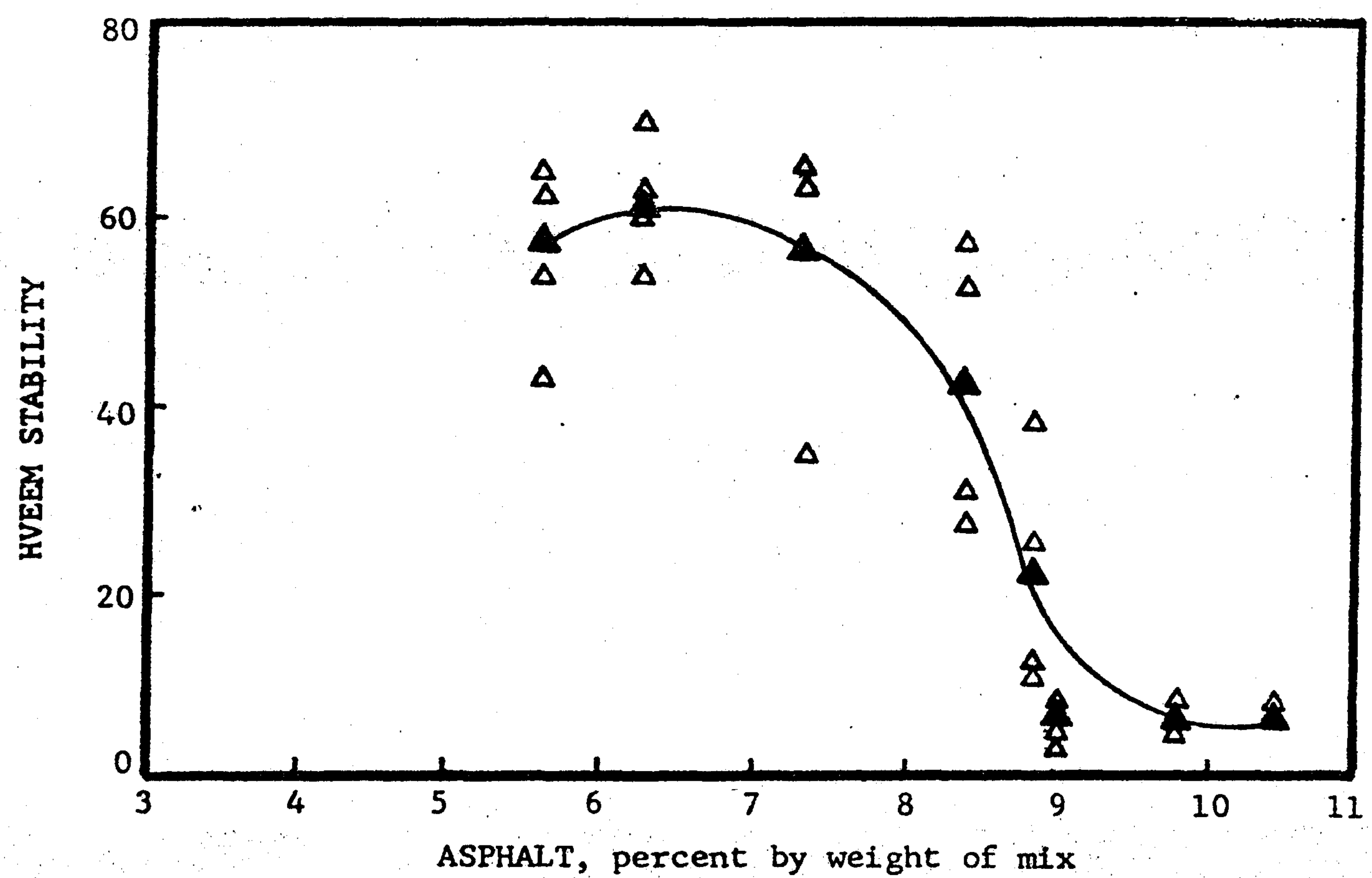

Figure 65. Hveem stability versus asphalt content for sandstone mixtures (before vacuum saturation).

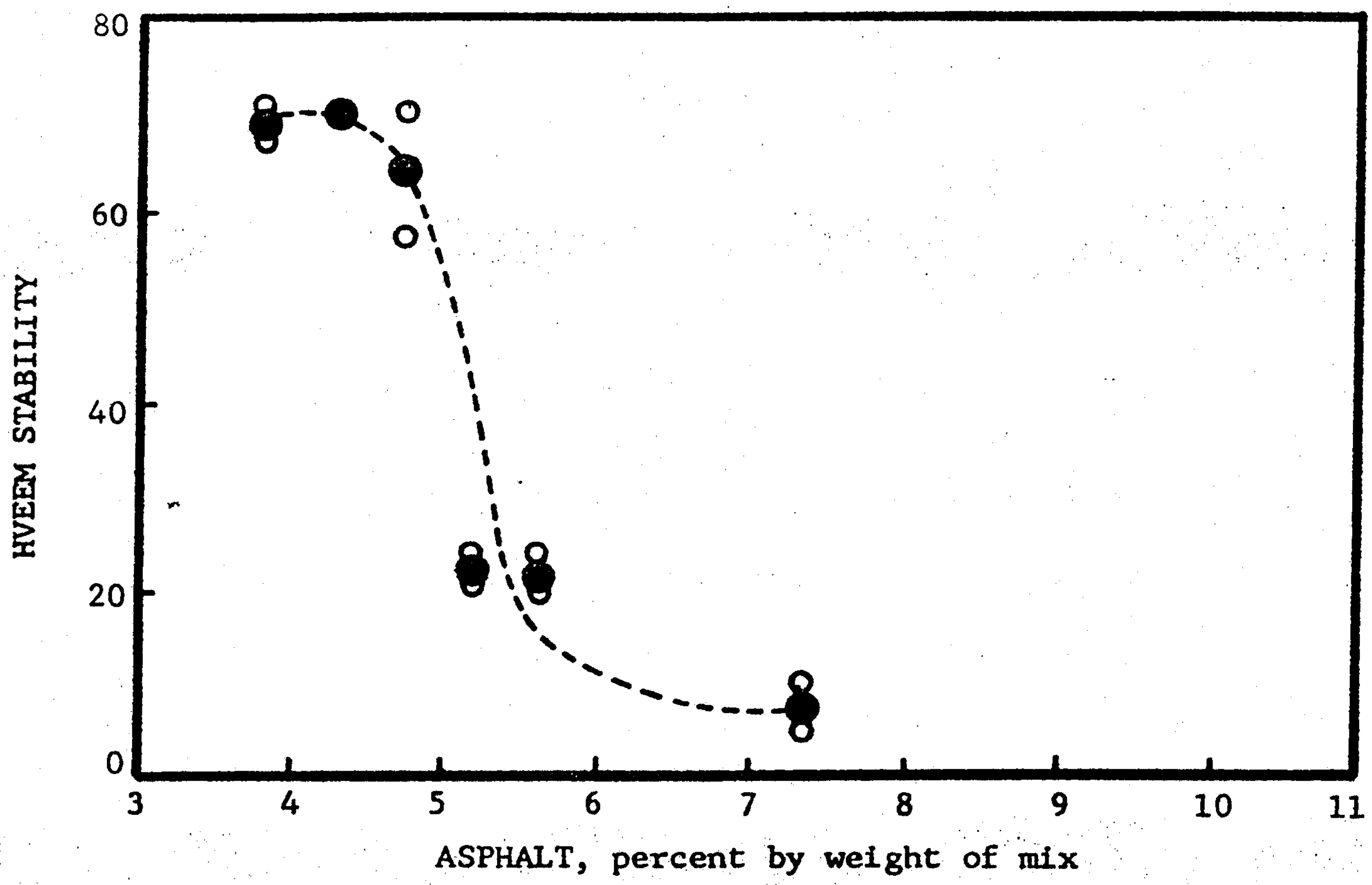

Figure 66. Hveem stability versus asphalt content for limestone mixtures (after vacuum saturation).

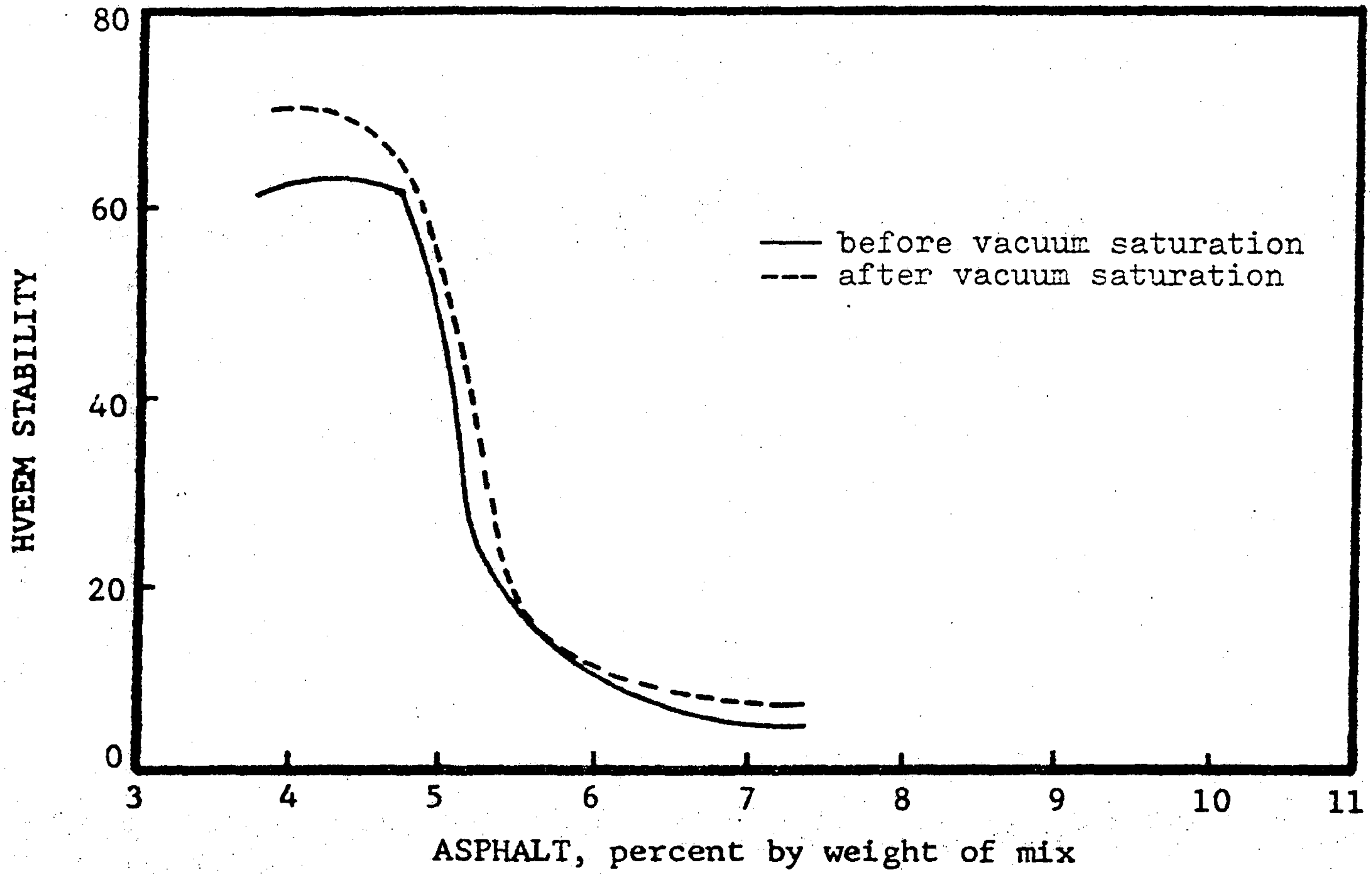

Figure 67. Effect of vacuum saturation on the Hveem stability of limestone mixtures.

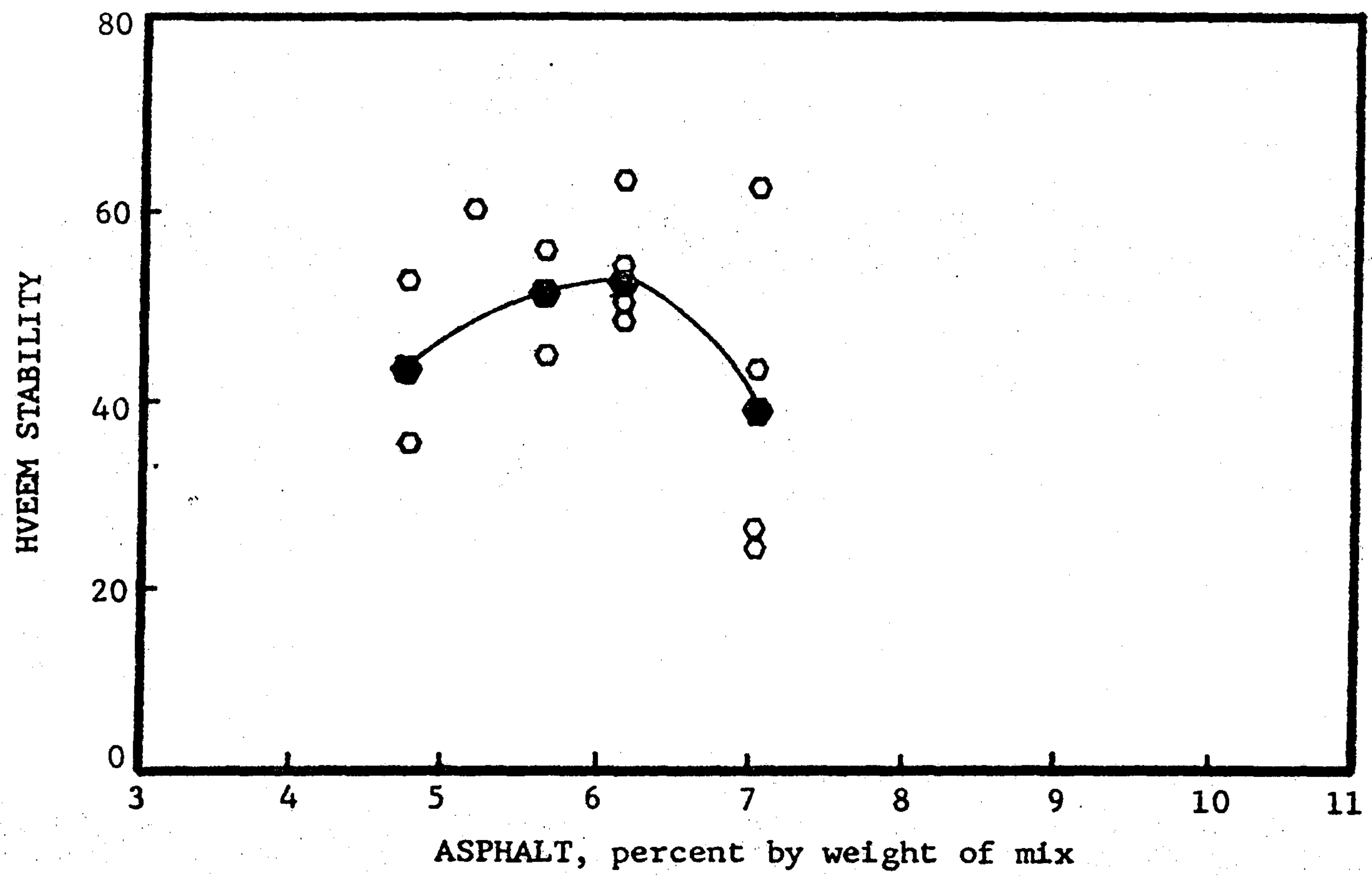

Figure 68. Hveem stability versus asphalt content for crushed caliche gravel mixtures (before vacuum saturation).

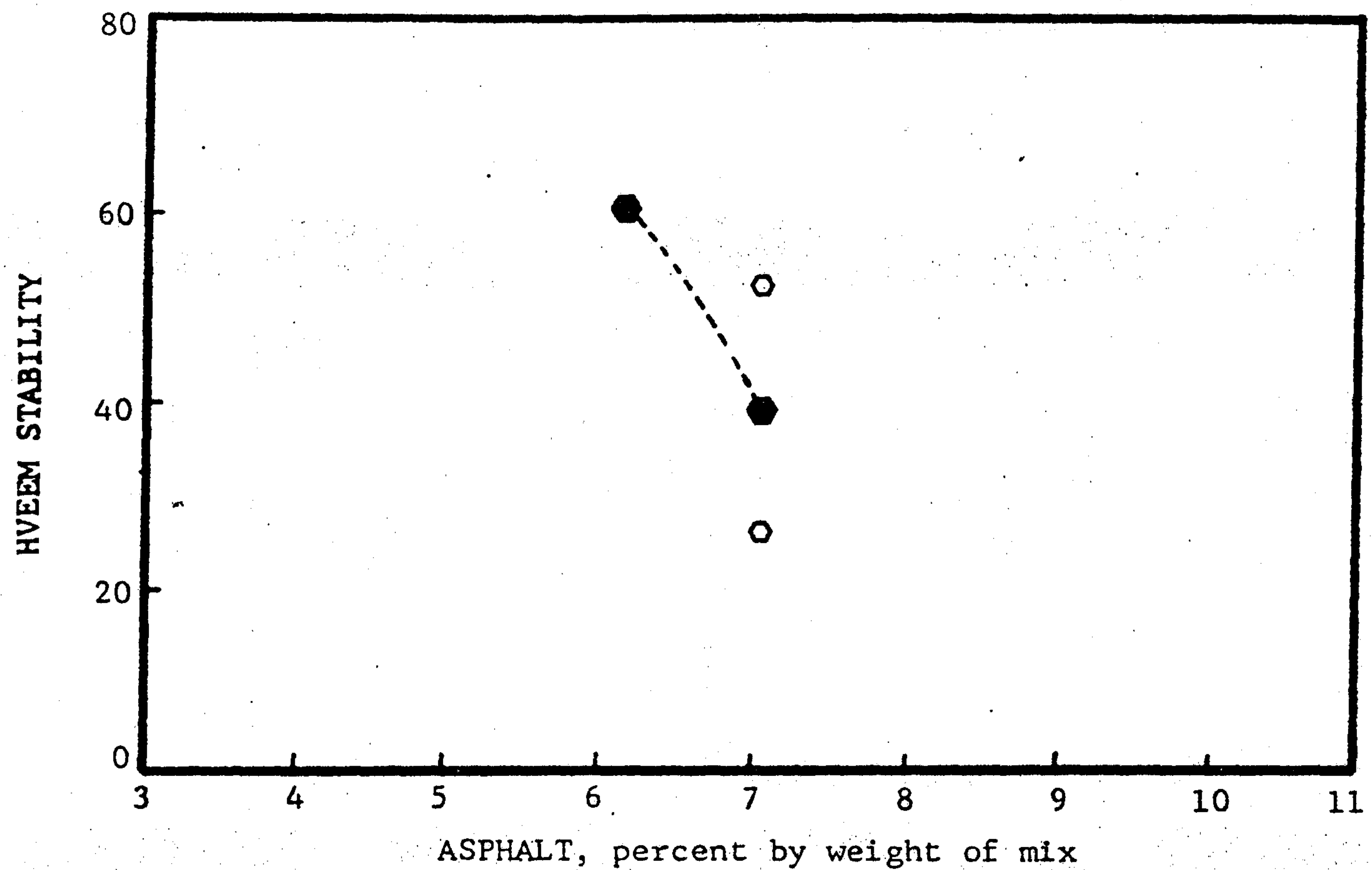

Figure 69. Hveem stability versus asphalt content for crushed caliche gravel mixtures (after vacuum saturation).

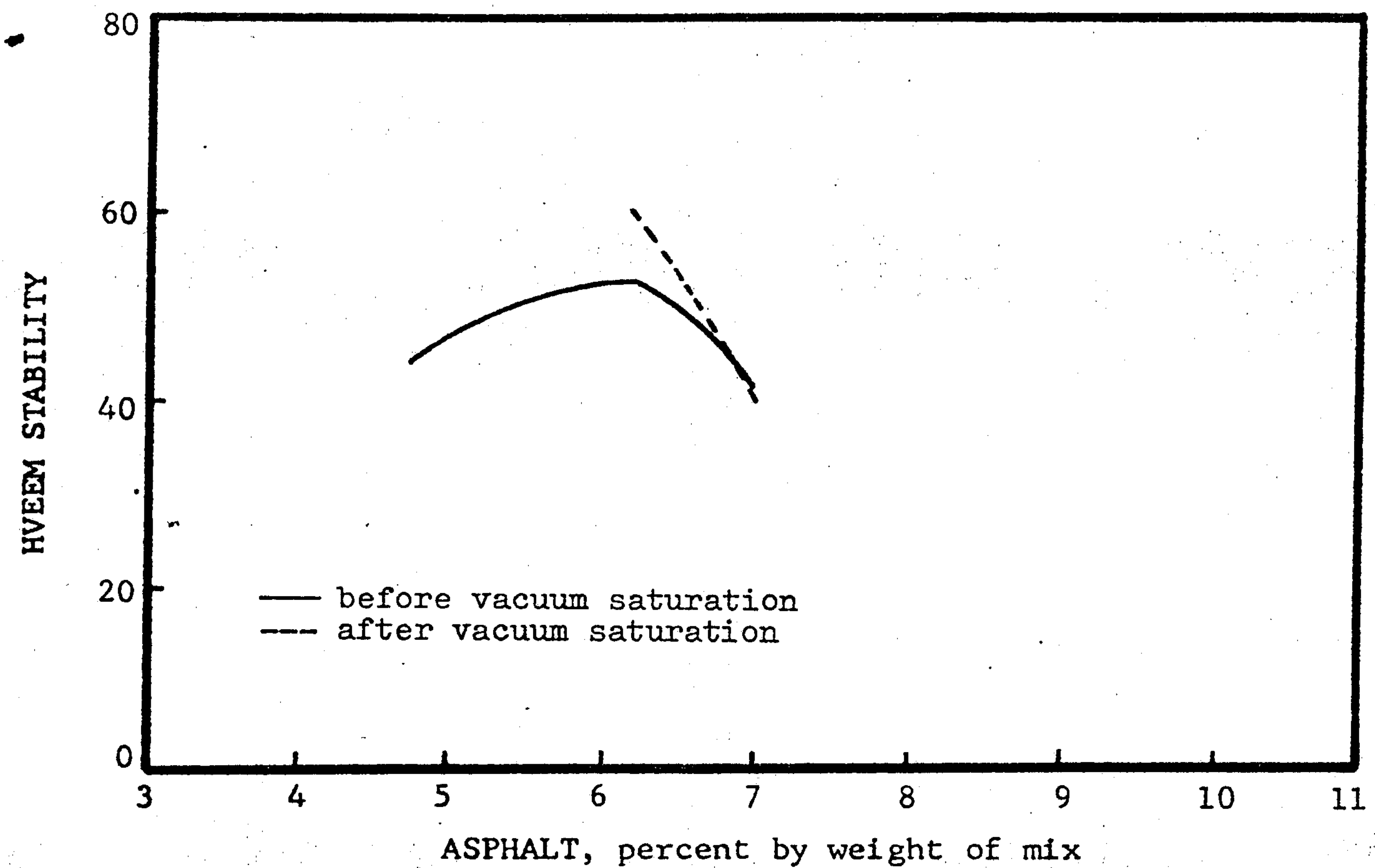

Figure 70. Effect of vacuum saturation on the Hveem stability of crushed caliche gravel mixtures.

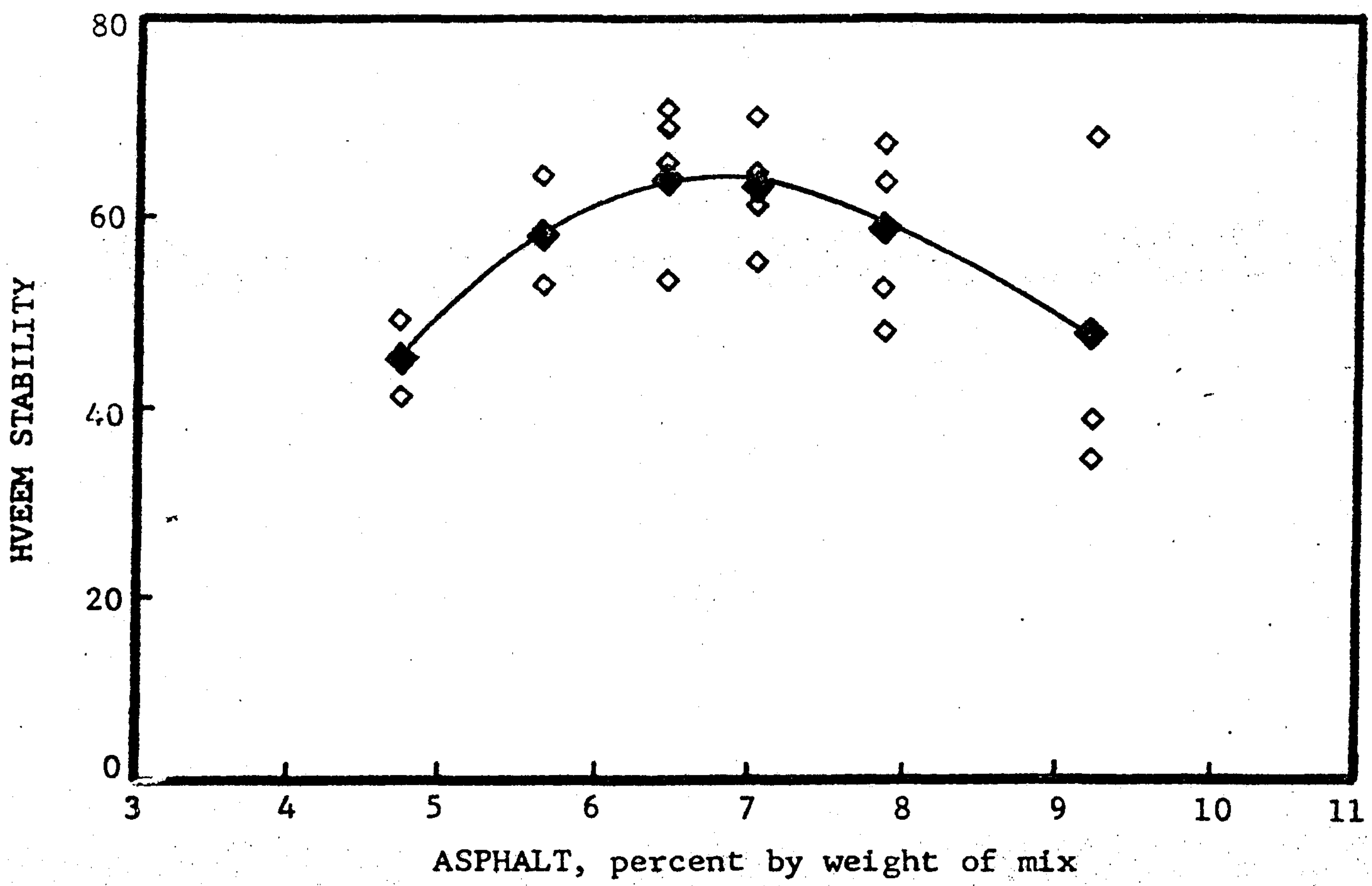

Figure 71. Hveem stability versus asphalt content for crushed sandstone mixtures (before vacuum saturation).

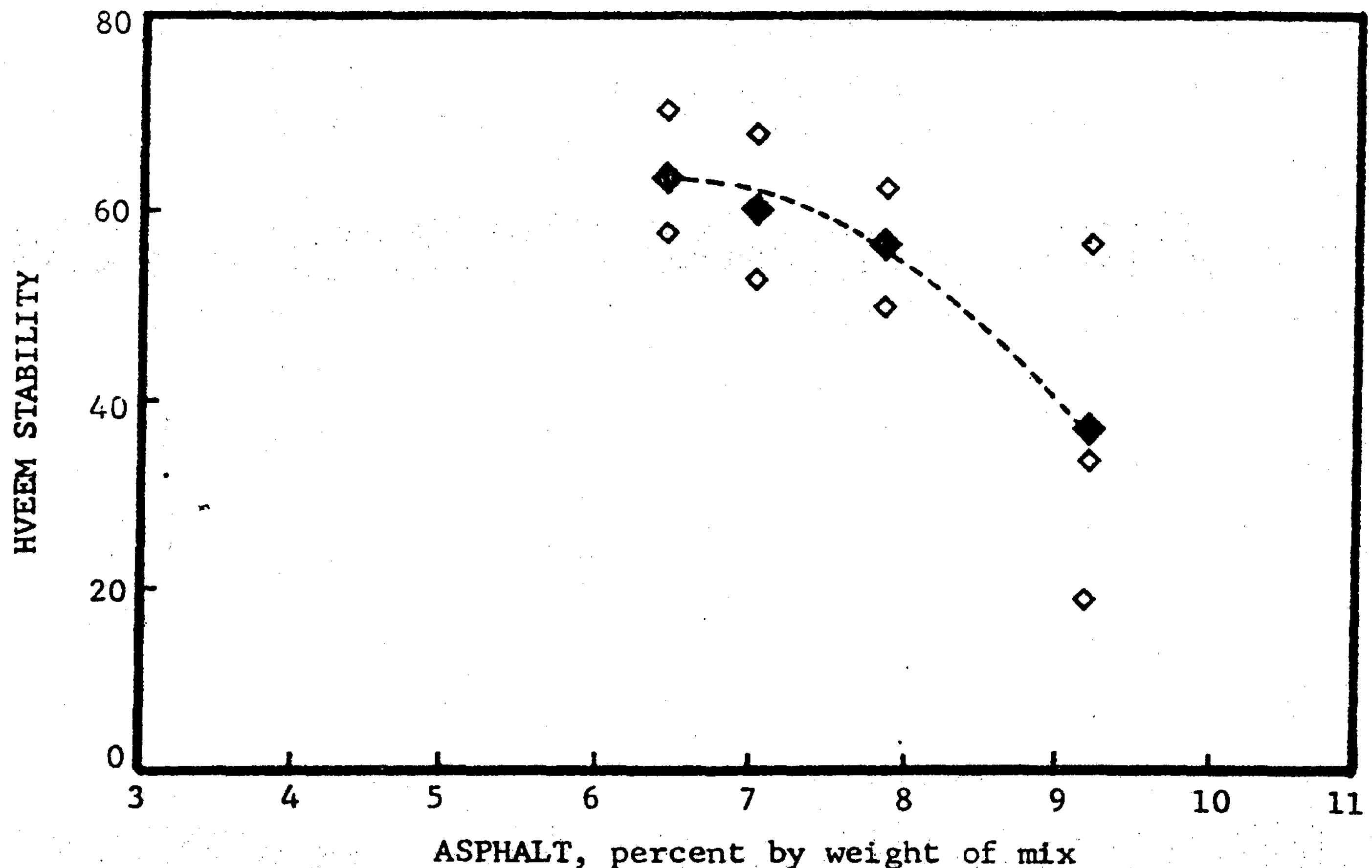

Figure 72. Hveem stability versus asphalt content for crushed sandstone mixtures (after vacuum saturation).

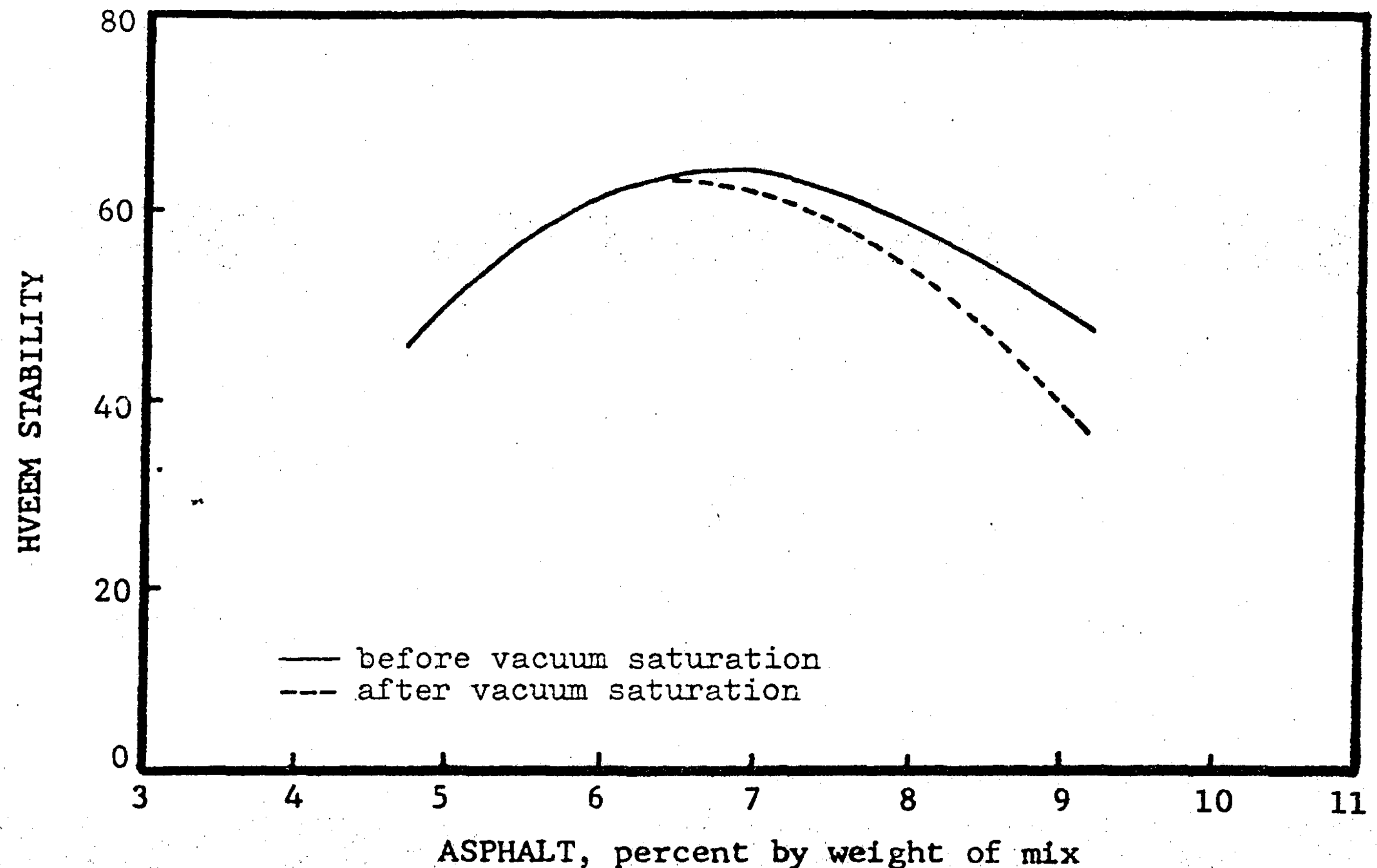

Figure 73. Effect of vacuum saturation on the Hveem stability of crushed sandstone mixtures.

a reduction in stability after vacuum saturation.

Figure 74 illustrates the effect of asphalt content on the stability of the Austin chalk mixtures. Although a break in the stability curve does not exist, it seems reasonable to assume that the maximum stability value occurs at or near 8.0 percent asphalt. The only Austin chalk sample suitable for testing after vacuum saturation is shown on Figure 75.

Livneh and Halpern (44) have investigated the effect of water action on bituminous mixtures using Marshall stability criteria and the immersion test, ASTM D 1075 (20). Livneh and Halpern contend that although the fines (material passing number 10 sieve) contribute to the stability of the mixture by reducing the percentage of voids and by stiffening the bitumen films that coat the aggregate, the fines may introduce sensitivity to water. The immersion compression tests indicated that soaking in water weakened the specimens. The authors conclude that the retained strength of the bituminous mixtures is a function of the amount and quality of the fines in the mix and that the optimum fines content is larger with the higher quality fines.

Tensile Properties

The splitting tensile test was the final item in the testing of the laboratory molded black base mixtures. As explained earlier, the 6-inch diameter by 8-inch height black base samples were sliced and cored into four specimens four inches in diameter and approximately two inches in height. Of these four specimens, two were subjected to a testing program which included a vacuum saturation procedure and two specimens were subjected to the same program but without vacuum saturation. After failing the four specimens by indirect tension, a comparison could be made

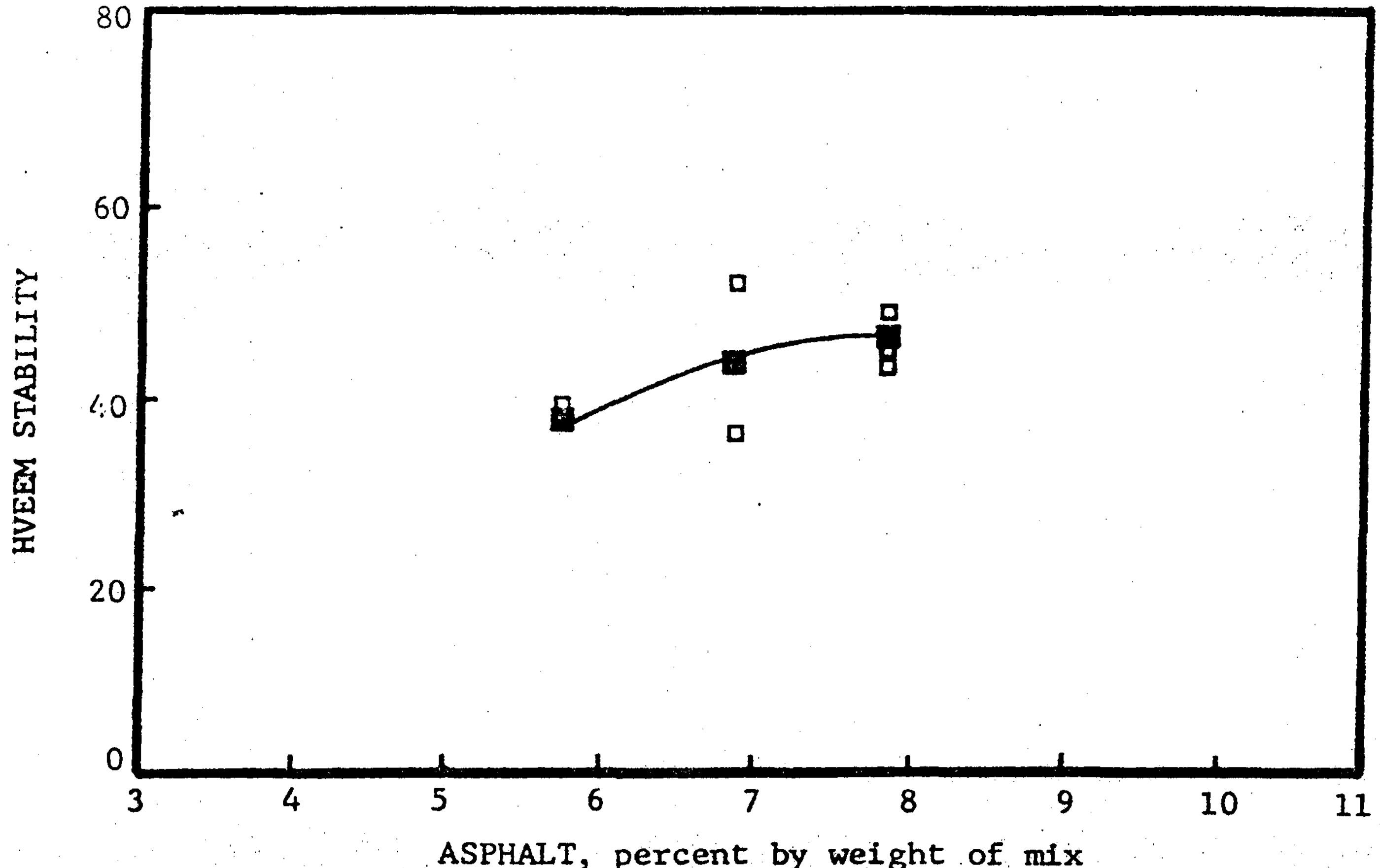

Figure 74. Hveem stability versus asphalt content for Austin chalk mixtures (before vacuum saturation).

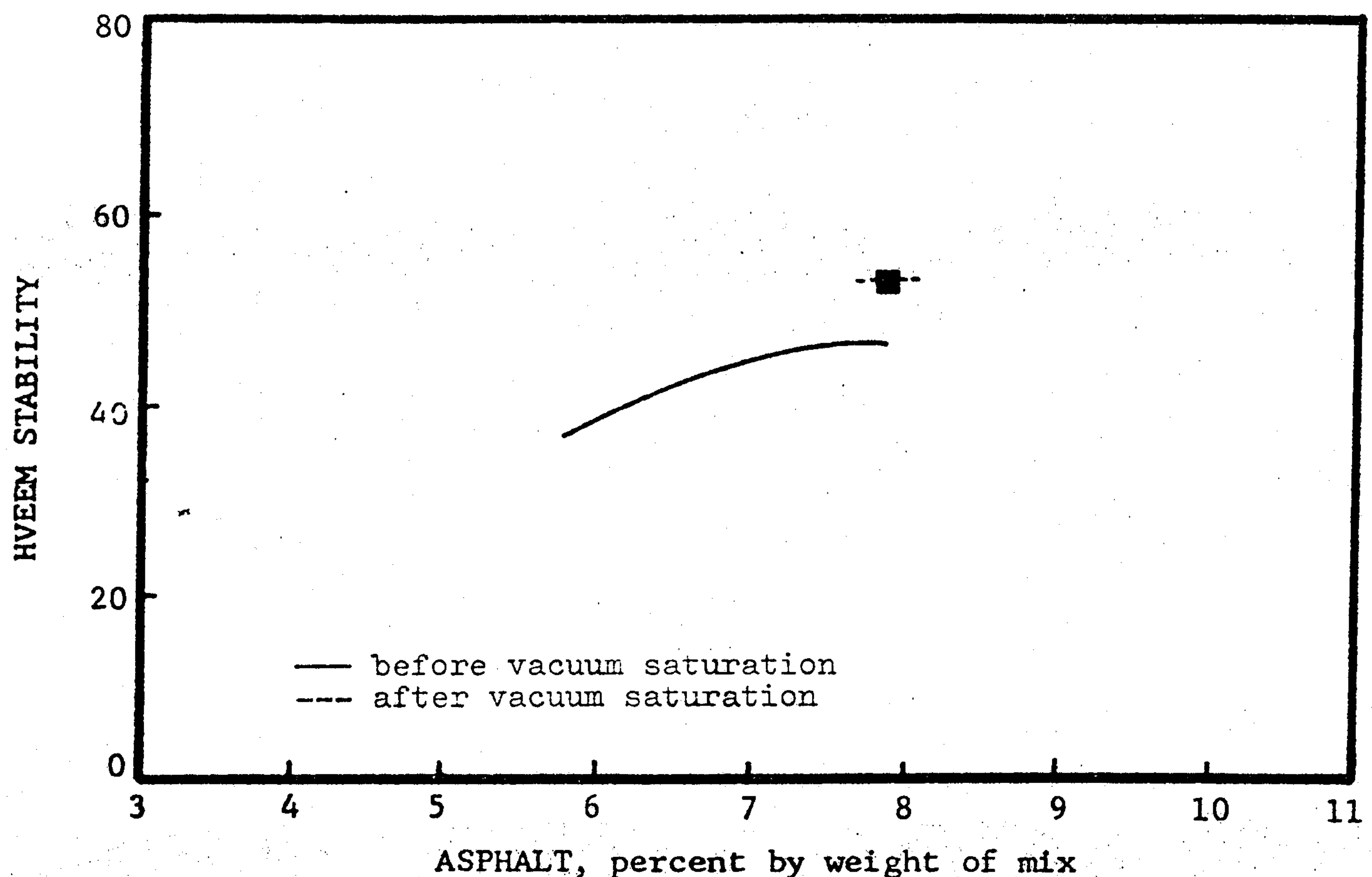

Figure 75. Effect of vacuum saturation on the Hveem stability of Austin chalk mixtures.

which would illustrate the effect of vacuum saturation on the modulus of elasticity at failure. It must be noted that contrary to the Schmidt testing procedure in which the vacuum saturated samples were allowed to dry to a constant weight (at 73°F and 25 percent relative humidity) before testing.

Figures 76 through 90, illustrate the effect of vacuum saturation on the modulus of elasticity of black base mixtures at 73°F as determined by the splitting tensile test. The figures demonstrate that the moduli values are somewhat reversible; that is, although the resilient moduli were considerably lower when tested in the saturated condition by the Schmidt test, the values of the saturated samples were generally consistent with those not saturated when tested in splitting tension. This phenomenon of "reversibility" was also noted by Schmidt (43).

Figure 78 compares the sandstone samples failed in splitting tension which were not vacuum saturated to those which were vacuum saturated. Interestingly, it is noted that a slightly lower "optimum" asphalt content would be determined from the vacuum saturated curve. Figure 81 illustrates an increase in modulus of elasticity for the vacuum saturated limestone samples. Figure 84 appears to be lacking in sufficient data so as to report reliable conclusions about the splitting tensile test on the crushed caliche gravel samples. Likewise, Figures 87 and 90 appear to be lacking in sufficient information to report reliable conclusions for the crushed sandstone and Austin chalk samples.

The computer program utilized for analyzing the splitting tensile data may be found in reference 44.

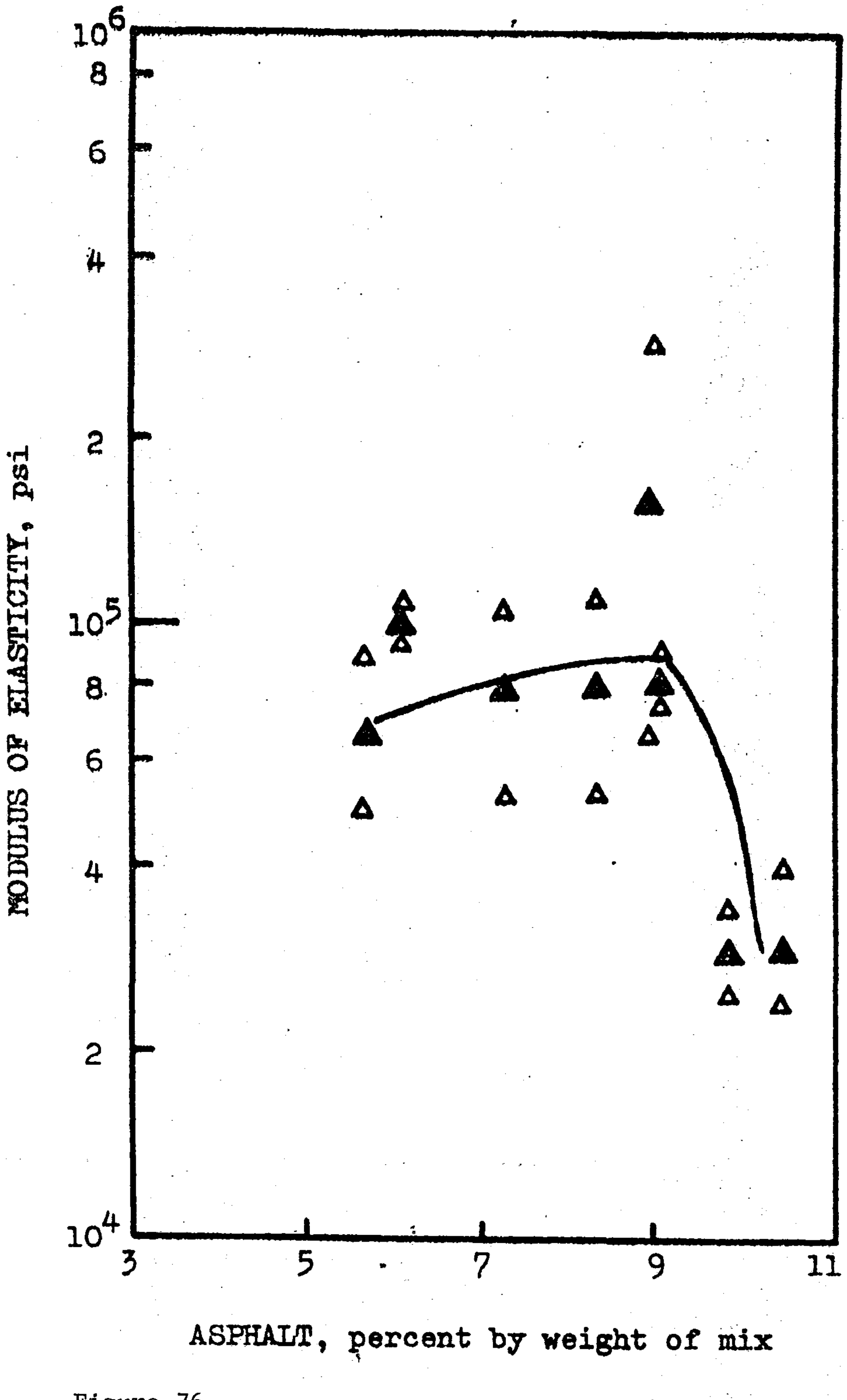

Figure 76. Modulus of elasticity (by splitting tensile test) versus asphalt content for sandstone samples (not vacuum saturated).

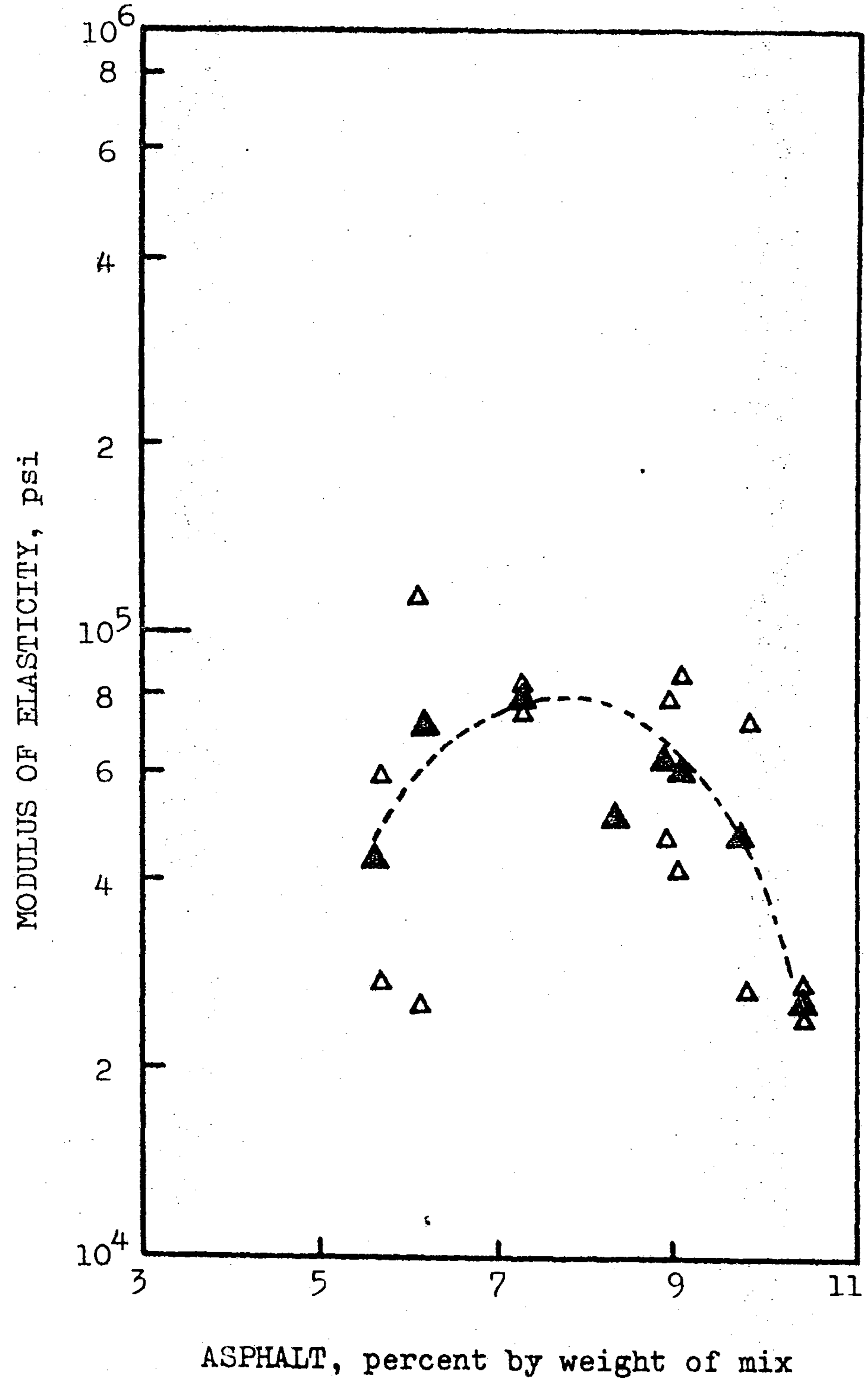

Figure 77. Modulus of elasticity (by splitting tensile test) versus asphalt content for sandstone samples (after vacuum saturation).

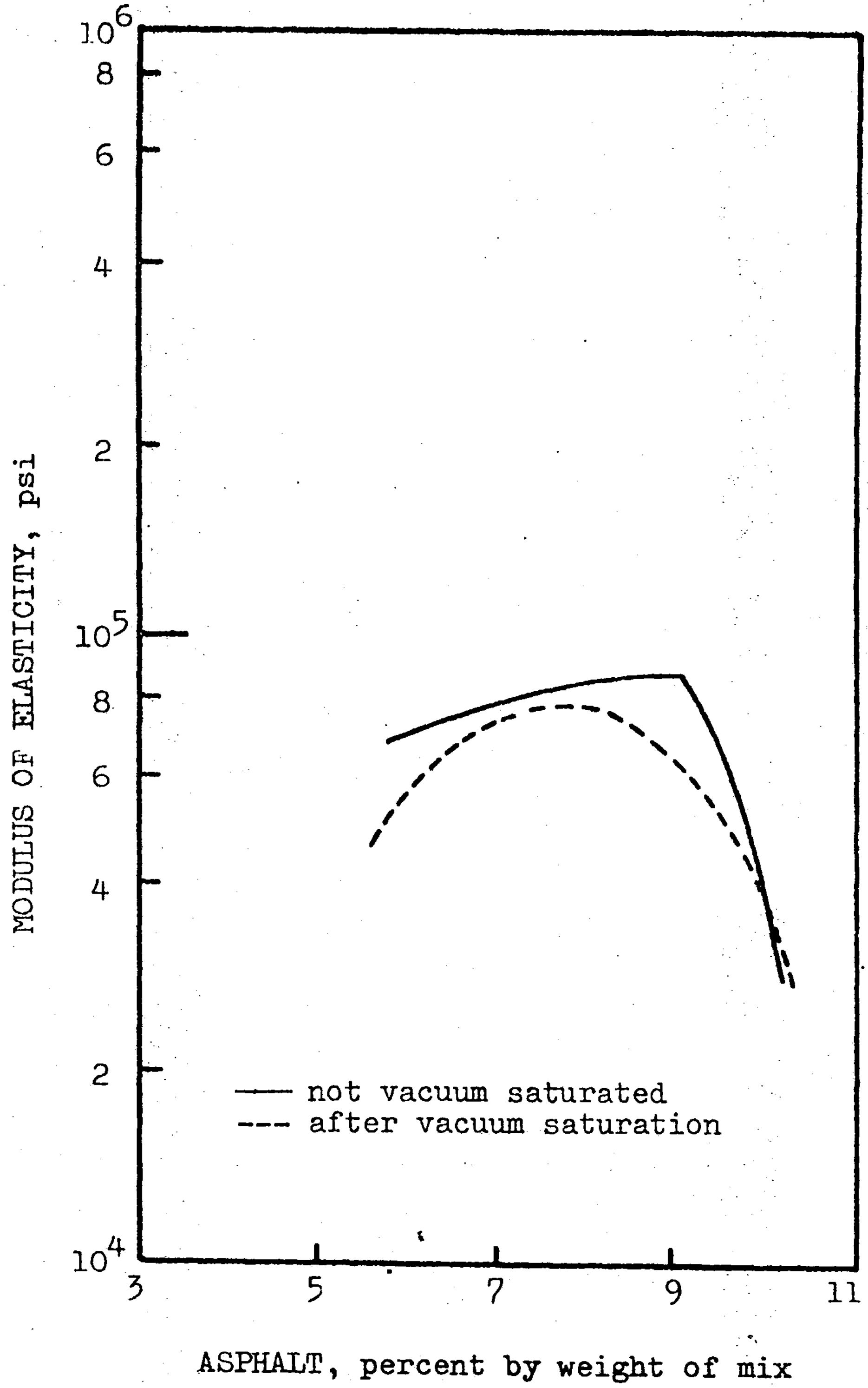

Figure 78. Effect of vacuum saturation on modulus of elasticity (by splitting tensile test) on sandstone samples.

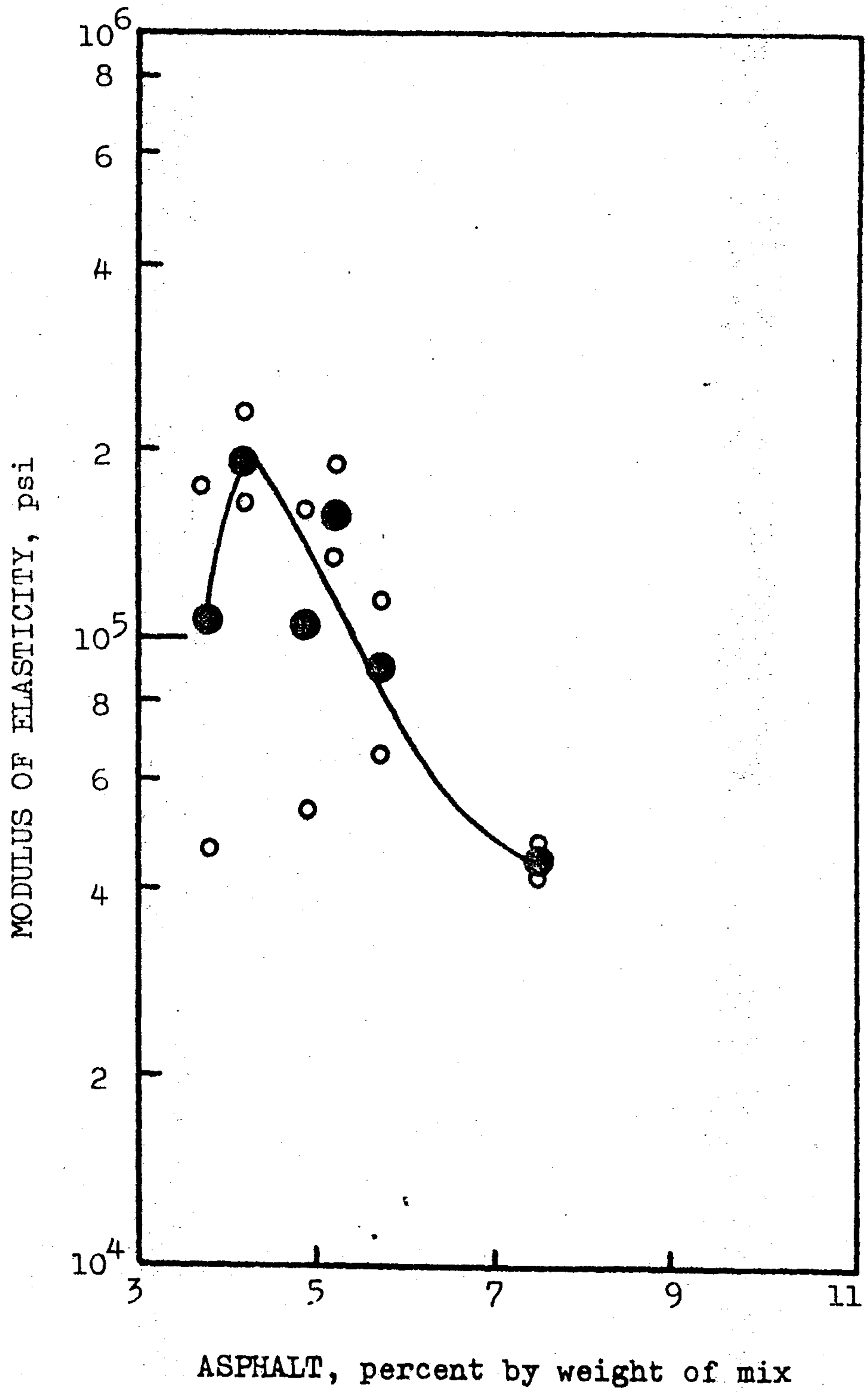

Figure 79. Modulus of elasticity (by splitting tensile test) versus asphalt content for limestone samples (not vacuum saturated).

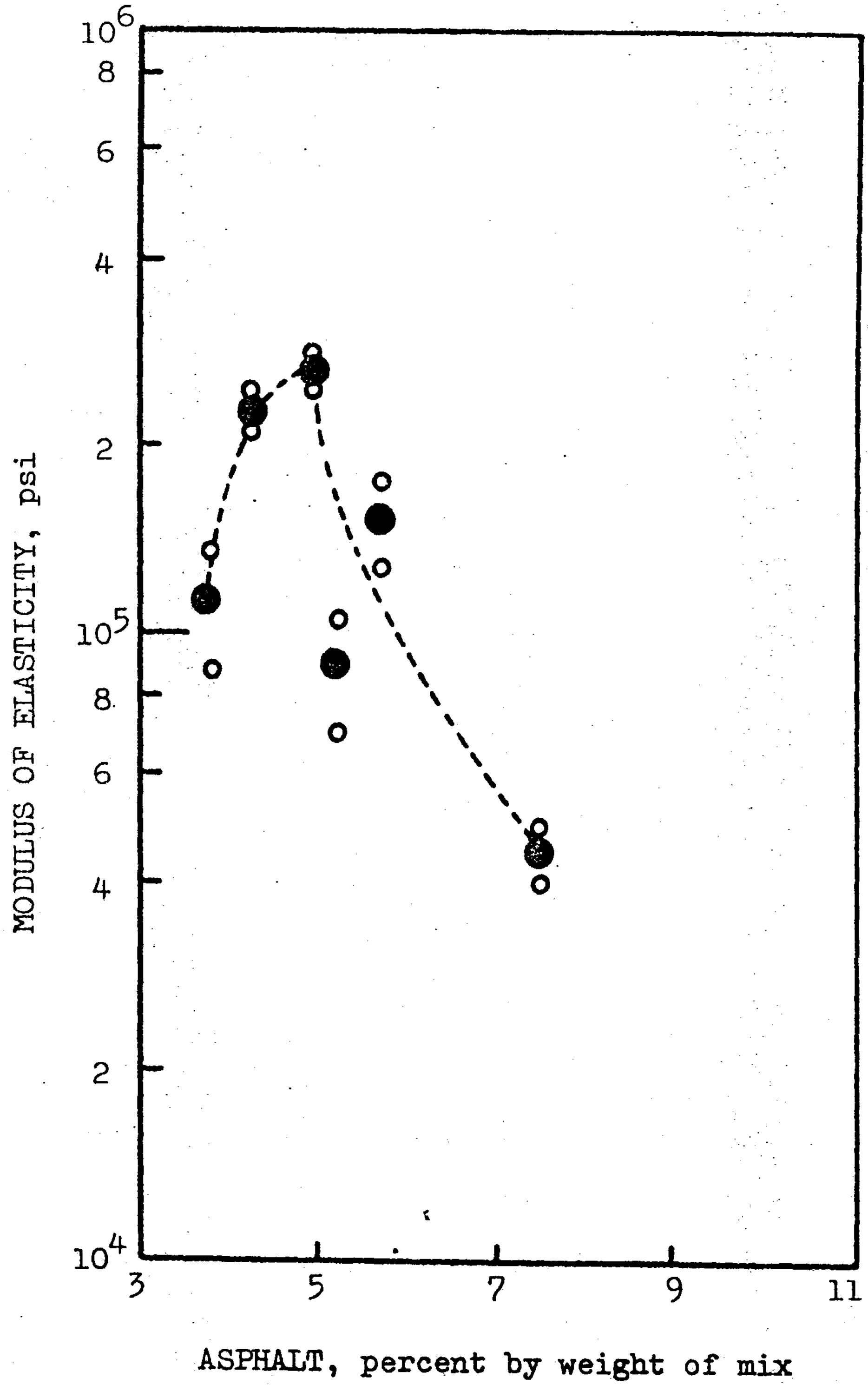

Figure 80. Modulus of elasticity (by splitting tensile test) versus asphalt content for limestone samples (after vacuum saturation).

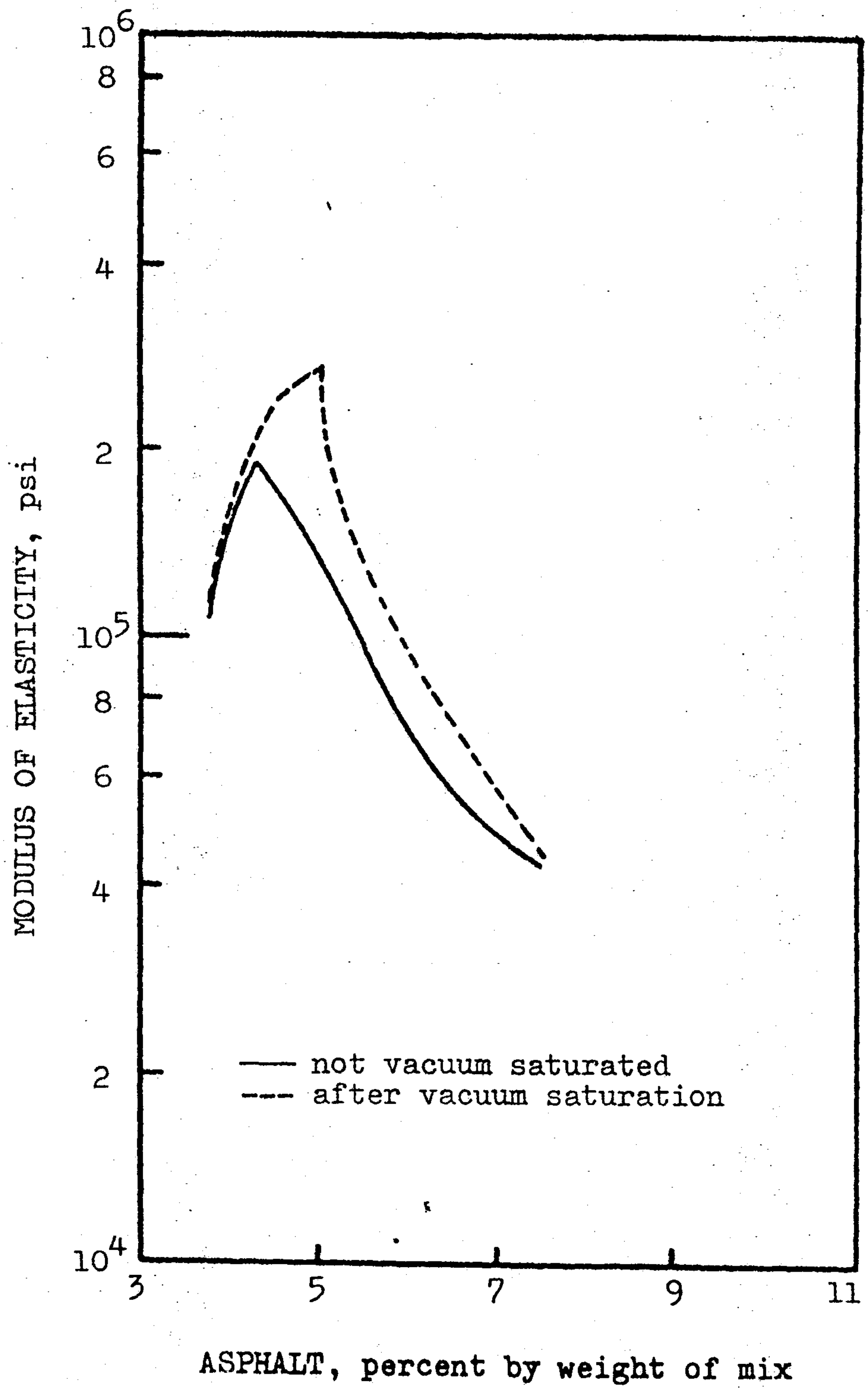

Figure 81. Effect of vacuum saturation on modulus of elasticity (by splitting tensile test) on limestone samples.

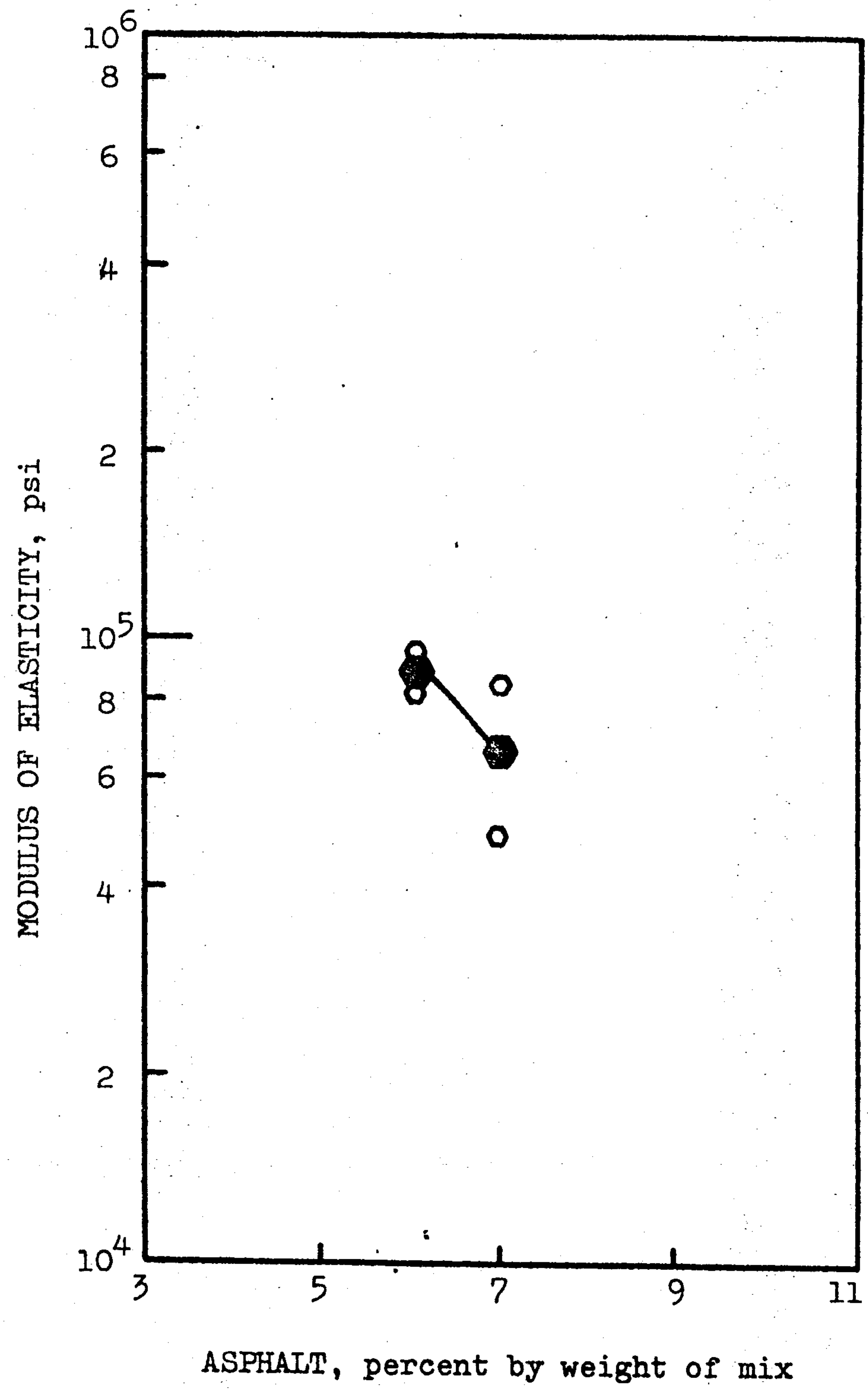

Figure 82. Modulus of elasticity (by splitting tensile test) versus asphalt content for crushed caliche gravel samples (not vacuum saturated).

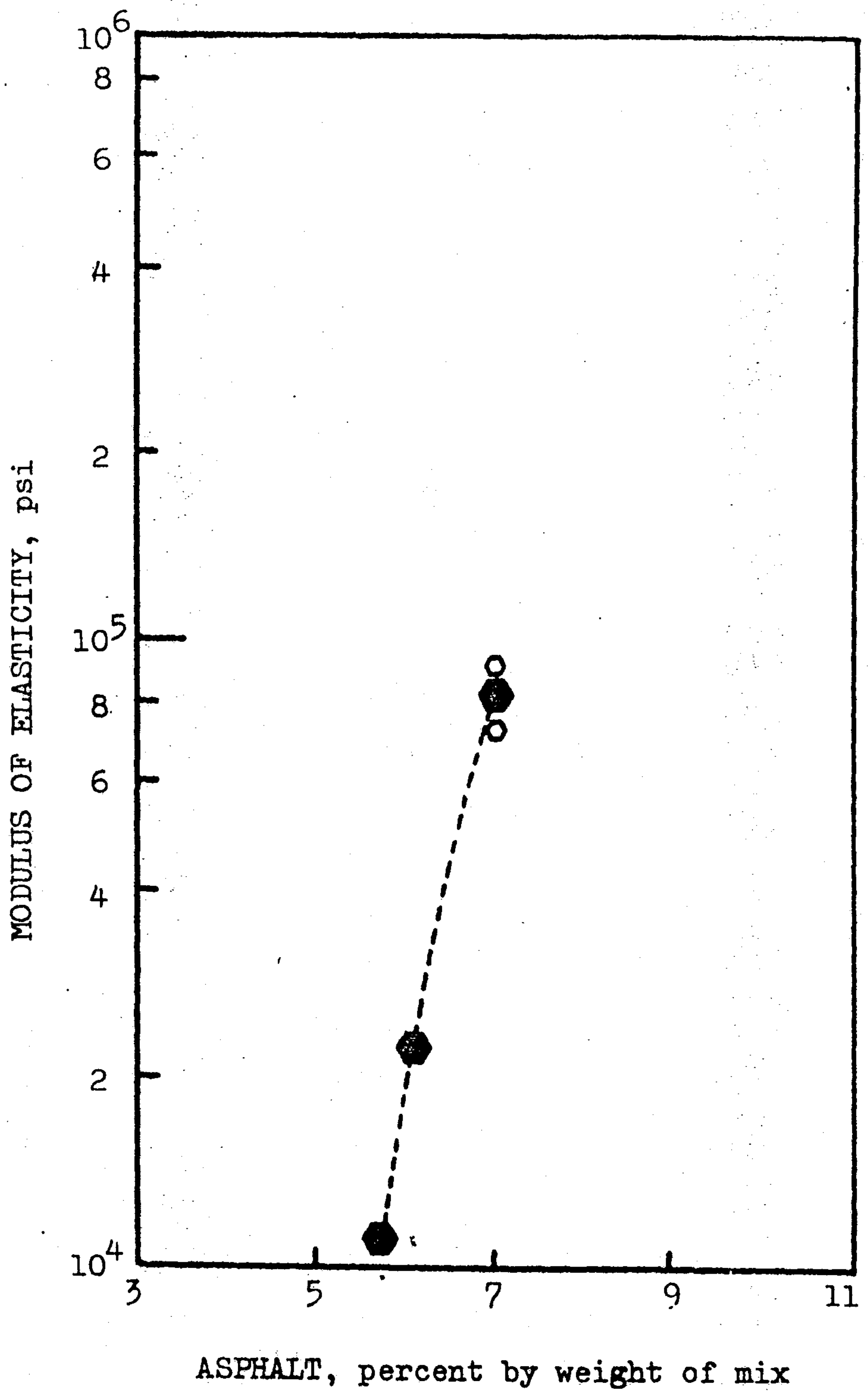

Figure 83. Modulus of elasticity (by splitting tensile test) versus asphalt content for crushed caliche gravel samples (after vacuum saturation).

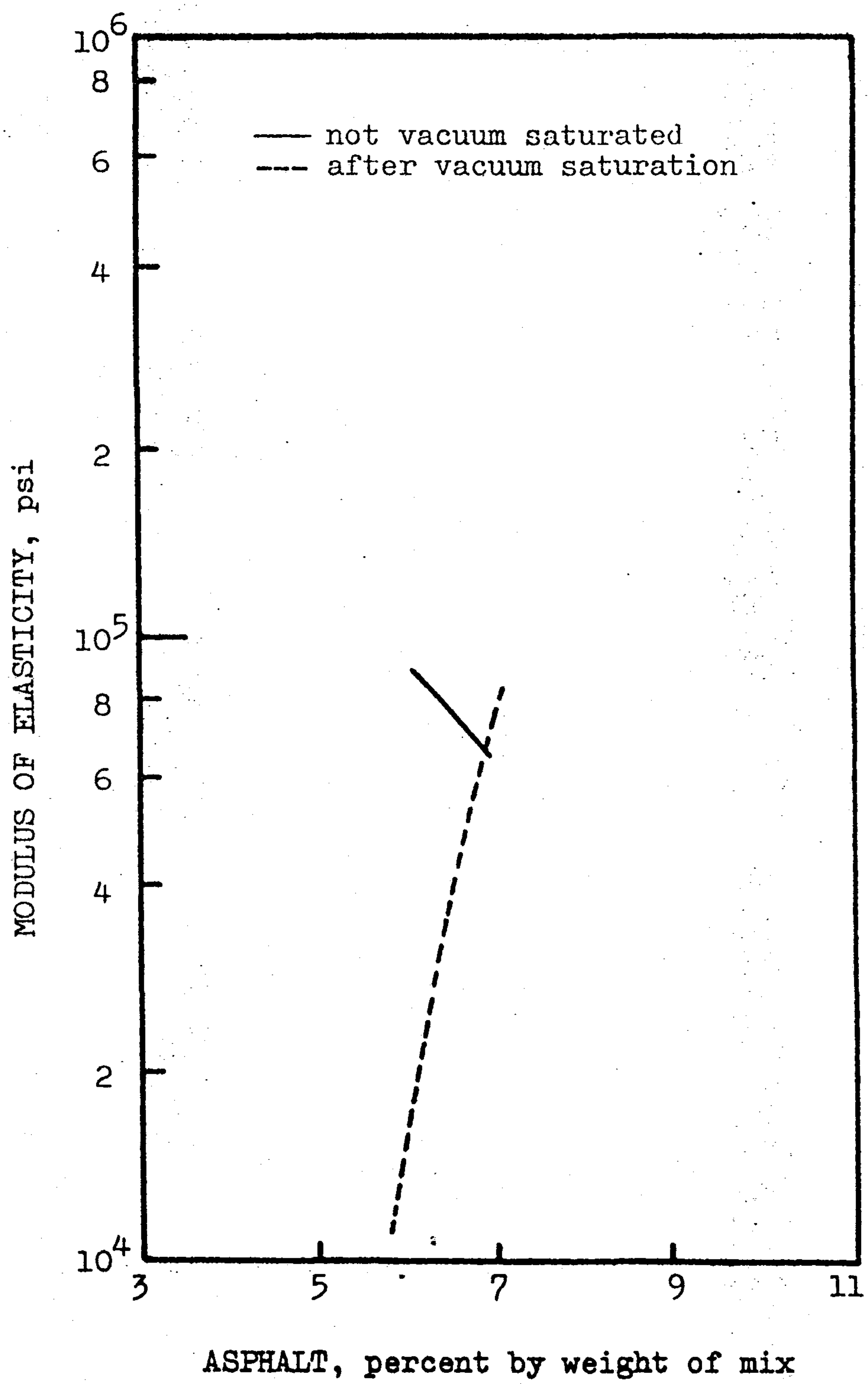

Figure 84. Effect of vacuum saturation on modulus of elasticity (by splitting tensile test) on crushed caliche gravel samples.

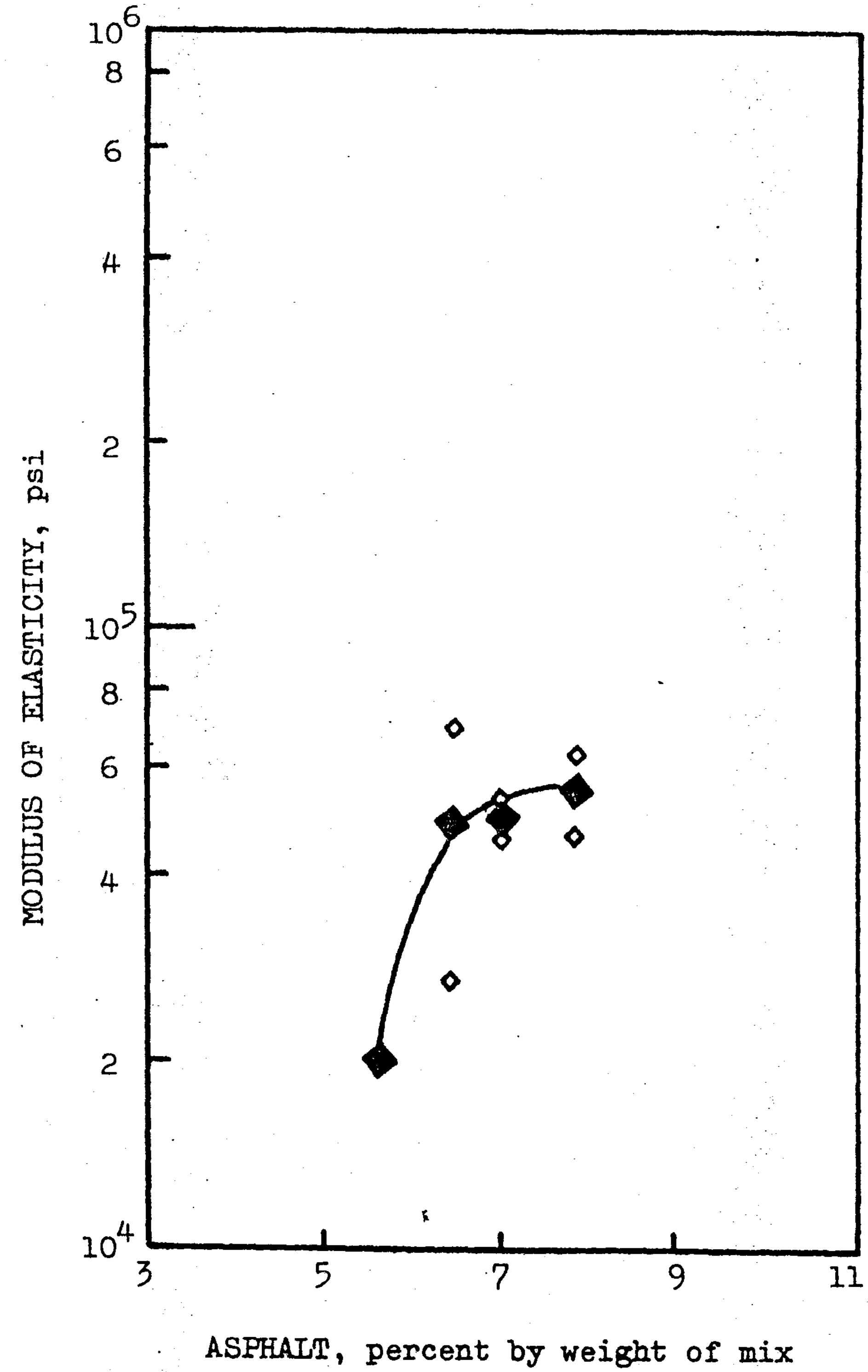

Figure 85. Modulus of elasticity (by splitting tensile test) versus asphalt content for crushed sandstone samples (not vacuum saturated).

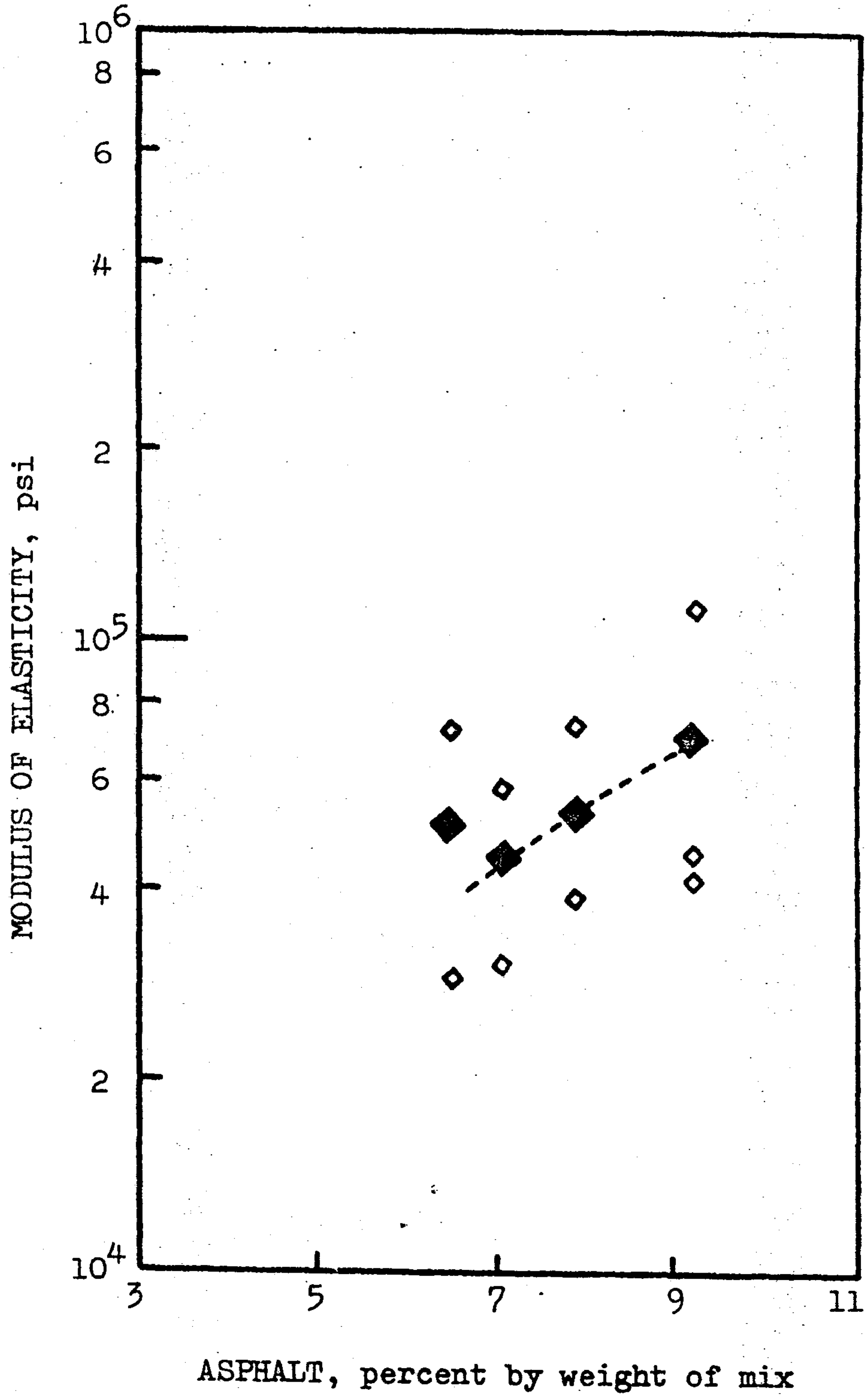

Figure 86. Modulus of elasticity (by splitting tensile test) versus asphalt content for crushed sandstone samples (after vacuum saturation).

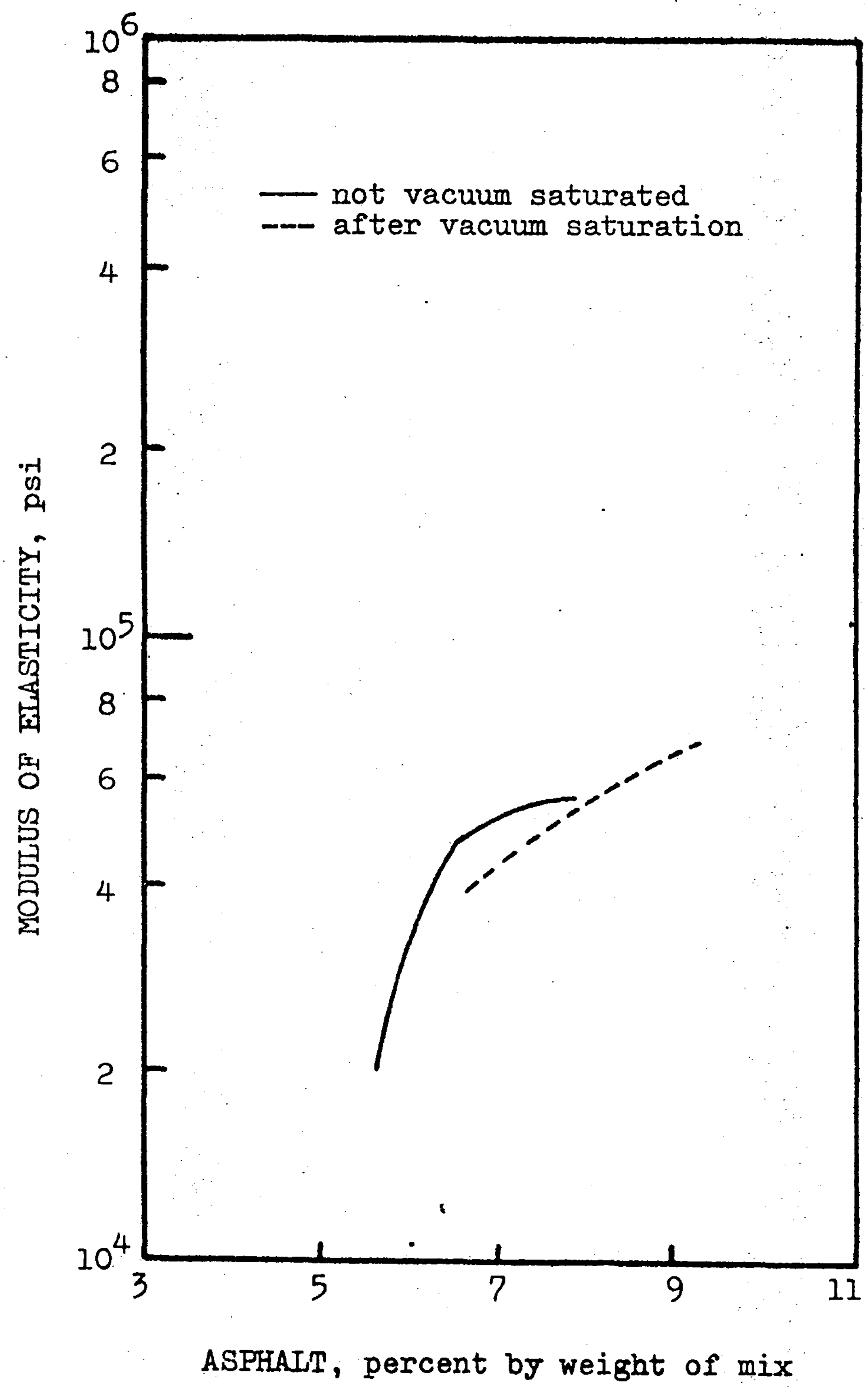

Figure 87. Effect of vacuum saturation on modulus of elasticity (by splitting tensile test) on crushed sandstone samples.

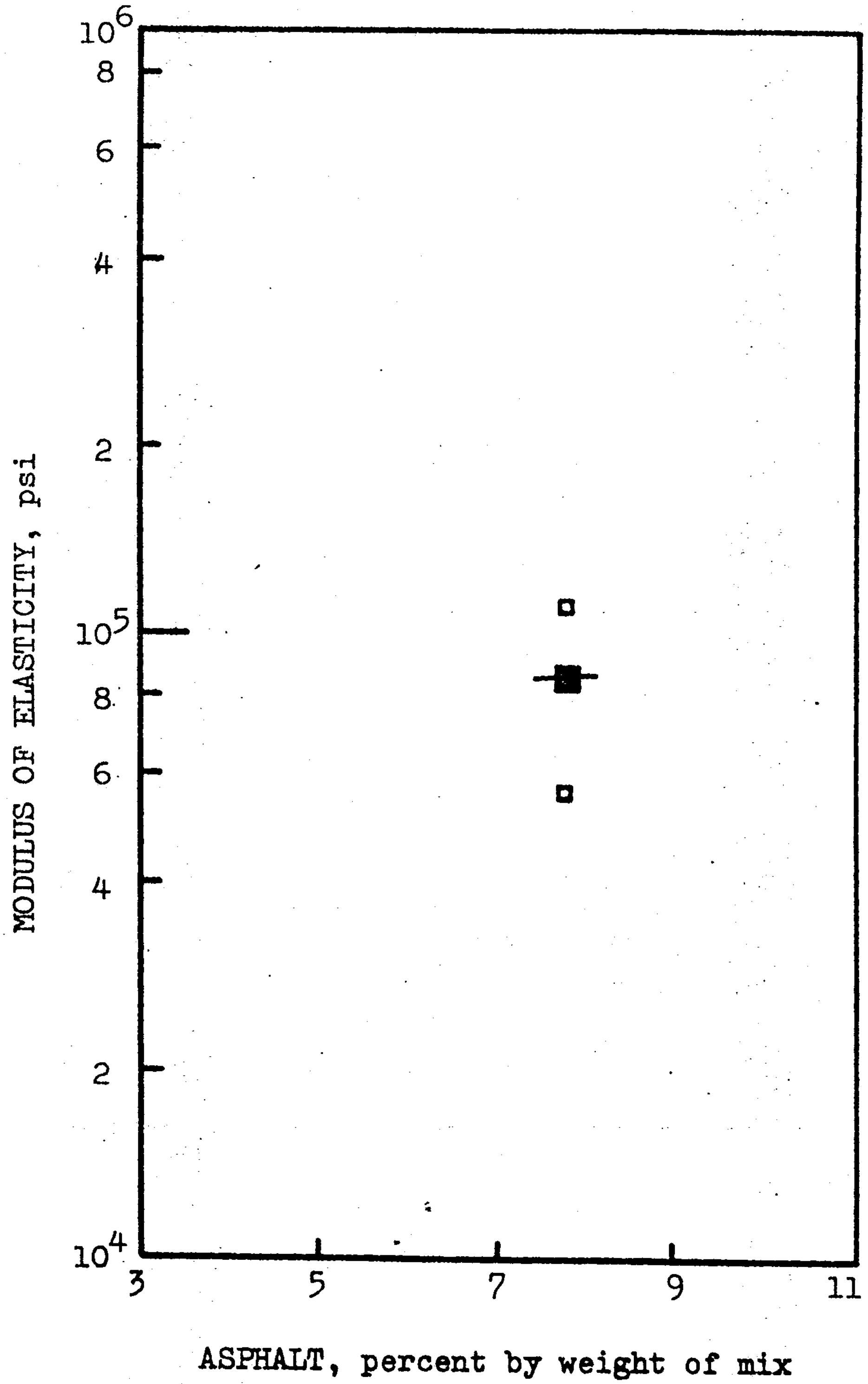

Figure 88. Modulus of elasticity (by splitting tensile test) versus asphalt content for Austin chalk samples (not vacuum saturated).

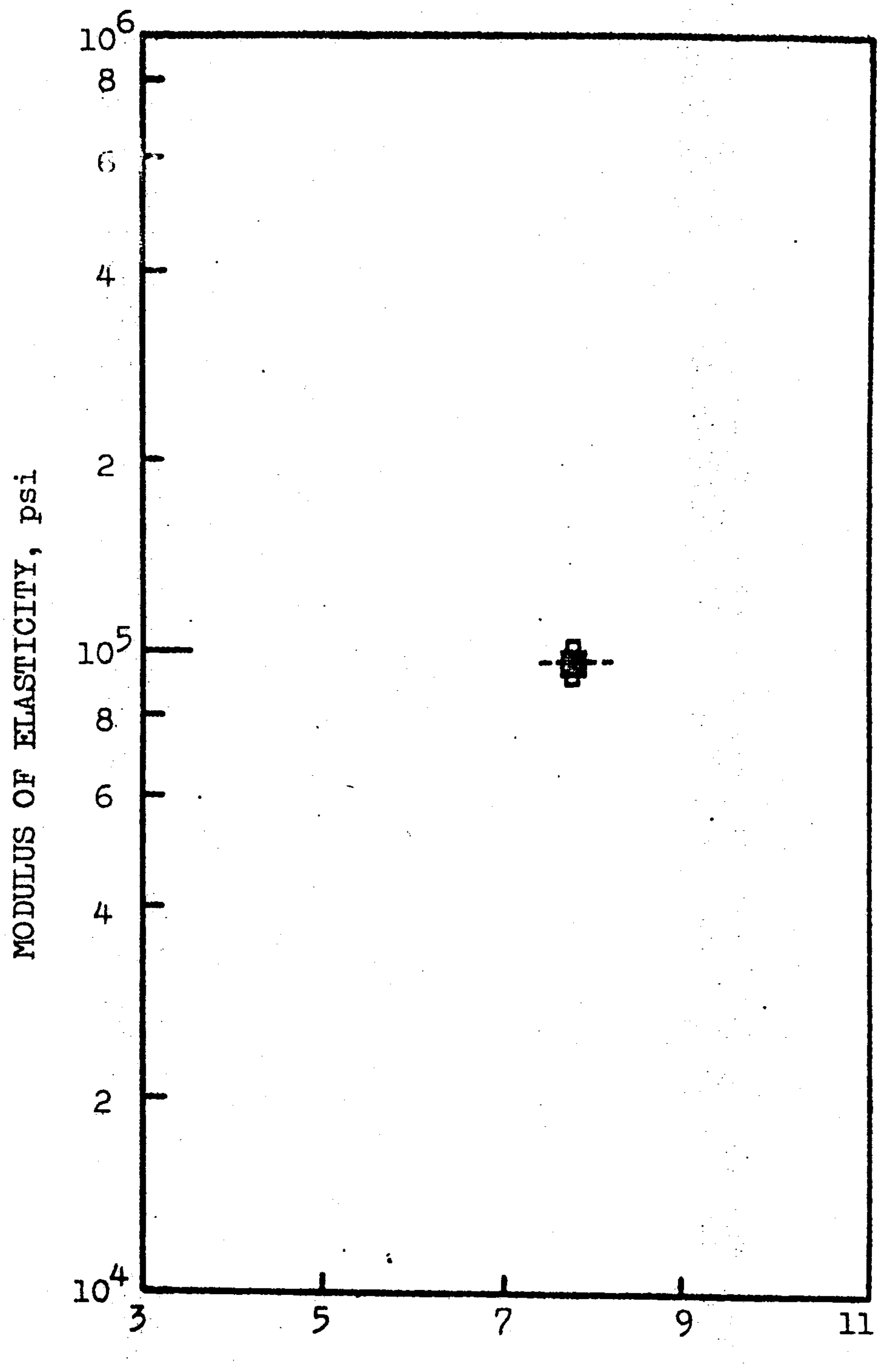

Figure 89. Modulus of elasticity (by splitting tensile test) versus asphalt content for Austin chalk samples (after vacuum saturation).

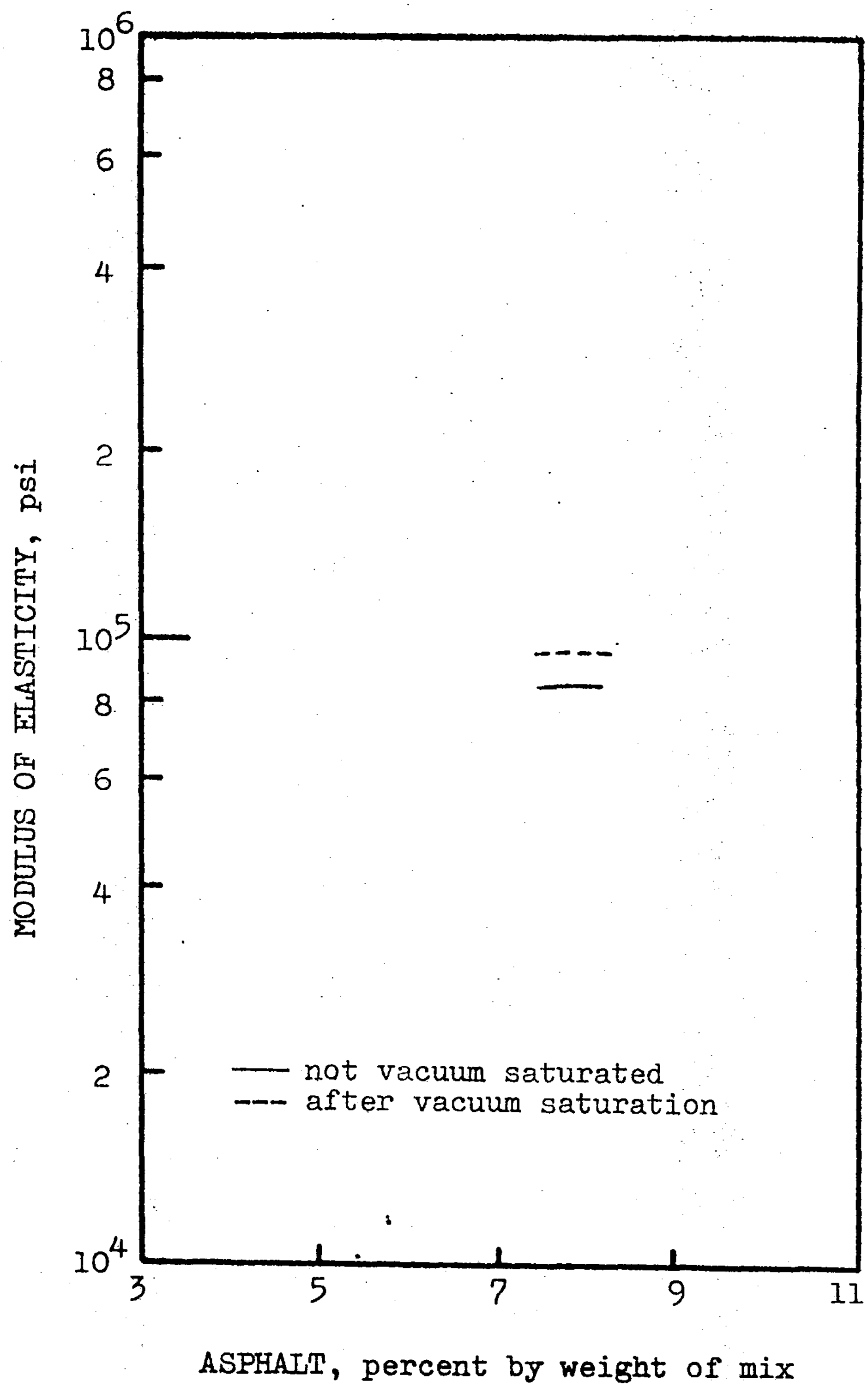

Figure 90. Effect of vacuum saturation on modulus of elasticity (by splitting tensile test) on Austin chalk samples.

Determination of Optimum Asphalt Content

The laboratory molded black base mixtures were investigated through an extensive testing program. From several of the tests, an optimum asphalt content was determined as illustrated in Table 4.

The eleven procedures illustrated in Table 4 generally report consistent optimum asphalt contents. The procedure used by many states which requires an air voids content to be between three and seven percent indicates generally lower optimum asphalt contents than those indicated by the Texas method. The highest optimum asphalt contents reported were determined by the resilient modulus test procedure using after vacuum saturation results. This is because the vacuum saturation procedure creates the most severe condition to which black base mixture would be exposed. The procedure, however, does not report asphalt contents which are much higher than those reported by many of the other tests.

Realizing problems associated with black base mixtures (stripping, transverse cracking pattern etc.) it appears that the vacuum saturation procedure might be the most reliable method available for determining optimum asphalt contents. The Texas method of test Tex-126-E (26), for example, requires that the black base samples be molded, allowed to cool to 140°F and then be subjected to pressure wetting with hot water. Texas State Department of Highways and Public Transportation personnel have been well pleased with Tex-126-E. Recent black stripping problems indicate a need for more extensive materials evaluations, particularly when investigating marginal aggregate mixtures.

For roadways with expected lower traffic volumes, it seems reasonable

Table 4. Determination of optimum asphalt contents.

Black Base Mixture	Compacted Specific Gravity	Air Voids, 3% - 7% Method	Air Voids, Texas Method	M_R Before Vacuum Saturation (6" x 2") (73°F)	M_R Before Vacuum Saturation (4" x 2") (73°F)	M_R After Vacuum Saturation (73°)	M_R Before Vacuum Saturation (34°F)	M_R Before Vacuum Saturation (100°F)	Maximum Hveem Stability After Vacuum Saturation	Splitting Tensile Samples Not Vacuum Saturated (73°F)	Splitting Tensile After Vacuum Saturation (73°F)
Sandstone	9.0	8.0	9.0	7.5	8.0	9.0	10.0	8.0	7.5	9.0	8.5
Limestone	5.0	4.5	5.0	4.5	4.3	4.5	5.0	5.0	4.0	4.5	5.0
Crushed Caliche Gravel	above 7.0	6.5	7.0	6.5	7.0	7.5	7.5	7.5	6.0		above 7.0
Crushed Sandstone	9.5	above 7.5	9.0	8.0	9.0	9.5	9.0	9.0	6.0	8.0	above 9.0
Austin Chalk	8.0	above 7.0	8.0	8.5	above 8.0	9.5	above 8.0	above 8.0	8.0		

that the black base layer could be designed and placed at an asphalt content lower than optimum. The use of a lower than optimum asphalt content in black base mixtures is related to both strength characteristics and economic considerations. Layered elastic pavement design computer analyses have been developed (45, 46) which enable an engineer to input the expected traffic volumes and loading conditions to determine layer thicknesses. Combining these results with current highway cost information, the most economical pavement section may be determined. This approach is illustrated in another section of this report.

LABORATORY TEST RESULTS - SANDS AND SAND GRAVELS

Testing Program

An outline of the testing program utilized for the sands and sand gravel mixtures is shown in Figure 91. This testing program is very similar to that utilized for the dense graded aggregate mixtures discussed above. Resilient modulus, Hveem stability and Marshall stability valves were obtained in this testing program both before and after vacuum saturation and soaking.

Specific Gravity and Air Voids Content

Figures 92 through 95 illustrate the relationship between compacted specific gravity and asphalt content for the laboratory molded mixtures at asphalt contents of 4.5 and 6 percent. The samples were compacted by use of the standard Texas gyratory method for 4-inch diameter samples (21). The range of asphalt content was not sufficient to determine the asphalt content provided maximum density.

Figures 96 through 99 illustrate the relationship between percent air voids and asphalt content. Air void contents for the sand mixes as expected were in excess of 5 percent at 6 percent asphalt. Considerably lower air void contents were obtained with the Hidalgo County sand-gravel.

Asphalt contents of the order of 10 percent or more would be required to reduce air voids to a level which would be acceptable by most specifications. However, successful field use of sands of this type have indicated that the high air void contents can be tolerated provided the mixture is utilized for a purpose compatible with its properties.

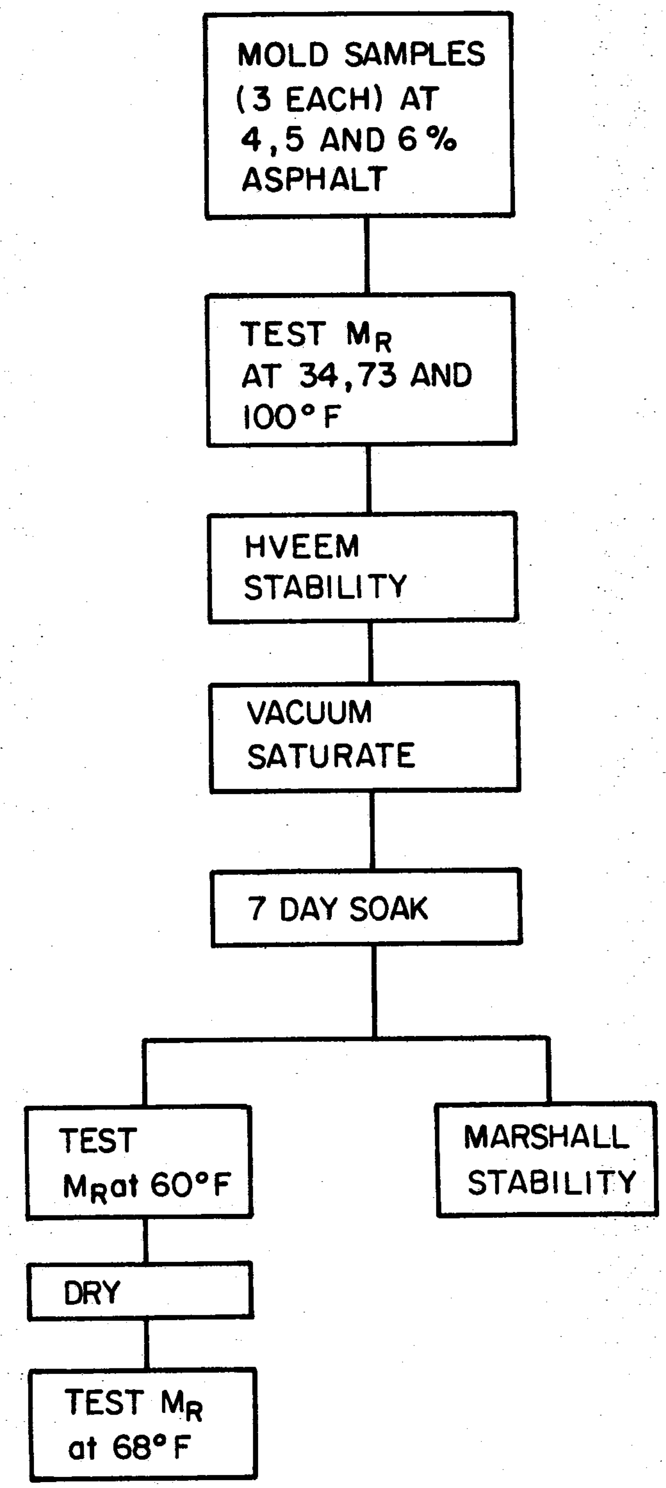

Figure 91. Laboratory Testing Procedure for Sand Aggregate Black Base Specimens.

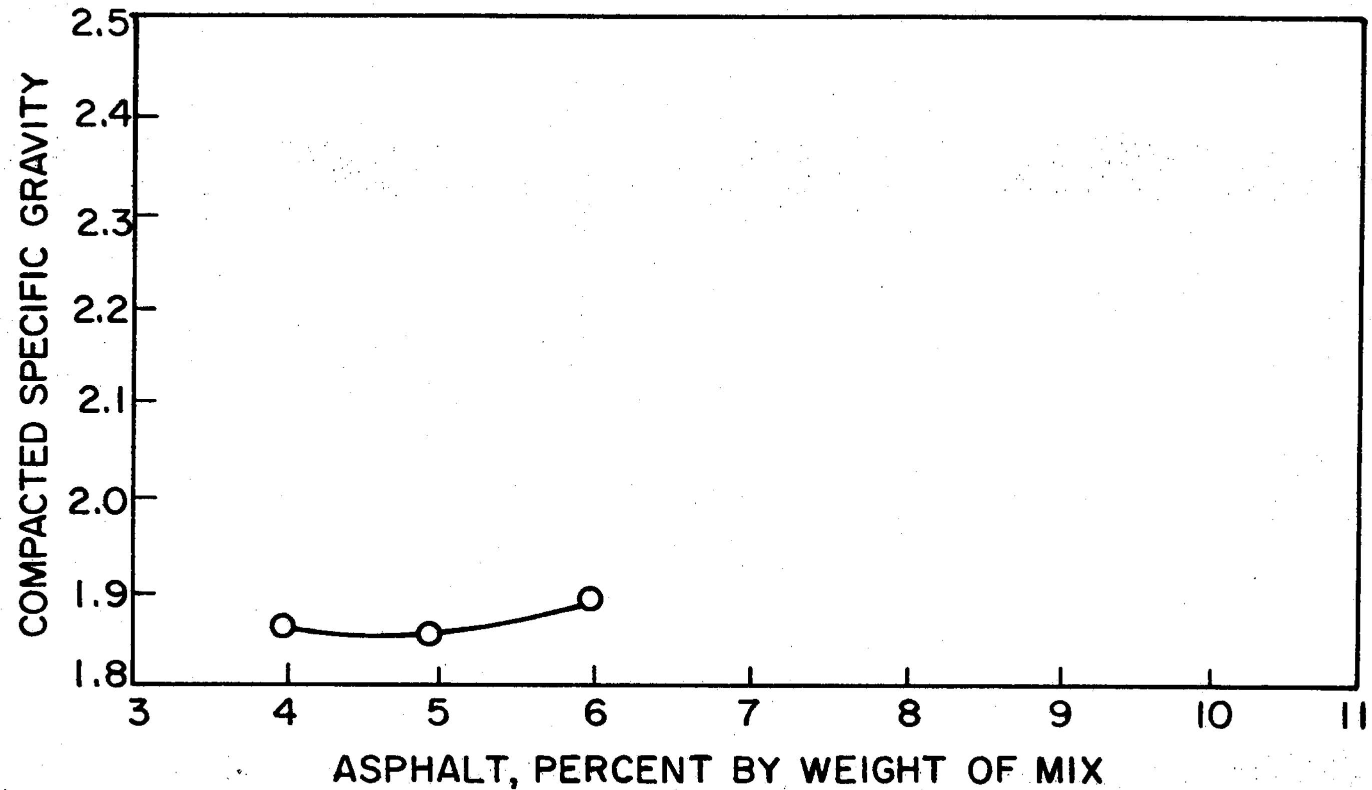

Figure 92. Compacted specific gravity versus asphalt content for Lamb County Sand (District 5 FM 168).

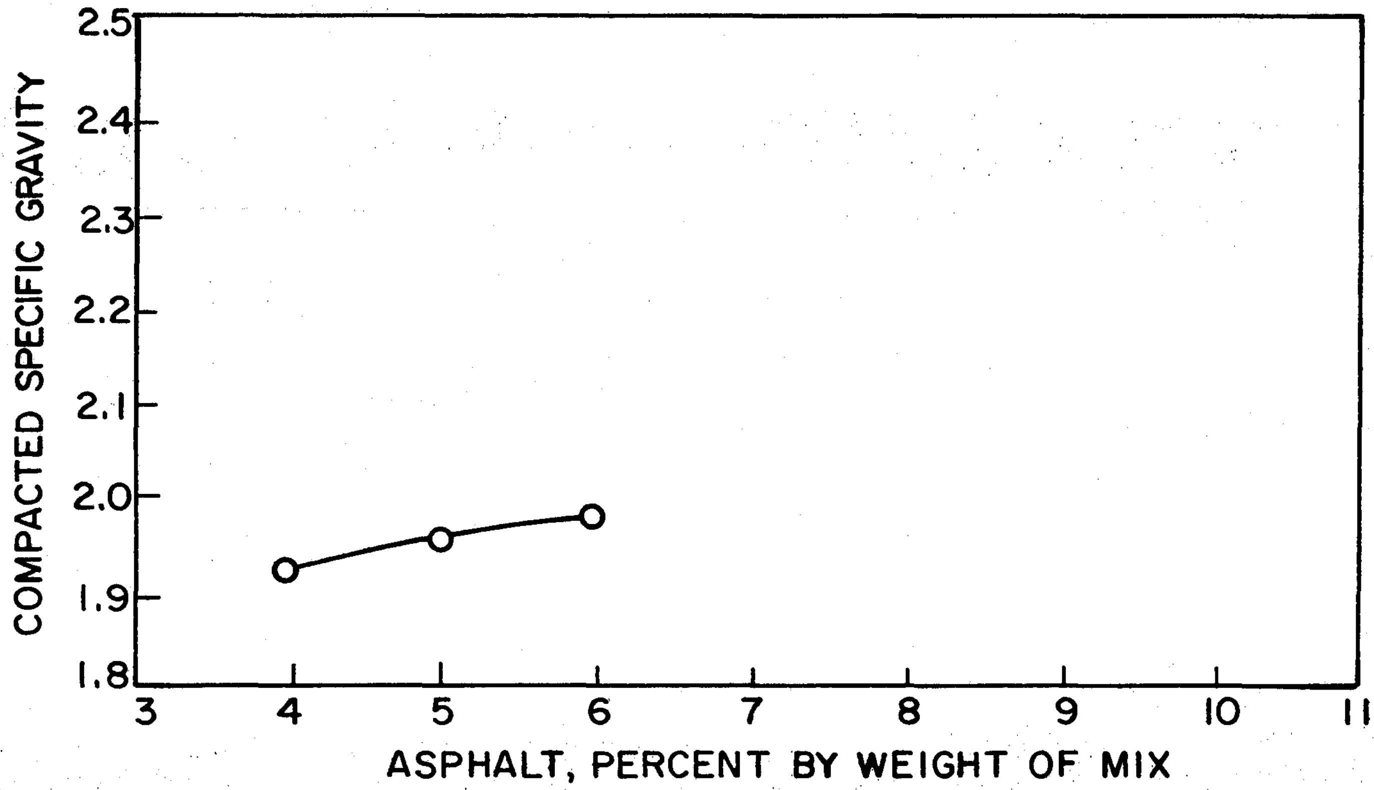

Figure 93. Compacted specific gravity versus asphalt content for Wheeler County (District 25 FM 3182).

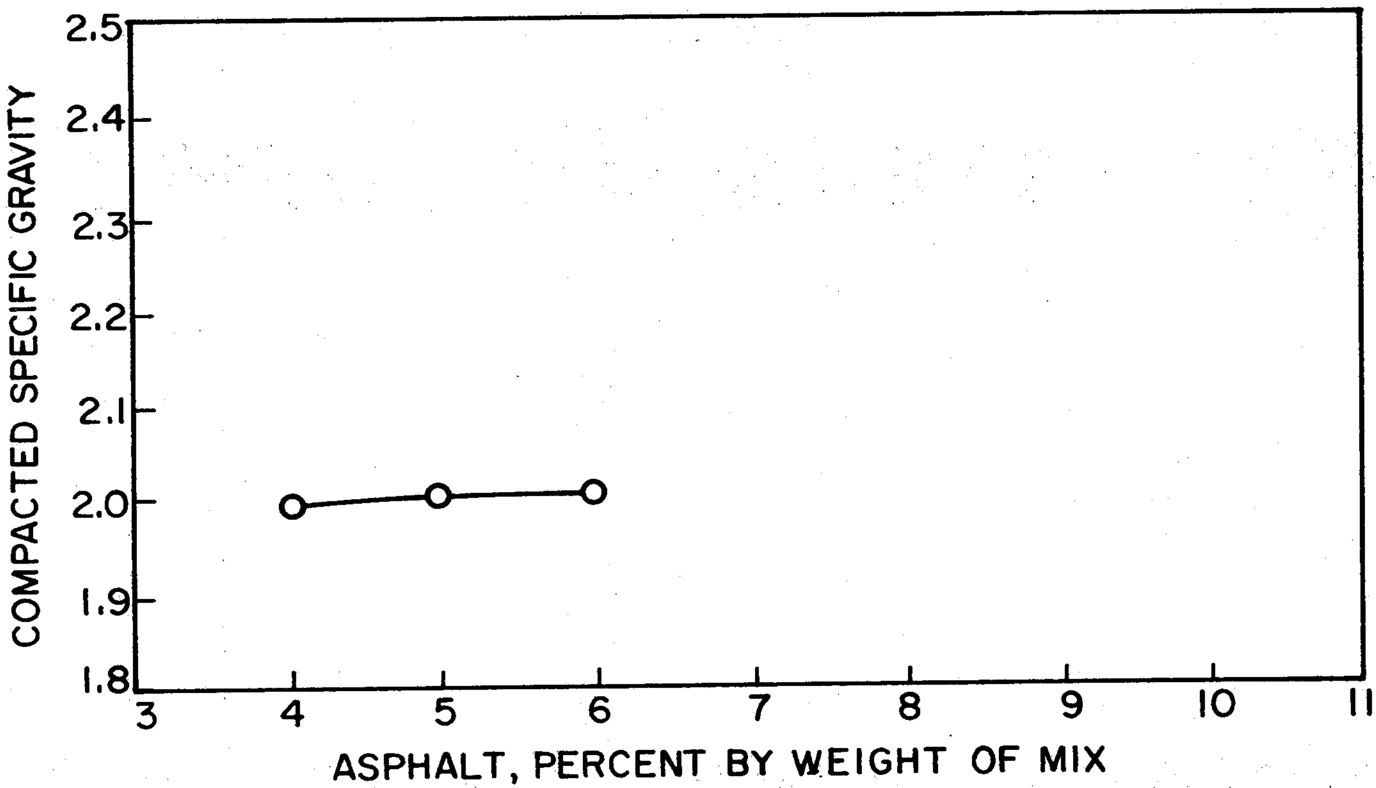

Figure 94. Compacted specific gravity versus asphalt content for Jasper County sand (District 20, U.S. 96).

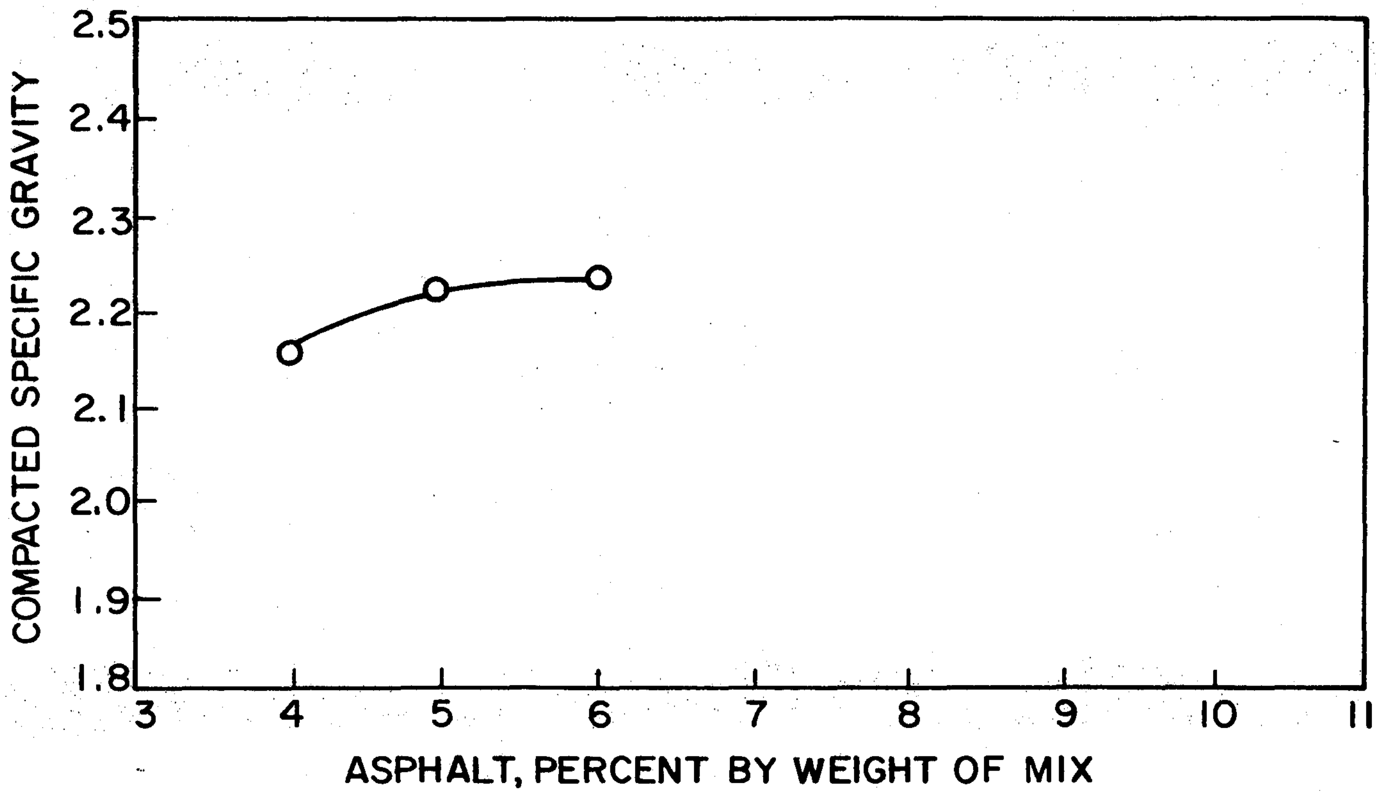

Figure 95. Compacted specific gravity versus asphalt content for Hidalgo County sand gravel (District 21, Beck pit).

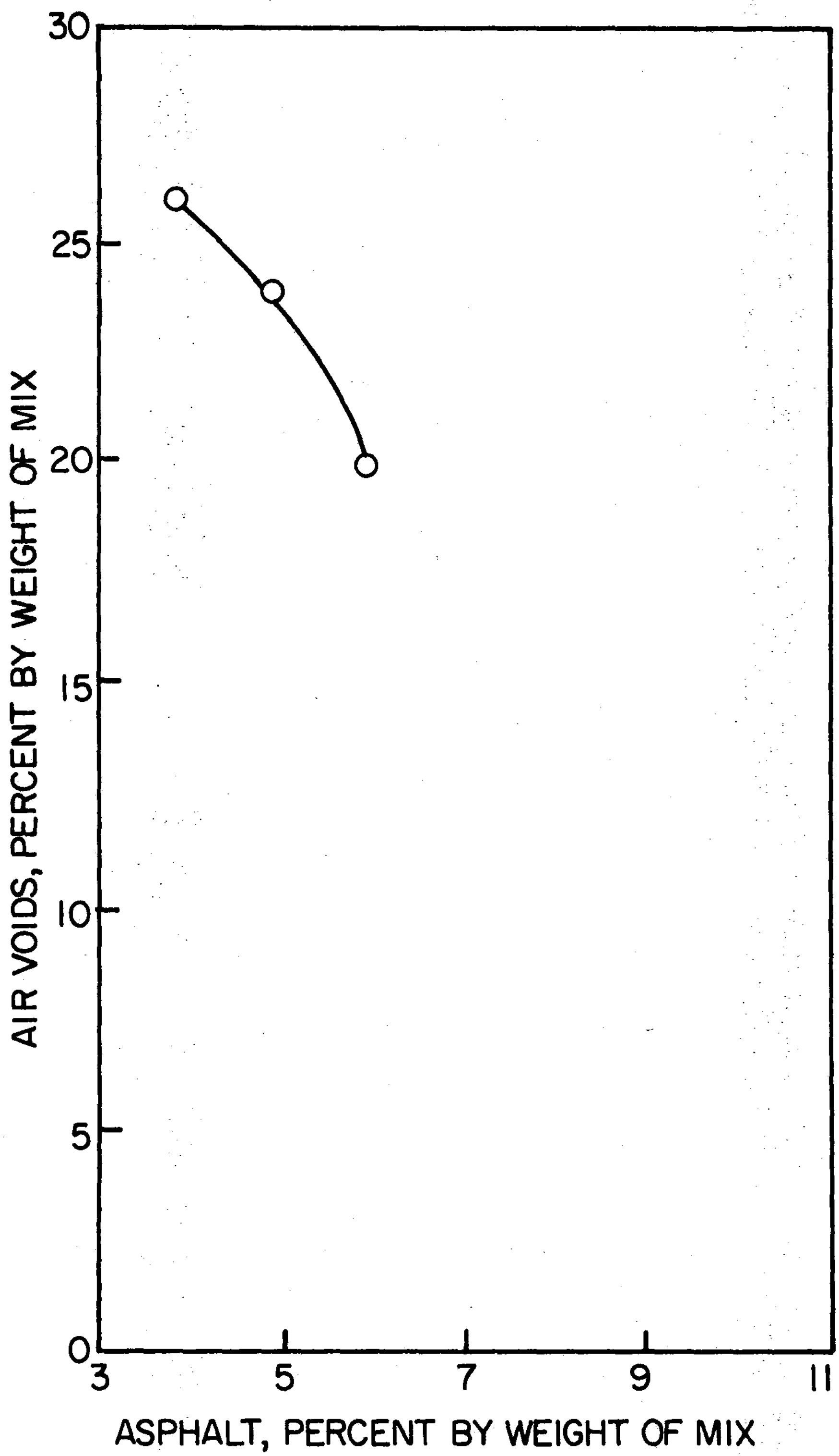

Figure 96. Air voids versus percent asphalt content for Lamb County sand (District 5, FM 168).

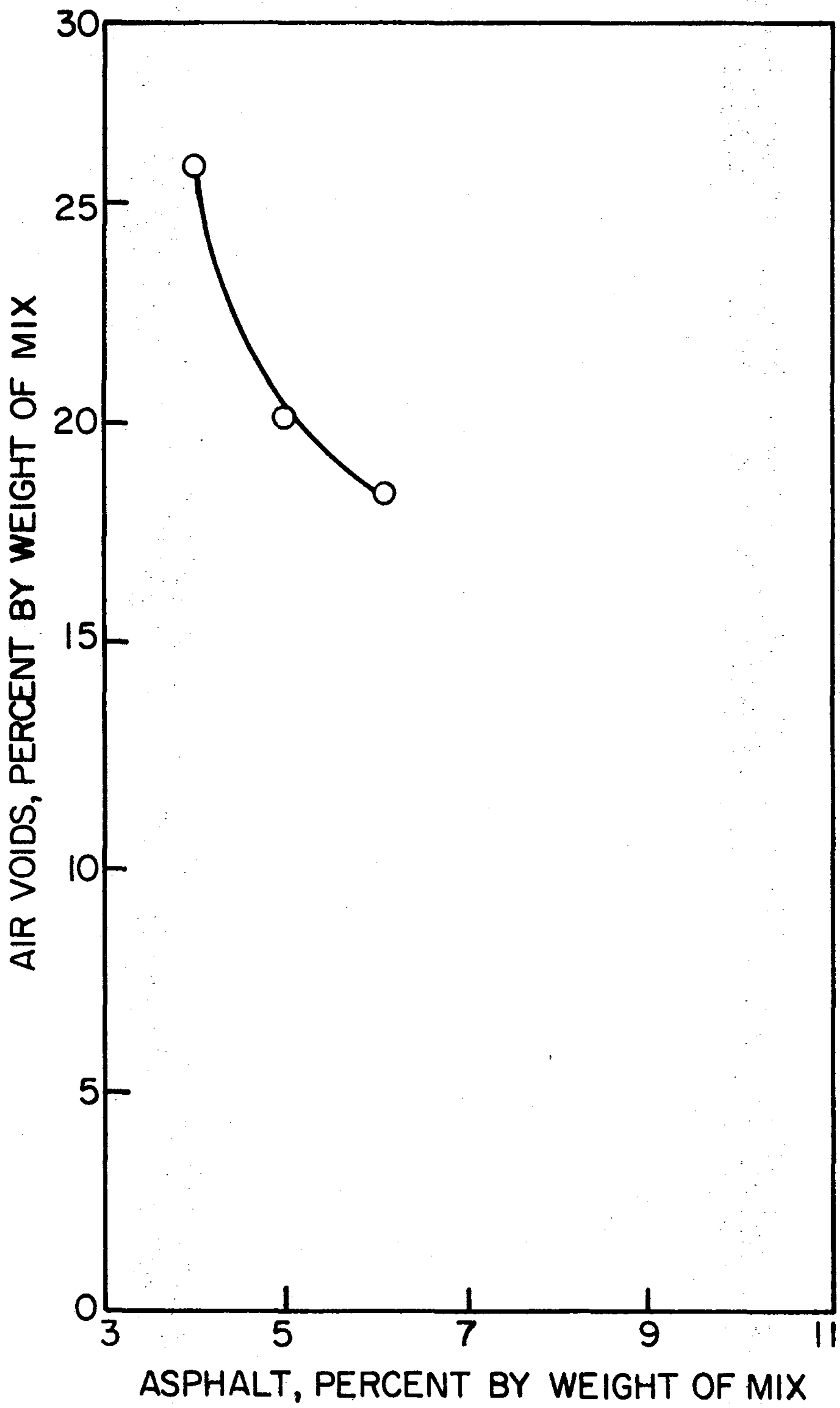

Figure 97. Air voids versus percent asphalt content for Wheeler County sand (District 25, FM 3182).

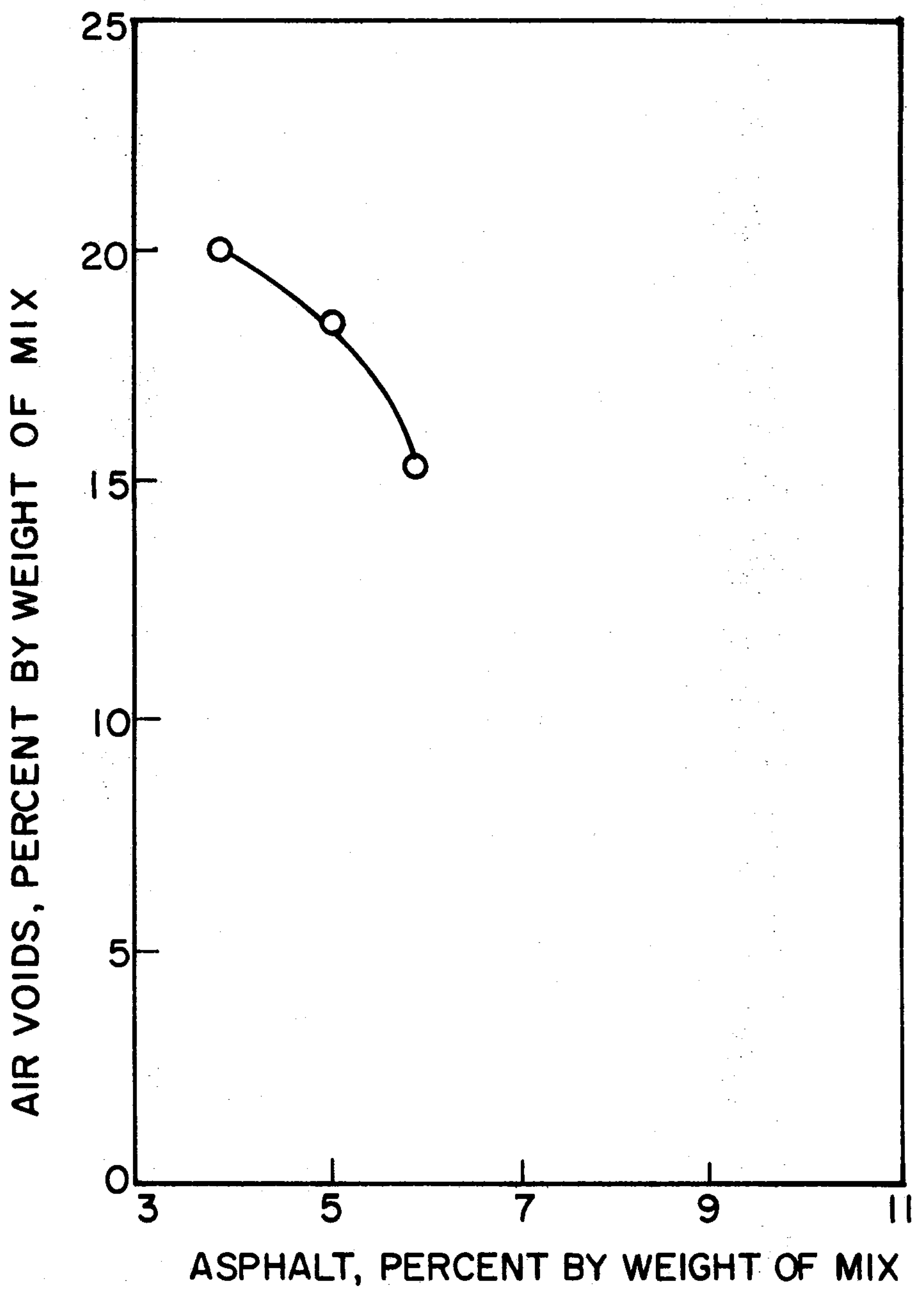

Figure 98. Air voids versus percent asphalt content for Jasper County (District 20, U.S. 96).

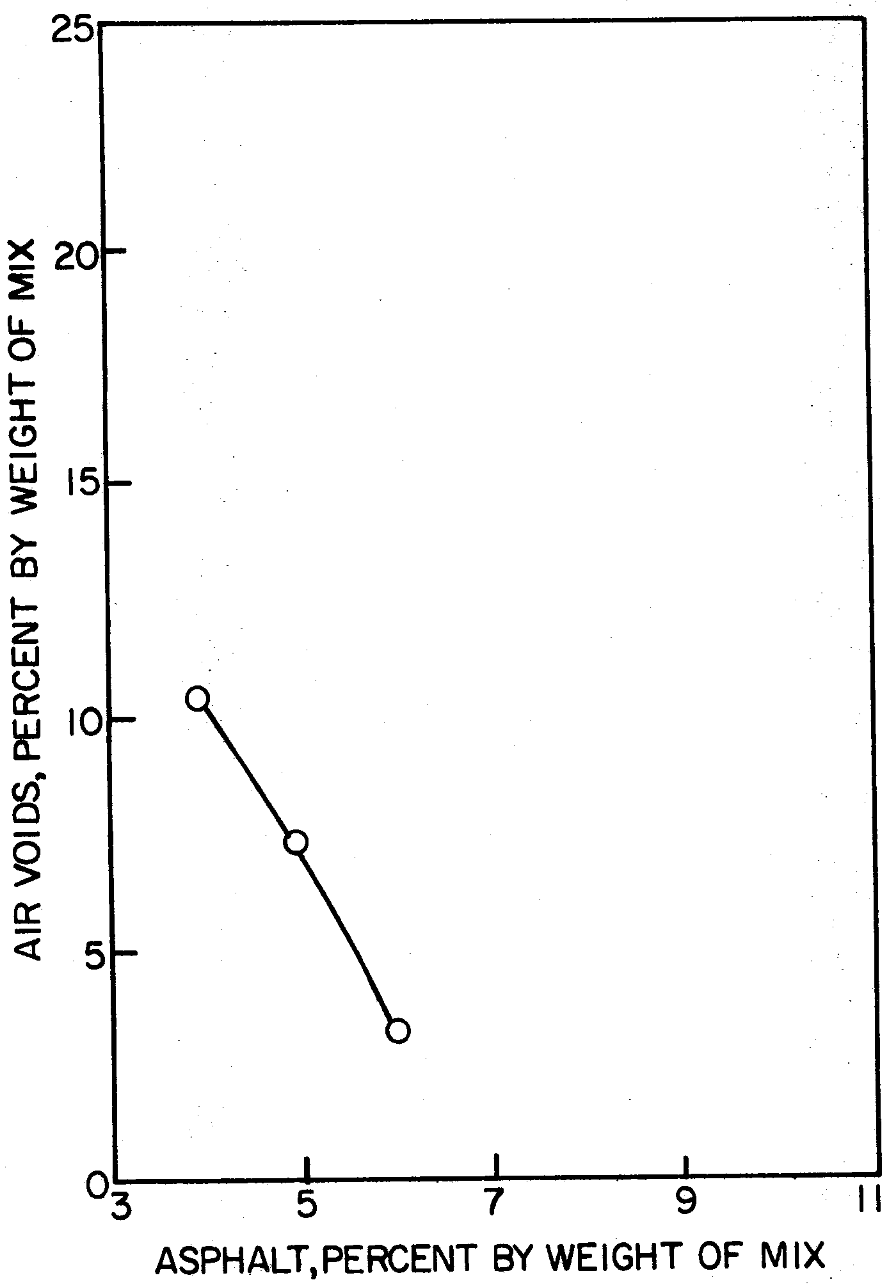

Figure 99. Air voids versus percent asphalt content for Hidalgo County sand-gravel (District 21, Beck pit).

Resilient Modulus

Although it is difficult to determine a trend from the limited resilient modulus results (Figures 100 to 103), it is apparent that a smaller difference exists between the original resilient modulus and the value after vacuum saturation for the mixtures at the higher asphalt contents. This observed trend was also noted for the marginal materials obtained from District 15 and 18 and discussed above.

Dry back of the samples after vacuum saturation has the effect of shifting the curves (Figures 100 to 103) to within close proximity of the curve developed before vacuum saturation.

Review of data obtained for the Lamb and Wheeler County sands indicates that maximum resilient modulus values as measured before saturation are obtained at about 5 percent asphalt. Asphalt contents in excess of 6 percent are required to produce a maximum resilient modulus for the materials from Jasper and Hidalgo counties.

Figures 104 through 107 show the effect of asphalt content on the relationship between resilient modulus and temperatures. Notice that the curves are roughly parallel and that the percent asphalt primarily affects the magnitude of the resilient modulus.

Stability

Figures 108 to 111 illustrate the relationship between Hveem Stability and asphalt content before and after vacuum saturation. The Hveem stabilities of the Lamb and Wheeler County sands are relatively low both before and after vacuum saturation (Figures 108 and 109). Acceptable stability values were obtained for the Jasper and Hidalgo county materials.

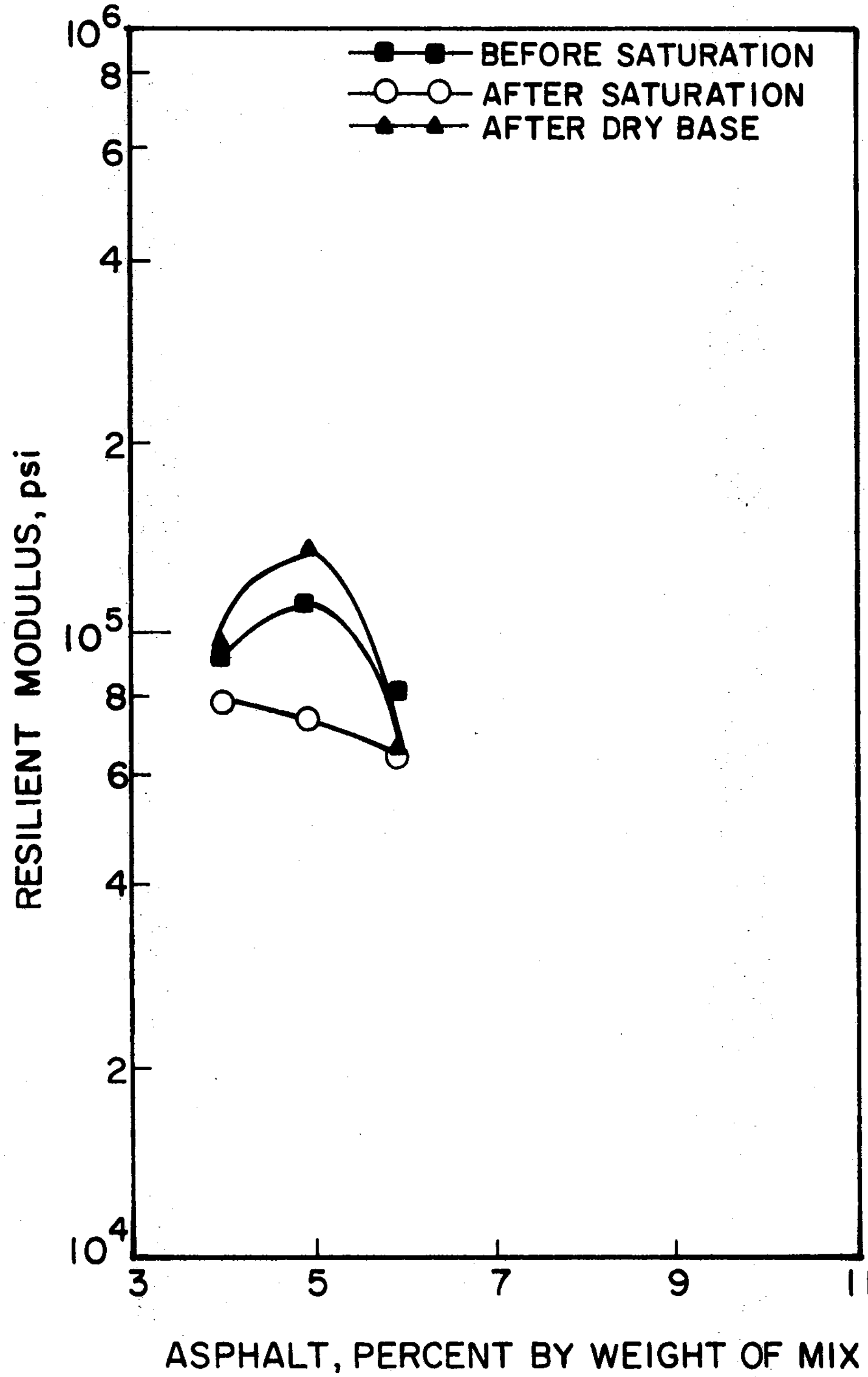

Figure 100. Resilient modulus versus asphalt content after vacuum saturation and after dry back for Lamb County sand mixes (District 5, FM 168).

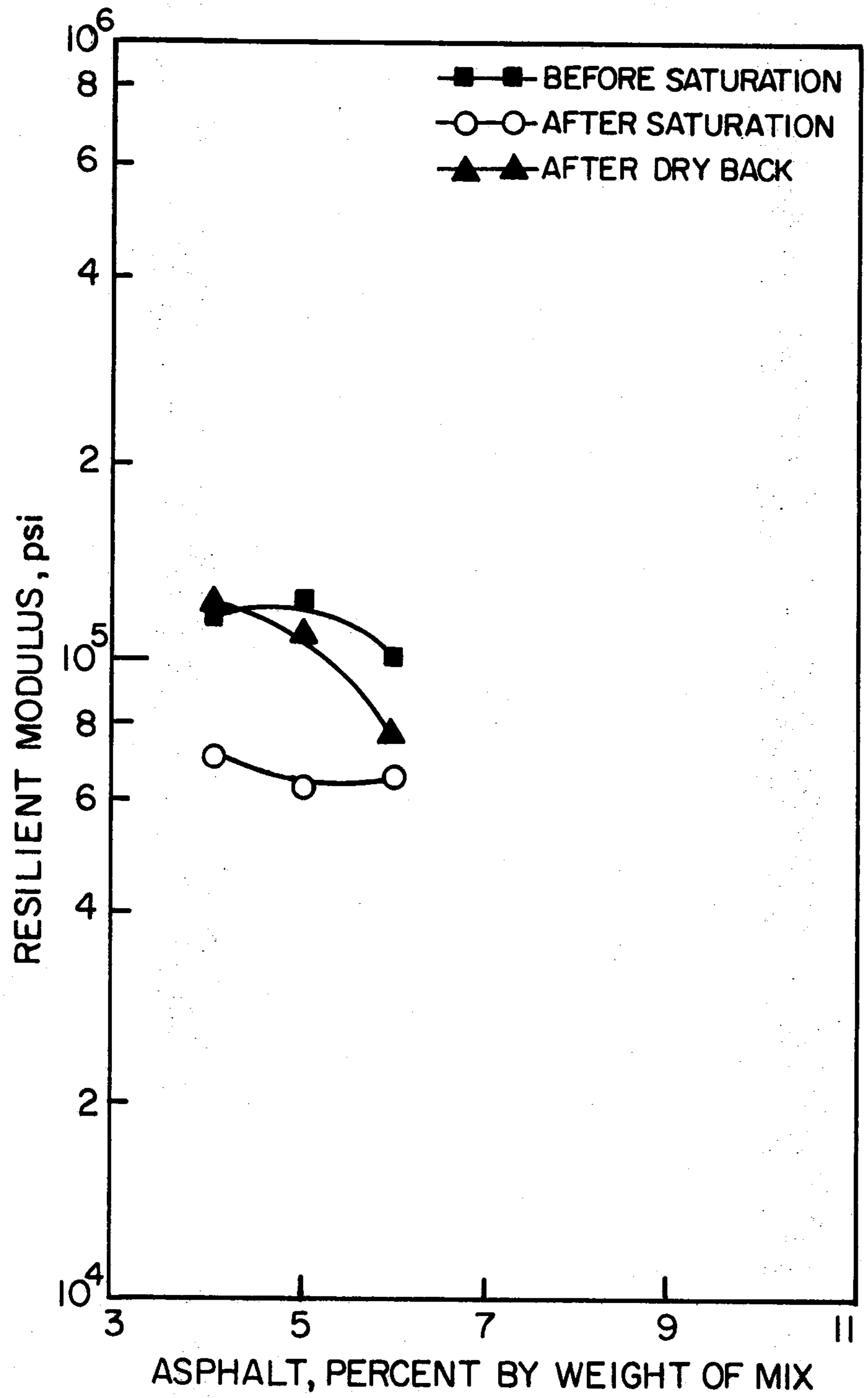

Figure 101. Resilient modulus versus asphalt content after vacuum saturation and after dry back for Wheeler County sand mixes (District 25, FM 3182).

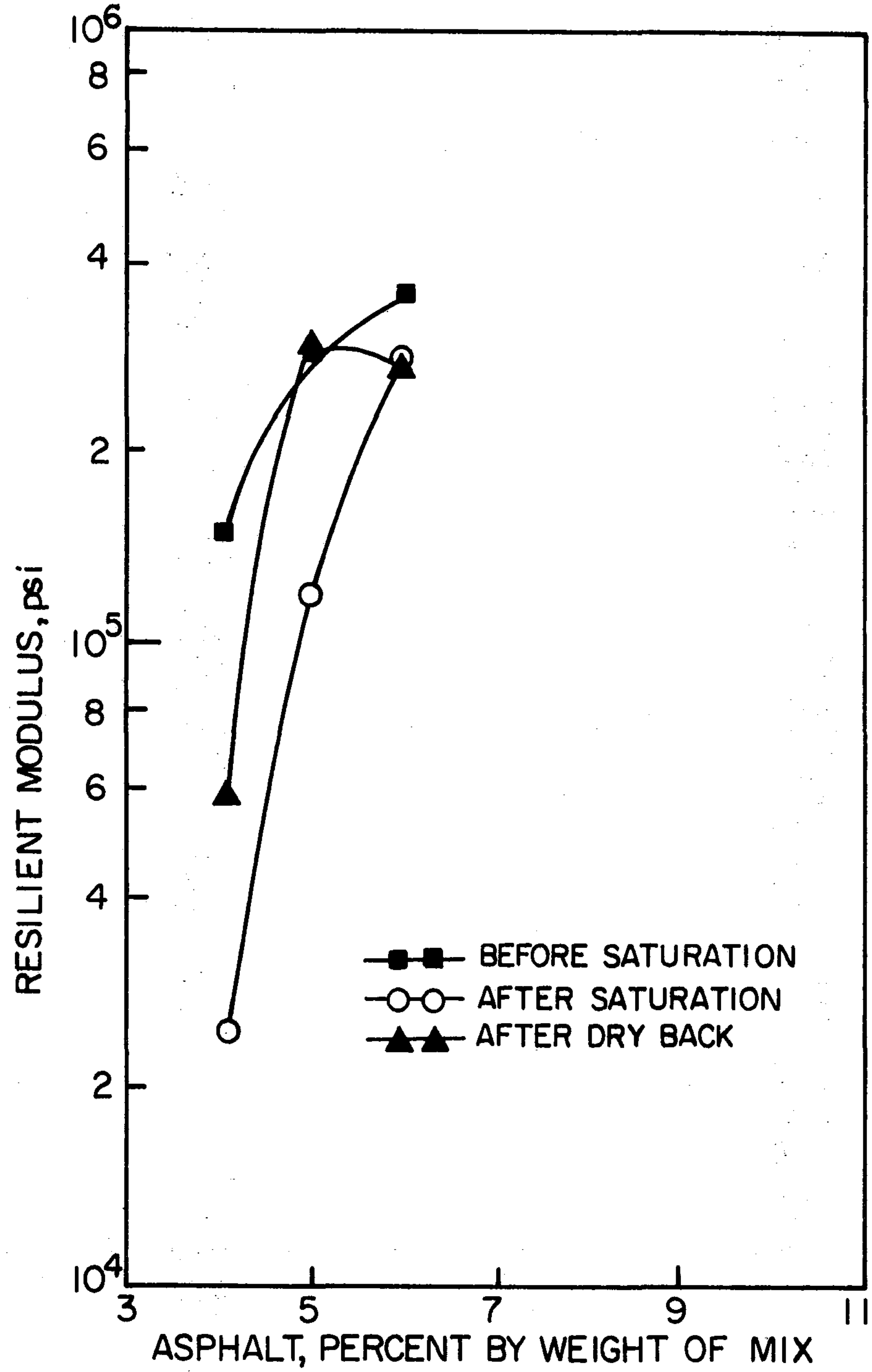

Figure 102. Resilient modulus versus asphalt content after vacuum saturation and after dry back for Jasper County sand mixes (District 20, U.S. 96).

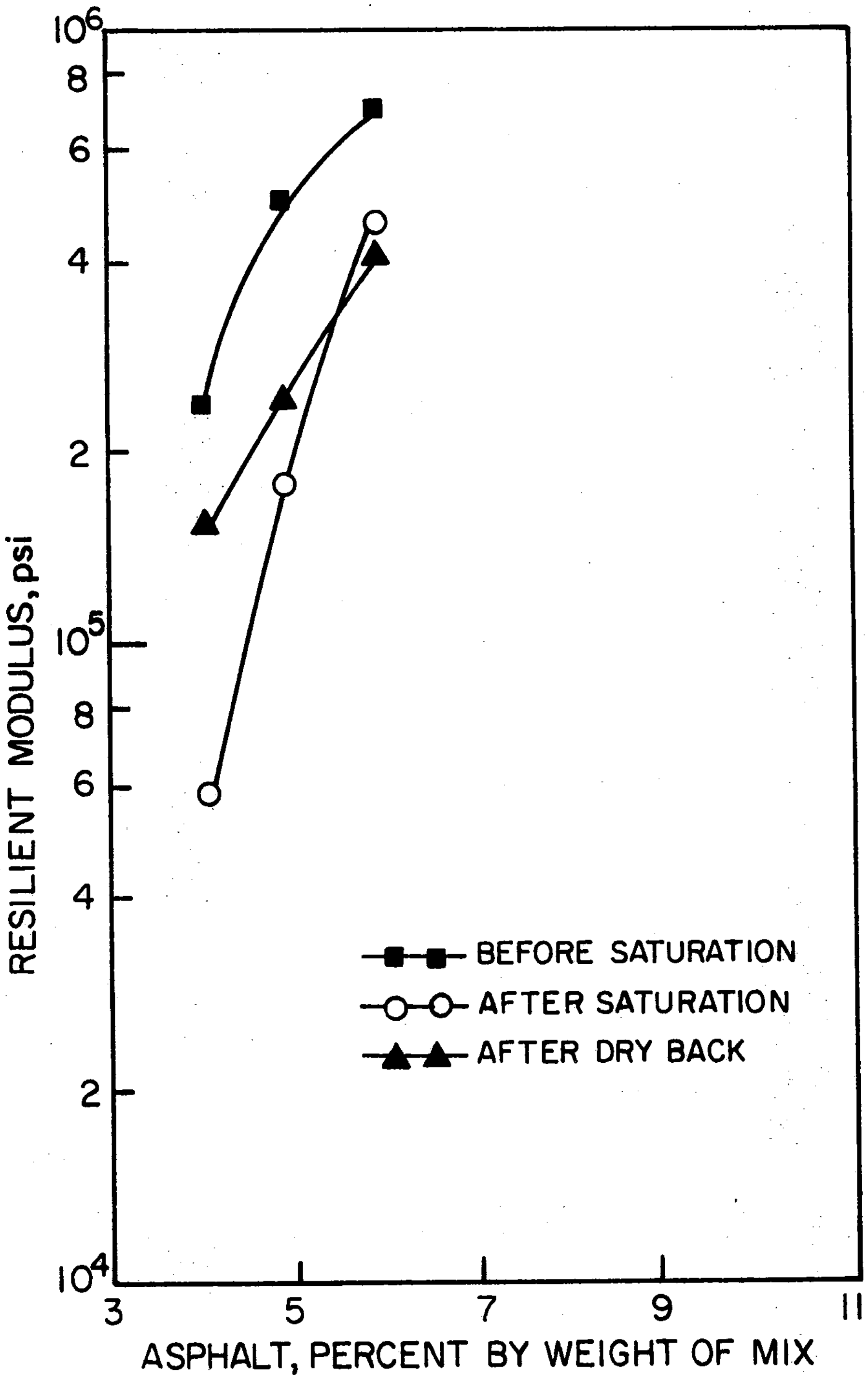

Figure 103. Resilient modulus versus asphalt content after vacuum saturation and after dry back for Hidalgo County sand mixes (District 21, Beck pit).

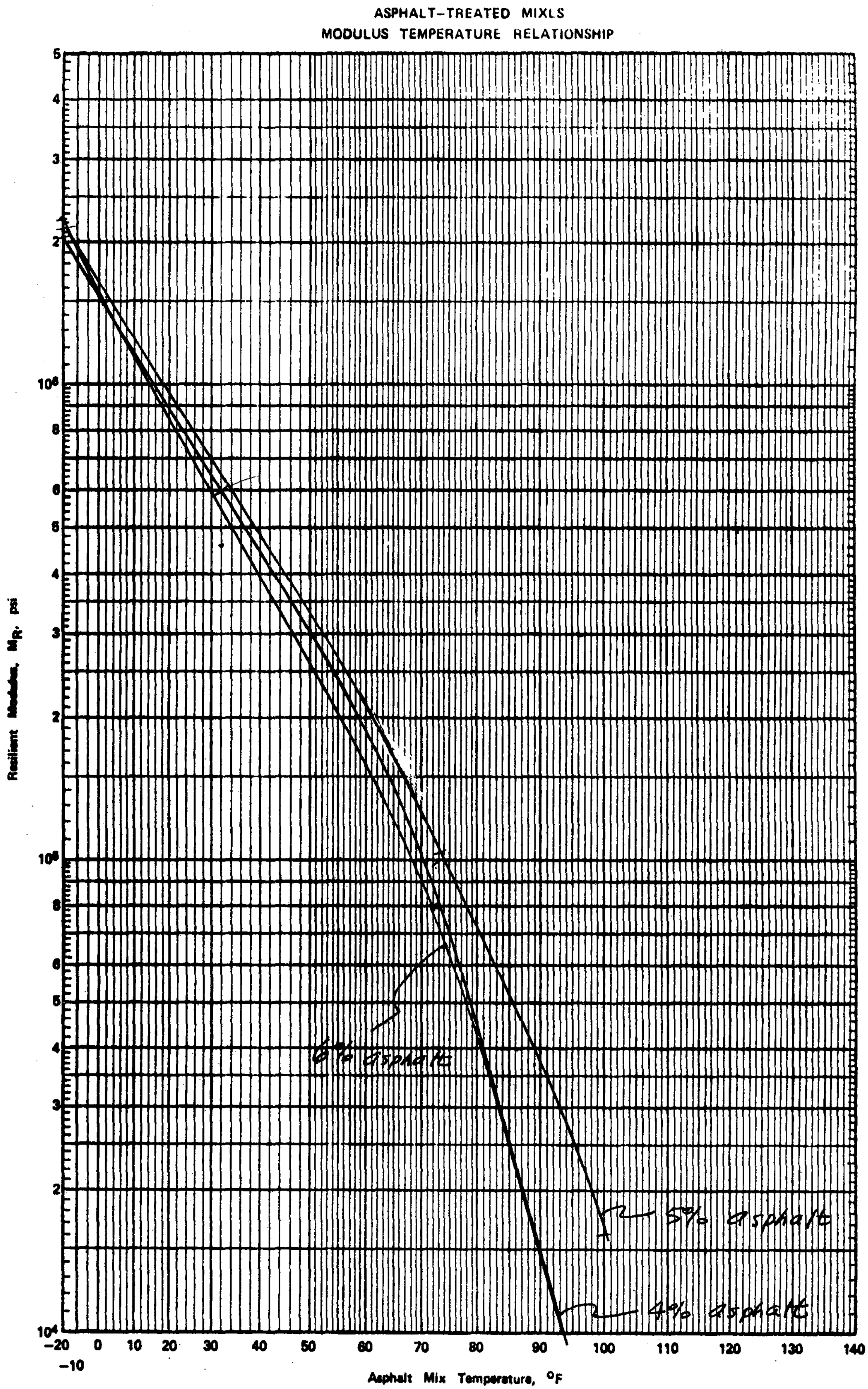

Figure 104. Temperature susceptibility of Lamb County sand mixes (District 5, FM 168)

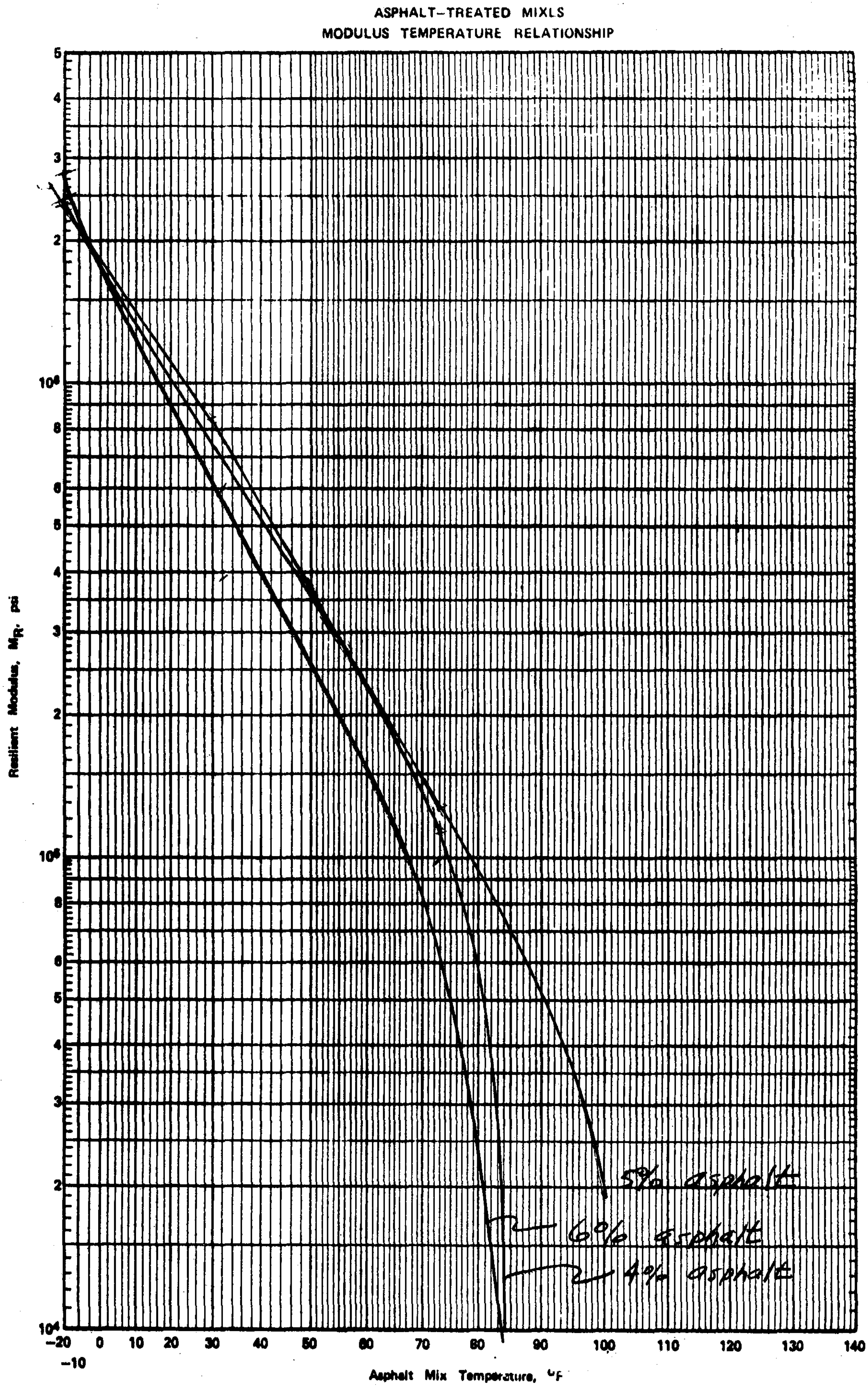

Figure 105. Temperature susceptibility of Wheeler County sand mixes (District 25, FM 3182)

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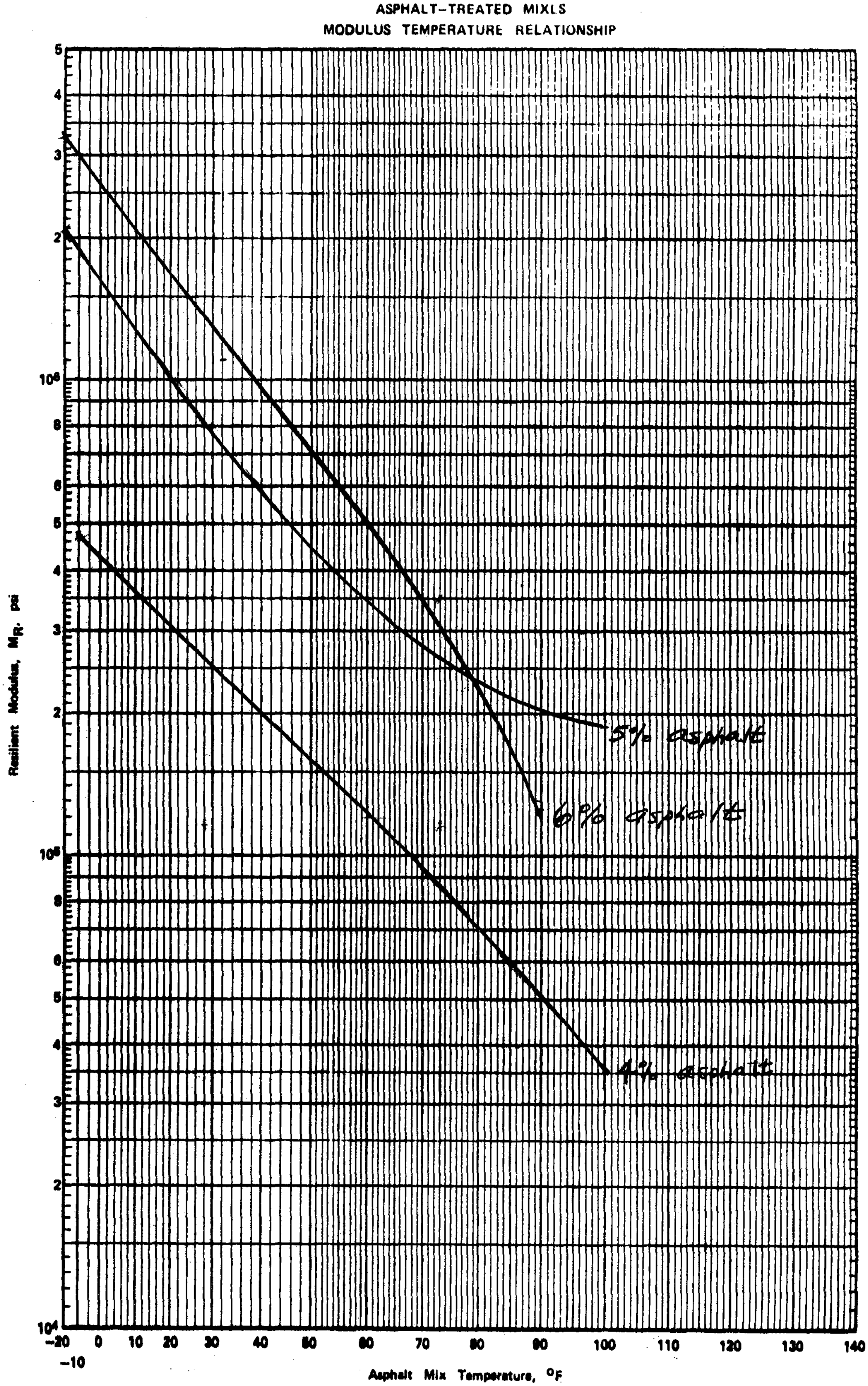

Figure 106. Temperature susceptibility of Jasper County sand mixes (District 20, U.S. 96)

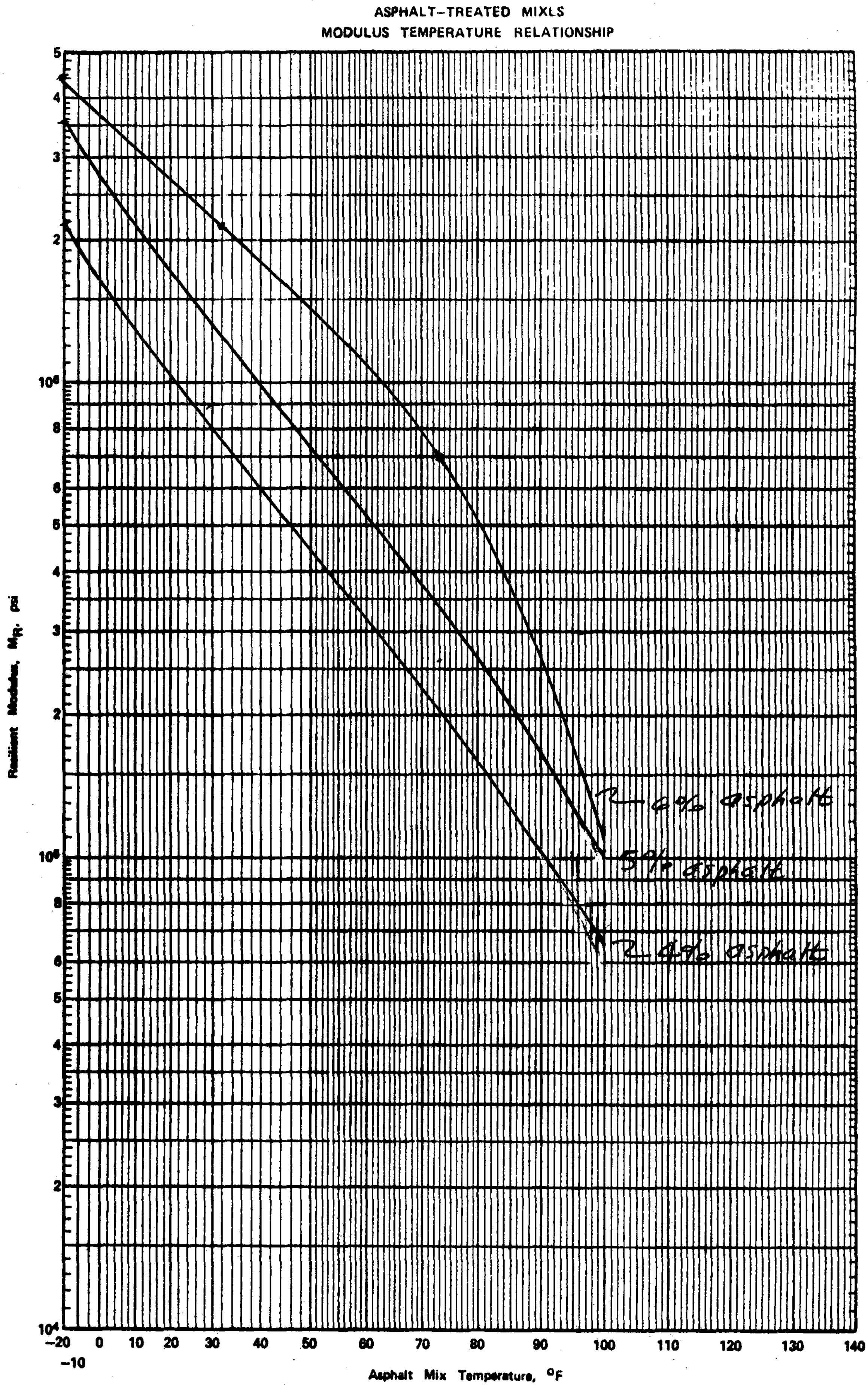

Figure 107. Temperature susceptibility of Hidalgo County sand gravel mixes (District 21, Beck pit)

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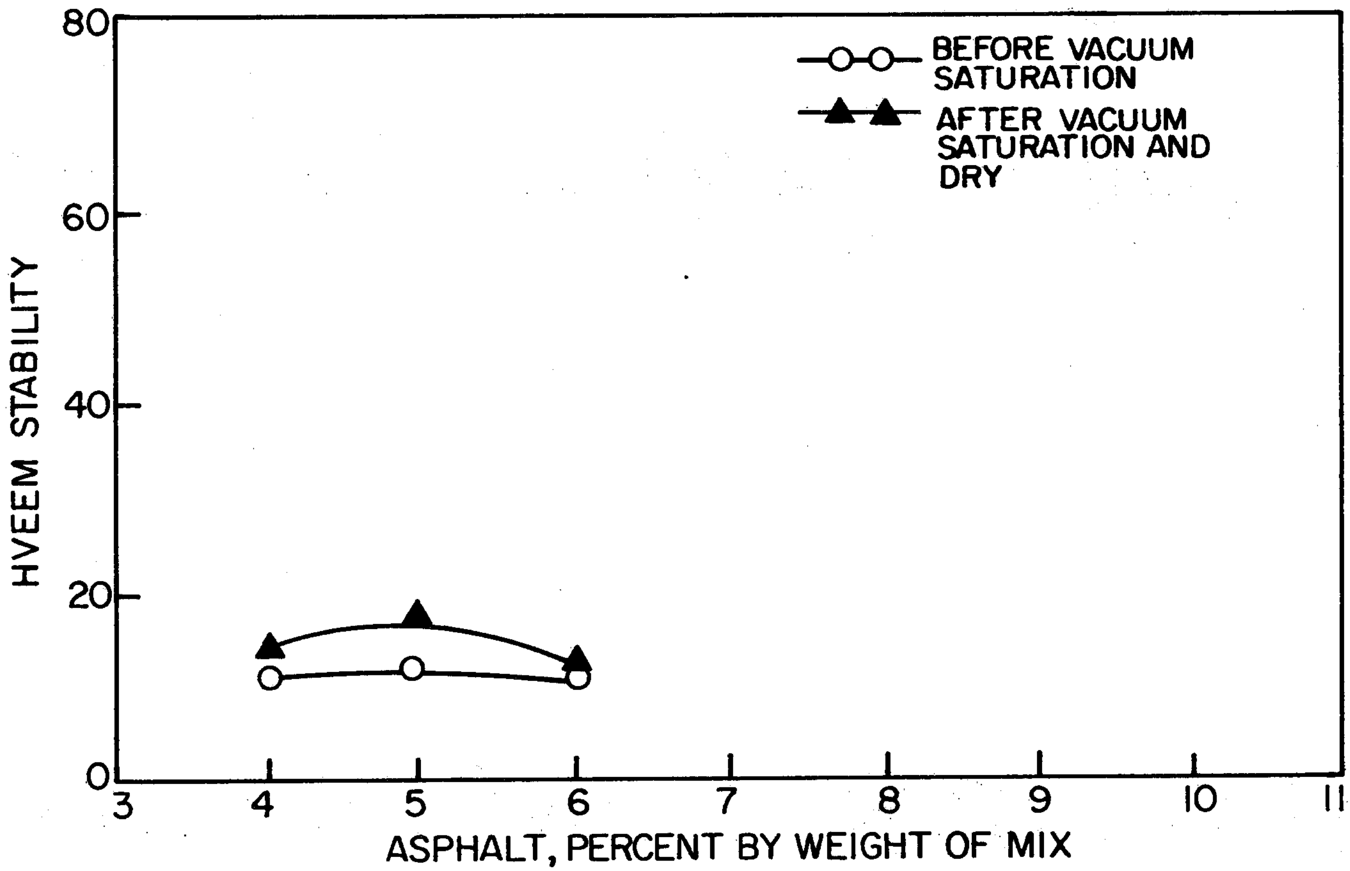

Figure 108. Stability values versus percent asphalt for Lamb County sand mixes (District 5, FM 168).

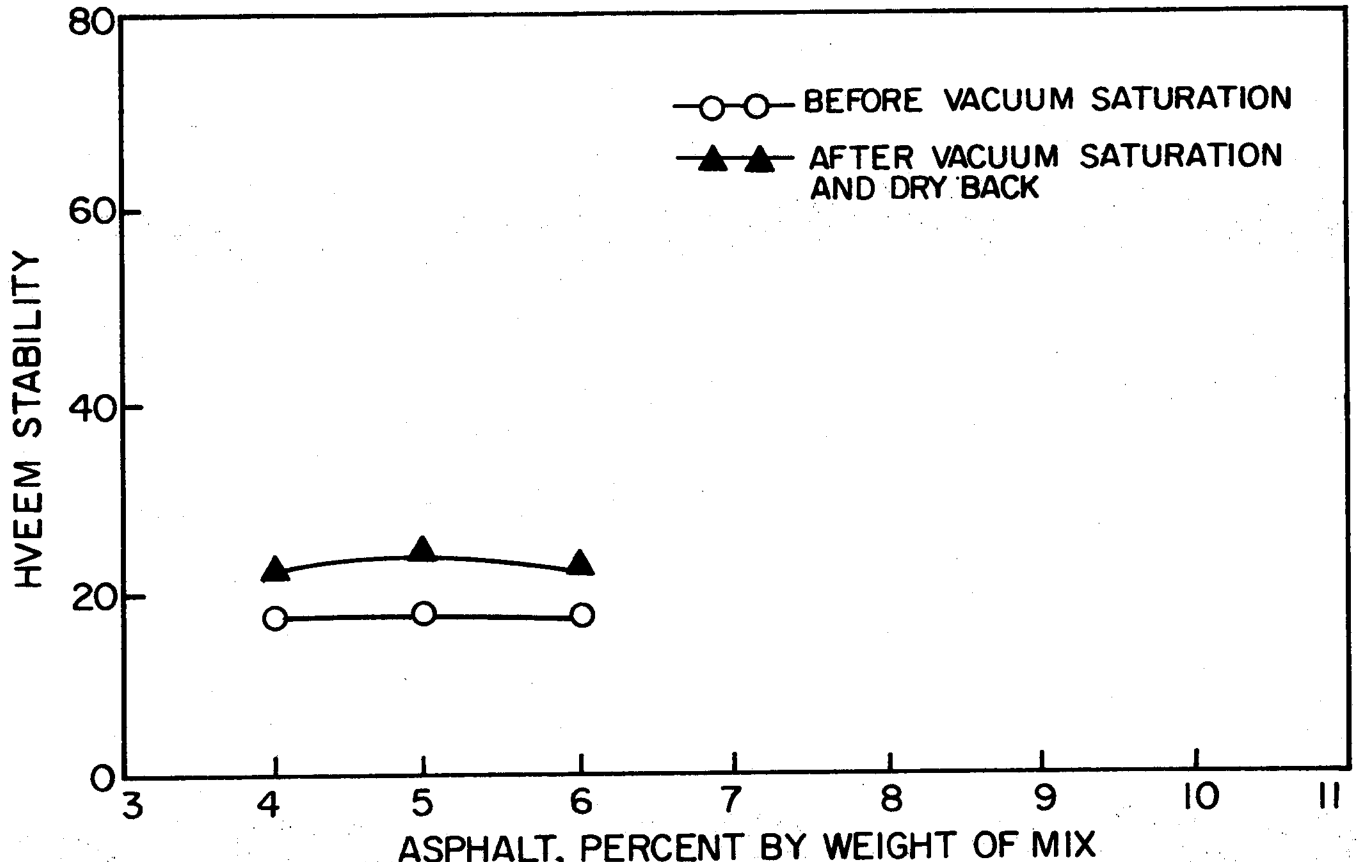

Figure 109. Stability values versus percent asphalt for Wheeler County sand mixes (District 25, FM 3182).

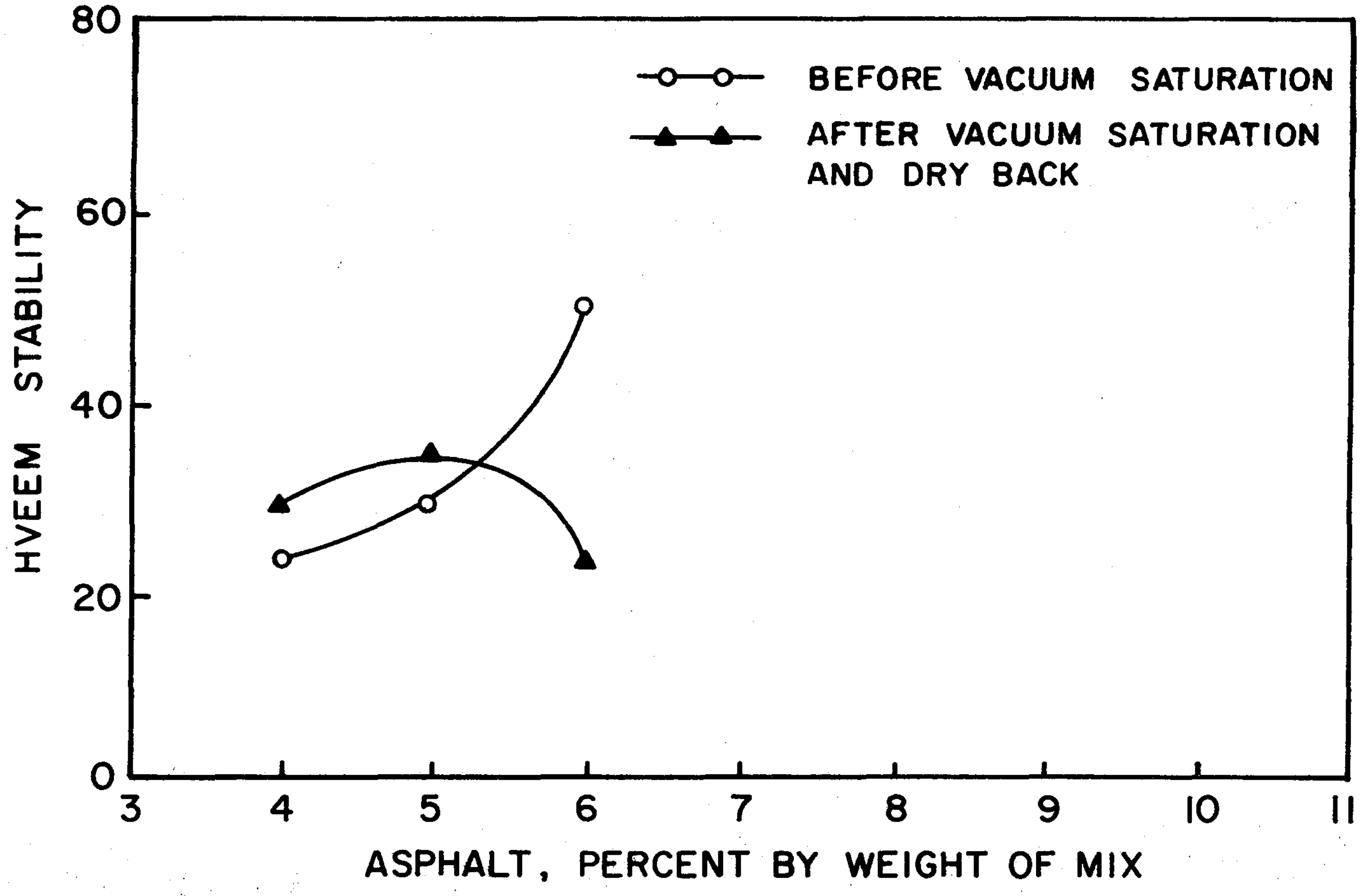

Figure 110. Stability values versus percent asphalt for Jasper County sand mixes (District 20, U.S. 96).

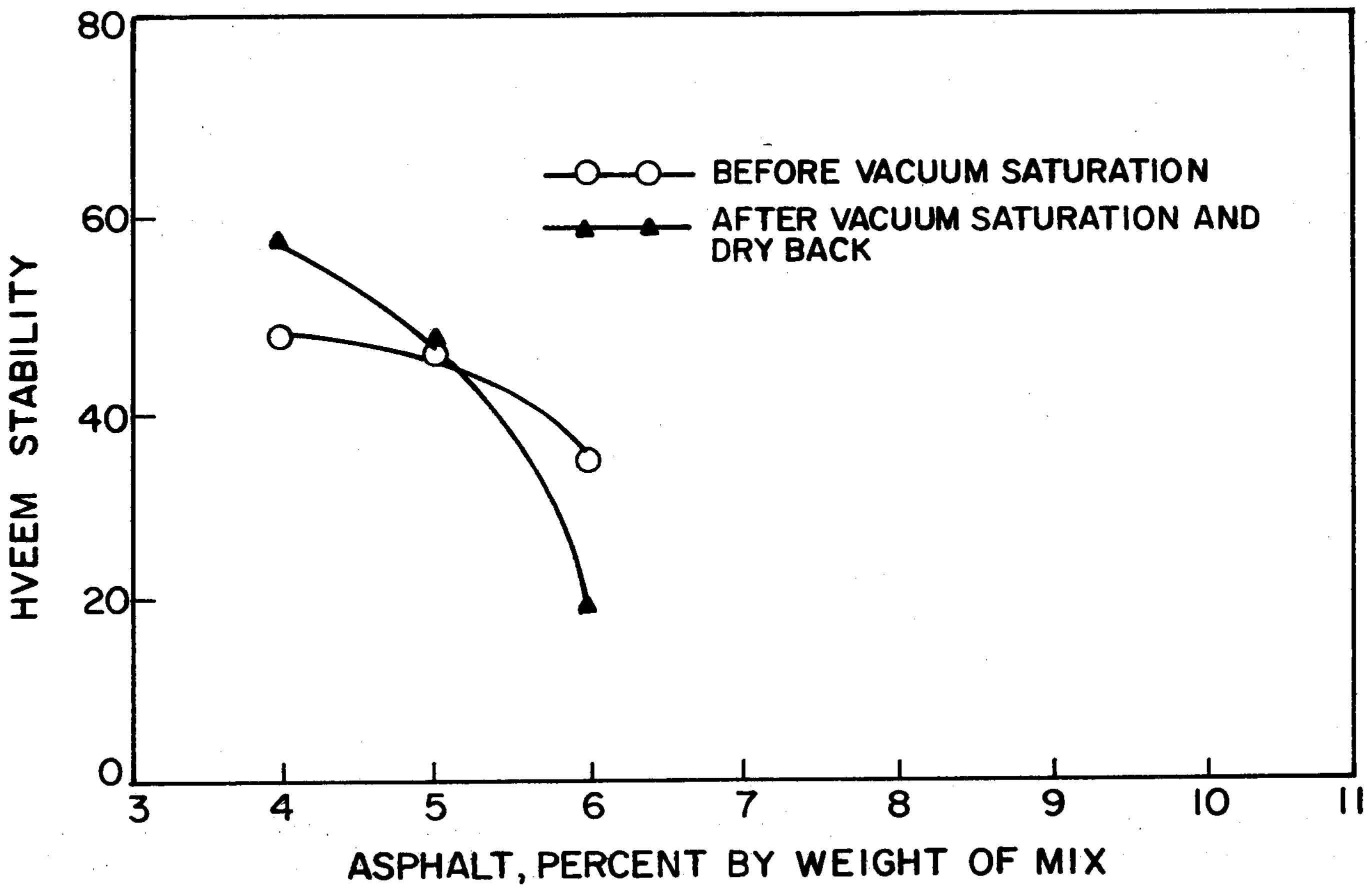

Figure 111. Stability values versus percent asphalt for Hidalgo County sand gravel mixes (District 21, Beck pit).

Suitability of Mixes

From a stability standpoint the Lamb and Wheeler county sands would be considered unsuitable according to existing Texas State Department of Highways and Public Transportation criteria. However, it is felt that these types of materials might perform satisfactorily as base course for roadways with low traffic volumes as the resilient modulus although low, is within an acceptable range based on field performance data.

The materials from Jasper and Hidalgo counties have acceptable stability values and should produce an adequate base material for roadways with average traffice volumes.

Additionally, selected blending of two or more sands for gradation improvement is a suggested alternative for improving structural properties.

LABORATORY TEST RESULTS - FIELD SAMPLES

Field core samples of several asphalt stabilized bases were obtained and tested to provide data for comparison with laboratory compacted samples. Both dense graded and sand bituminous treated bases were obtained from Districts 11 (Lufkin), 15 (San Antonio) and 20 (Beaumont). Results from cores containing dense graded mixutres from District 15 will be discussed initially, followed by results on sand asphalt mixtures from Districts 11 and 20.

District 15 and 18 Field Samples

IH-37, Pleasanton. Black base pavement sections along Interstate 37 south of Pleasanton, Texas have experienced severe stripping. The highway section was constructed in the fall of 1973. The aggregate in the black base mixture was a locally available crushed caliche gravel which included a slight amount of sandstone. The aggregate was quite similar to the Mack pit aggregate investigated in the laboratory molded black base testing program. The design asphalt content as determined by test method Tex-126-E (26) was 6.2 percent by weight of mix and the average asphalt content for the entire project was 6.0 percent by weight of mix.

The stripping occurred in both the north and south bound lanes. The structural sections of the two lanes are as follows:

North Bound Lane

1 1/2-inches Type D surface
3 1/2 to 8 1/2-inches gravel black base and salvaged pavement
sandstone subgrade

Table 5. Test results of black base field cores-1H.37-Pleasanton.

Sample Number	Sample Thickness inches	Compacted Specific Gravity	Air Voids	Hveem Stability Before Vacuum Saturation	Hveem Stability After Vacuum Saturation	$M_R \times 10^6$ psi Before Vacuum Saturation 73°F	$M_R \times 10^6$ psi After Vacuum Saturation 73°F
NBL 1	2.04	2.27	4	14	23	0.36	0.37
NBL 2-1	1.48	2.29	3	10	29	0.98	0.09
NBL 2-2	1.65	2.32	2	22	43	0.41	0.08
NBL 3	1.91	2.13	10		22	0.78	0.03
SBL 4-1	1.93	2.20		28	31	0.26	0.05
SBL 4-2	1.97	2.28		25	32	0.38	0.049
SBL 4-3	2.00	2.25		32	42	0.31	0.072

South Bound Lane

1 1/2-inches Type D surface

10 1/2-inches gravel black base placed in three equal lifts

9-inches untreated gravel subbase with a Plasticity Index of 8 to 10

6-inches lime treated sand-silty clay subgrade.

Field cores were obtained from both lanes and were subjected to a testing program similar to that used with the laboratory molded marginal mixtures.

Black base samples were first obtained with a pick and shovel and the moisture content (percent water in mix) of the material was determined. The moisture content was 5.6 percent.

After slicing to proper dimensions, the field specimens were allowed to dry to a constant weight at 90°F and 25 percent relative humidity. After drying, the specimens were tested by the diametral method and the resilient moduli determined. Figure 112 illustrates the resilient moduli values obtained from the Pleasanton project as compared to those of similar projects in District 15 and obtained during a previous study. The samples were tested for stability according to test method Tex-208-F (26). The specimens were subjected to vacuum saturation and tested in the saturated condition. Figure 113 illustrates the effect of vacuum saturation on the black base mixtures. After drying the cores were again tested according to Tex-208-F. Test results are illustrated in Table 5. The average stability before saturation was 22 with a range of 10 to 32. After saturation stability ranged from 22 to 43 with an average of 32.

IH - 35, Dilley. Black base pavement sections in the north bound lane of Interstate Highway 35 north of Dilley, Texas have also experienced severe stripping problems; particularly, the section between County Line Road and State Highway 85. The highway section was constructed in March of 1973.

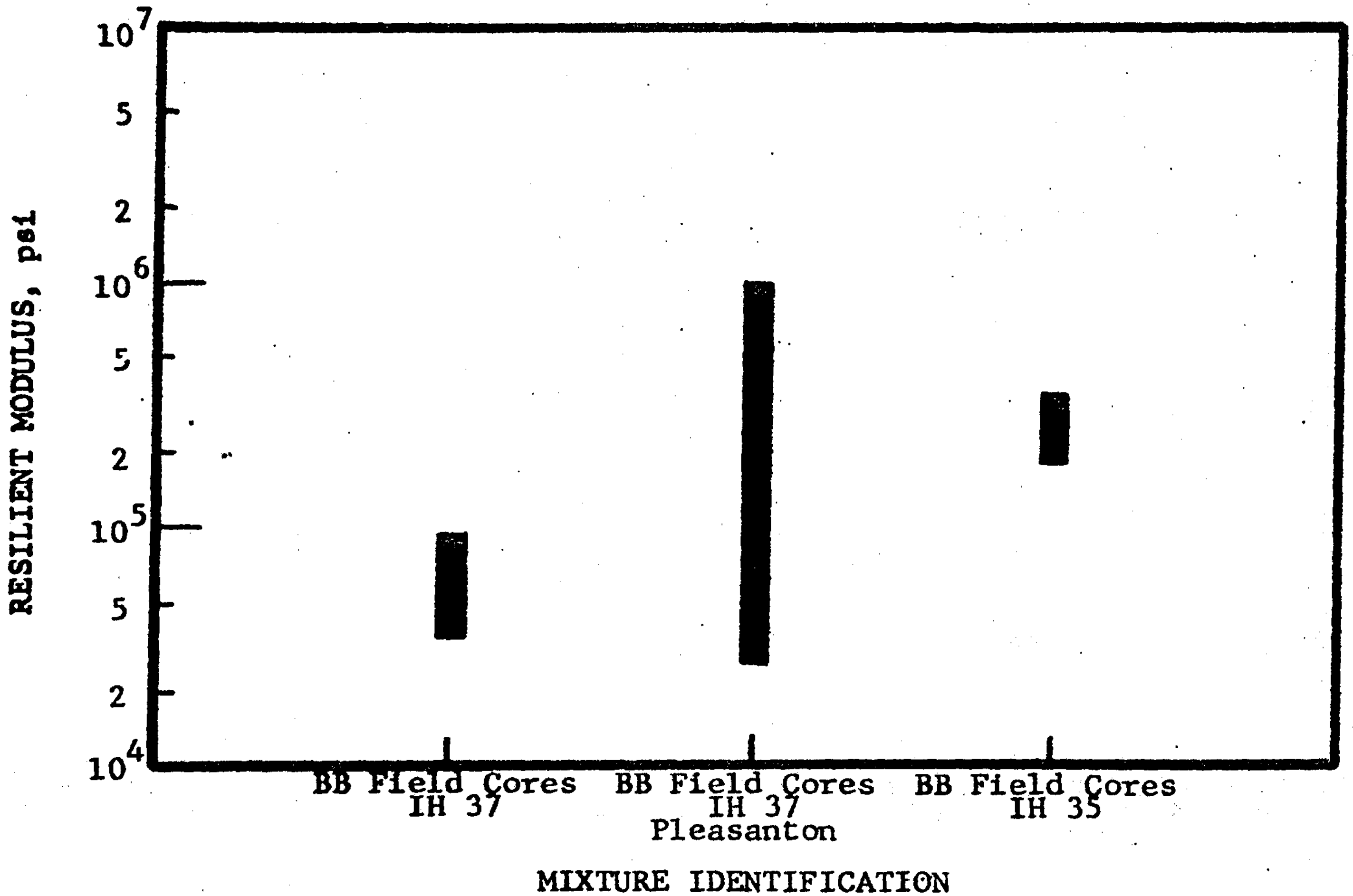

Figure 112. **Resilient modulus of black base mixtures at 73°F**

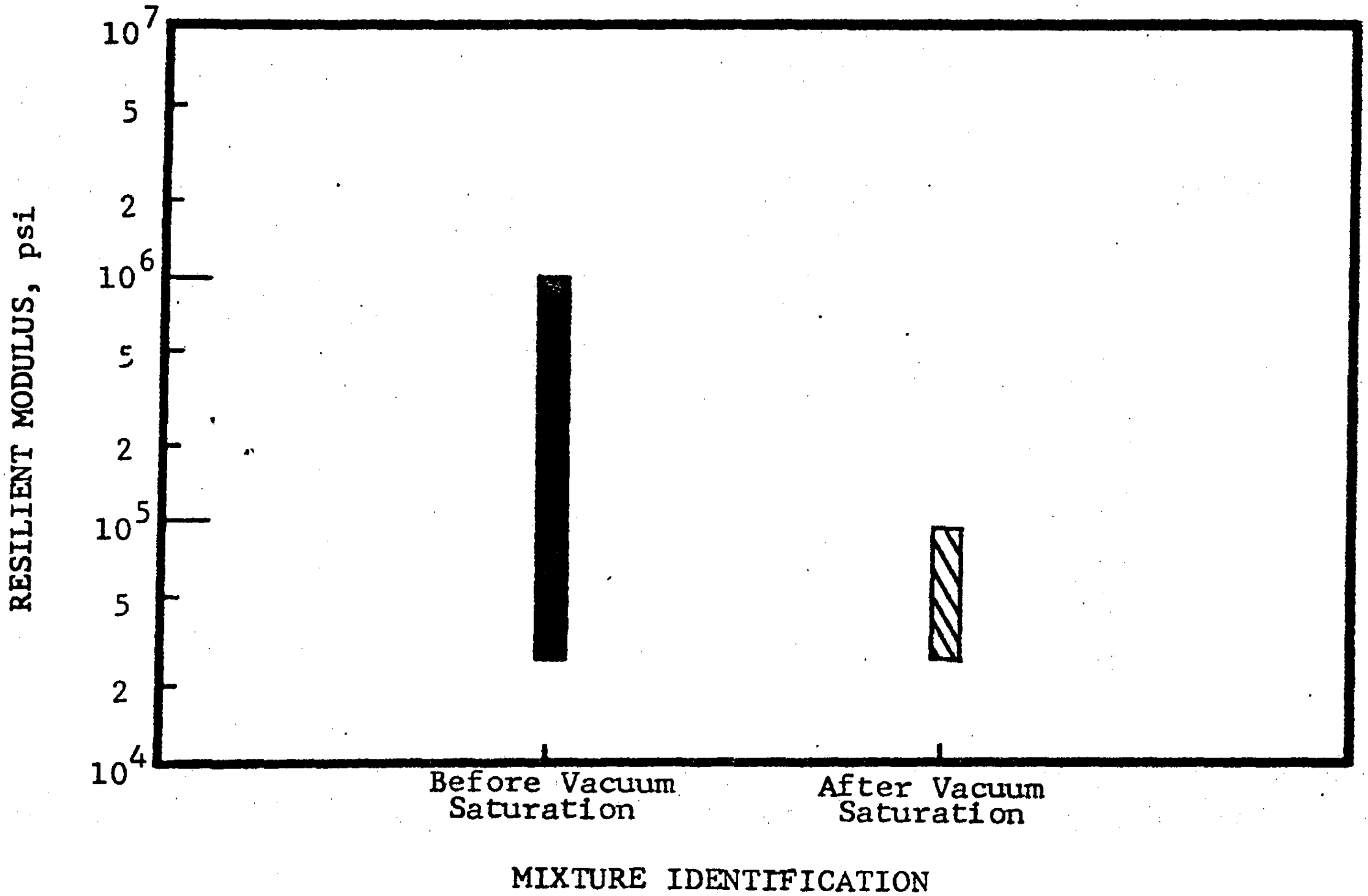

Figure 113. Effect of vacuum saturation on resilient modulus of black base field cores (District 15, IH 37 - Pleasanton)

The design asphalt content was determined to be 4.6 percent by weight of the mix using an AC 10 from Gulf States Refinery. The aggregate was classified as an extremely hard, caliche gravel obtained from the Lex Stuart property located east of Interstate Highway 35 and south of the Frio River. The aggregate appears to be quite similar to the Mack pit aggregate which was investigated in the laboratory molded black base testing program.

The first evidence of highway distress was rutting in the wheel paths which rapidly progressed to longitudinal cracking and finally alligator cracking and potholing. Waves and humps were quite evident on the pavement surface looking much like a "lumpy mattress". The rutting and resulting failures were almost exclusively noticed in the left wheel path of the right travel lane. Personnel from the Texas State Department of Highways and Public Transportation, District 15, contend that when sections of pavement were removed, the black base in the wheel paths was deteriorated only to a depth of about four inches and that the black base material below four inches and the material on either side of the wheel paths was generally structurally sound. This would indicate that water is either penetrating through the surface layer or entering through the sides of the pavement section and is concentrating itself in the left wheel path of the right travel lane.

District 15 personnel have conducted extensive laboratory testing in an attempt to solve the stripping related problems. In analyzing the black base material from the Dilley project, it was determined that the asphalt content ranges from 3.82 percent to 5.04 percent by weight of the mix. In testing the aggregate stockpiles, the following results were obtained:

Sieve Size	Accumulative Percent Retained
1 3/4 in.	0
7/8 in.	17-44
#4	55-70
#40	67-78

Plasticity Index 6-8

Liquid Limit 18-21

The Asphalt Institute conducted tests on the black base material (Dilley) and determined the percent passing the number 200 sieve to be between 17.2 and 19.2 percent and the percent passing the number 50 sieve to be between 26.4 and 31.0 percent.

Cores were not obtained from this project and thus comparative data were not obtained.

IH 37, Campbellton. A black base pavement section was placed in Atascosa County on Interstate Highway 37 near Campbellton, Texas in May of 1975. The aggregate in the black base was a crushed sandstone obtained from the "74" Ranch pit; the same material tested in the laboratory molded black base program. The design asphalt content was 8.1 percent by weight of the mix. To date, the pavement section has indicated no evidence of stripping.

District 18. Through conversations with District 18 personnel, it was learned that a black base section using the Austin chalk aggregate had been placed near Dallas, Texas. The section is a one-lane, half mile long, detour route and is expected to service construction traffic for three years. The structural section is two inches of Type "D" surface course and ten inches of Austin chalk black base.

The black mixture contained 8.5 percent asphalt (AC 20) by weight of the mix and one and one-half inch maximum size aggregate. The fines (minus number 40 sieve) were not scalped from the mix for this project but District 18 personnel have found that scalping approximately ten percent of the minus 40 material and adding back ten percent river sand, will reduce the optimum asphalt content to 7.0 percent by weight of the mix.

District 11 and 20 Field Samples

Samples of sand asphalt bases were obtained from Districts 11 and 20. Information as to the sources of sands utilized for these mixtures is shown in Table 6. Properties of the mixtures are shown on Table 7. Hveem stability values at 140°F are for the most part lower than that commonly specified and air voids are above those normally specified.

Performance of these pavements has not been studied; however, for the most part acceptable performance has been reported for these pavements by the districts.

Discussion of Laboratory Results

Figure 114 illustrates the range of resilient moduli values (at 73°F and before vacuum saturation) for the five laboratory molded specimens from District 15 and 18. The range shown is for all asphalt contents investigated. The Austin chalk and crushed sandstone demonstrate a wide range of moduli values. The poorest of the five mixtures, the Austin chalk, could not withstand the vacuum saturation procedure. Many of the crushed caliche gravel samples also fell apart during vacuum saturation.

A comparison of the resilient modulus of several types of materials as measured from laboratory molded specimens and field cores is shown in Figure 115. As shown the asphalt stabilized bases in Texas have a wide range in properties. Thus, it is important that each material utilized as a base course be

Table 6. Location and Material Field Cores, District 11 and 20.

Section No.	District	County	(Control) Highway	Location	Materials	
					Aggregate	Asphalt
1	11	Angelina	SH 103	West of LP 287	50 percent Daniels Sand, Angelina Co. 50 percent Vincent Sand, Angelina Co.	
2	11	Angelina	LP 287	West of US 59	60 percent Temple Sand, Trinity Co. 40 percent Daniels Sand, Angelina Co.	
3	11	Angelina	LP 287	East of US 59	100 percent Gipson Sand, Angelinz Co.	
4	11	Trinity	US 287	East of Woodlake	100 percent Bradley Pit Sand, Trinity Co.	
5	20	Jefferson	(307-2) SH 87	2.3 M.W. Sabine Pass	Subgrade Sand	EA-CMS-2
6	20	Jasper	(64-7-21) SH 96	5.2 M.S of Sabine Pass To 2.4 M.N	Hot Mix Plant Site	AC-10
7	20	Jasper	(E-877-1-8) FM 255	1. 1 M. W. Sam Rayburn Dam	Subgrade Sand Layer "B"	RC-2
8	20	Jasper	(E-877-1-8) FM 255	1. 1 M. W. Sam Rayburn Dam	Subgrade Sand Layer "C"	RC-2
9	20	Newton	(3197-3-4) FM 255	2 M. E. of SH 87	100% Sand Pit No. 3	EA-CMS-2
10	15		IH 35	N. of Dilley Cotulla Tex	River Gravel - 1.5 inch max. size	AC-10
11	15	Frio	(276-7) US 57	ST#710 to St# 1271	Limestone Rock Asphalt	AC-10

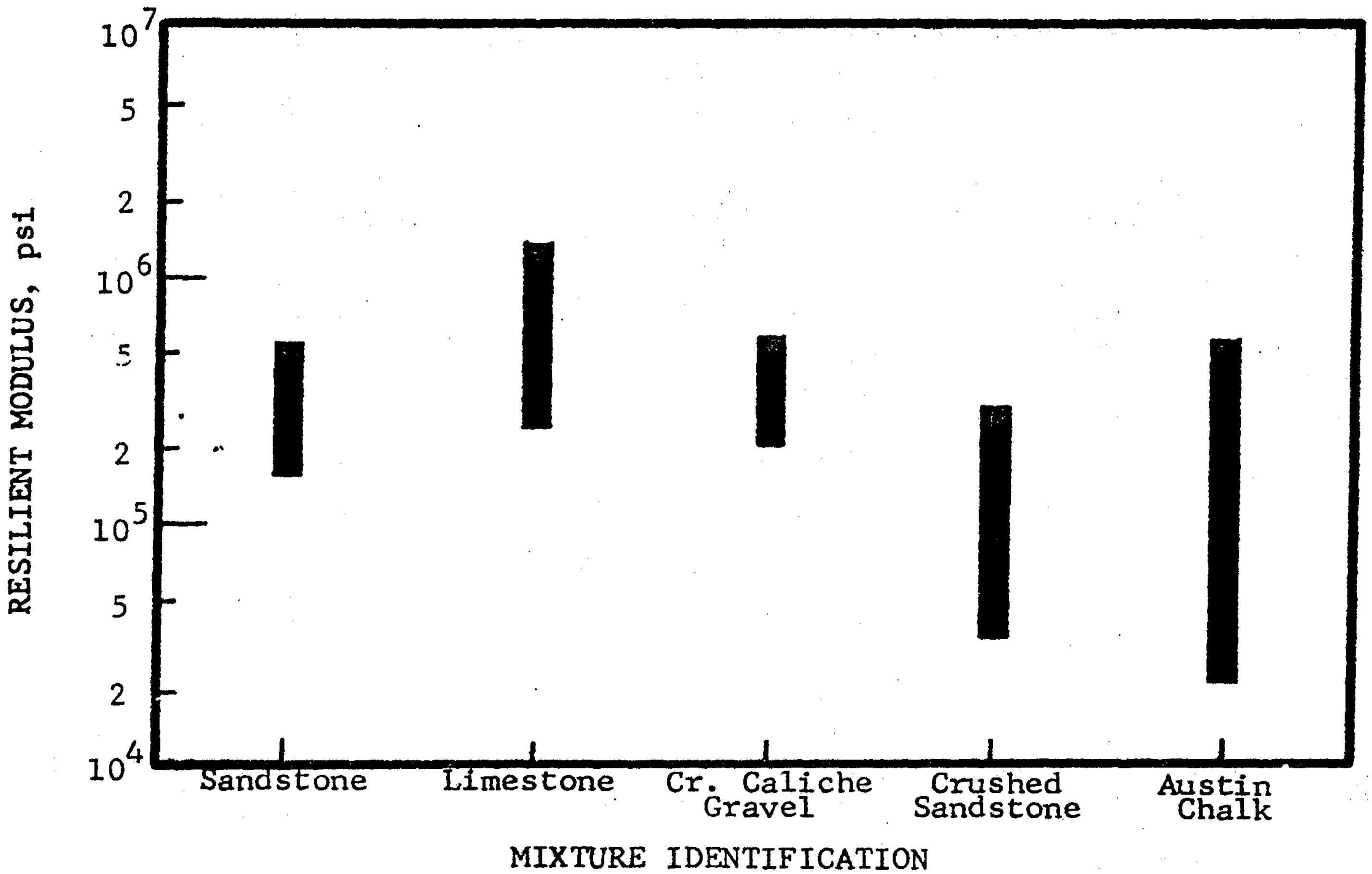

Figure 114. Resilient modulus of laboratory molded black base mixtures at 73^{o}F versus asphalt content (before vacuum saturation).

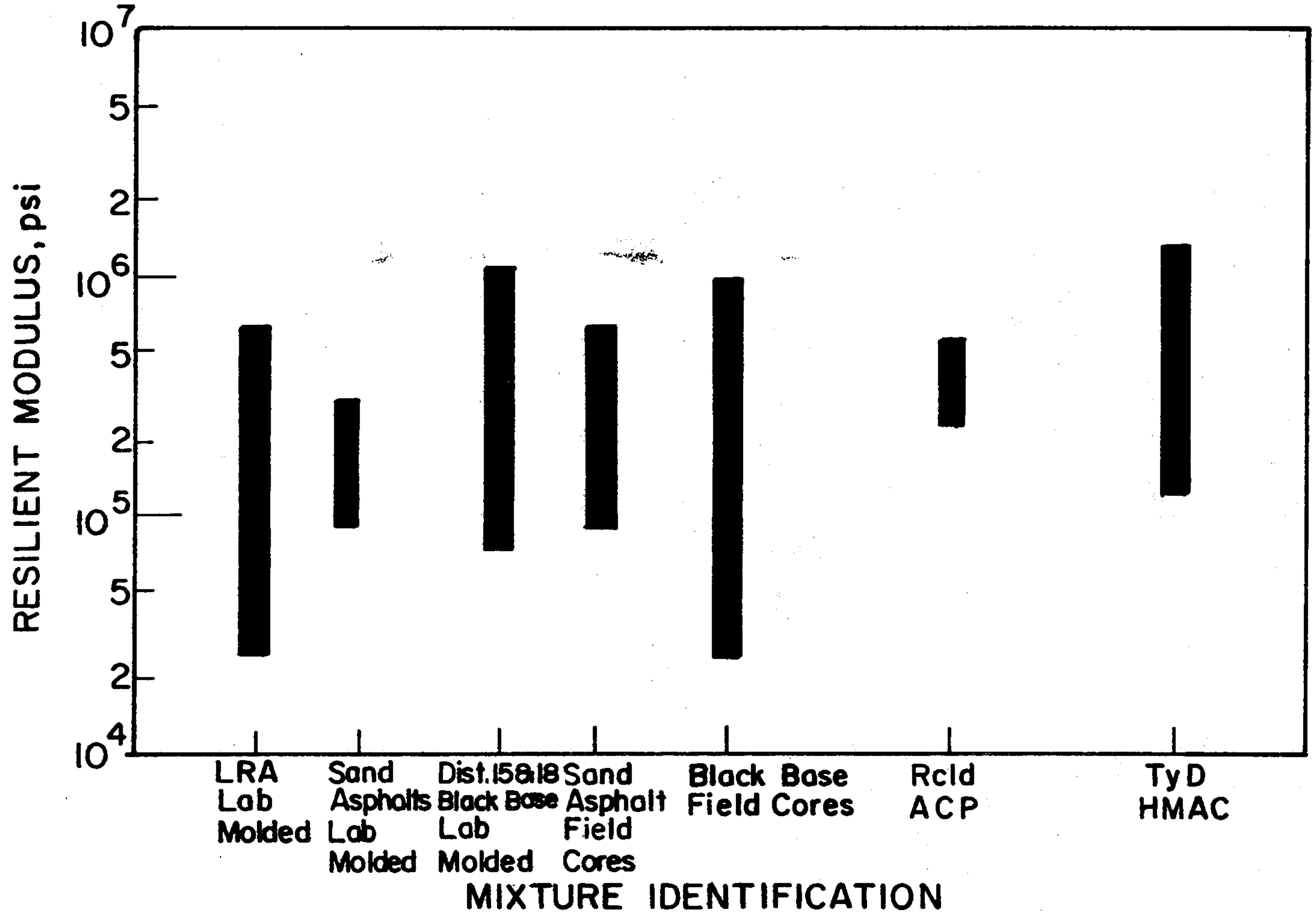

Figure 115. Comparison of resilient moduli of various bituminous mixtures at 73°F.

Table 7. Laboratory Test Results-Field Cores

Section No.	Dist.	Material I. D.	Sample#	Ave. Rice Sp. Gr.	Core Sp. Gr.	Air Voids	Test Temp	Hveem Stability	Marshall Stability	Marshall Flow	Resilient Modulus, Psix10^3 68°
1	11	SH103 Sand Asphalt	1	2.410	2.027	15.89	77°	41	7234	21	.170
			2		2.019	16.22		40	7498	21	0.94
			3		2.027	15.89	140°	24	805	13	.134
			4		1.962	18.59		23	876	13	.145
2	11	LP287 Sand Asphalt	1	2.393	1.857	22.40	77°	36	4657	20	.220
			2		1.865	22.06		38	4448	19.5	.134
			3		1.834	23.36	140°	20	594	17.5	.144
			4		1.873	21.73		19	403	17	.110
3	11	LP287 Sand Asphalt	1	2.385	1.951	18.20	77°	48	6204	19.5	.208
			2		1.953	18.11		53	6755	15	.217
			3		1.945	18.45	140°	30	1716	13	.324
			4		1.942	18.57		28	1529	14	.252
4	11	US287 Sand Asphalt	3	2.425	2.094	13.64	77°	46	6311	19	.748
			4		2.072	14.56		46	5518	20	.574
			1		2.096	13.69	140°	29	1267	14	.447
			2		2.053	15.34		23	977	15	.578
5	20	SH87 Sand Asphalt	A	2.405	1.982	17.59	77°	22	1539	34	.0840
			B		1.996	17.01		26	1576	25	.0844
			C		1.984	17.50	140°	17	2	12	.0869
			D		1.991	17.21		19	2	25	.102
6	20	SH96	A	2.369	1.966	17.01	77°	23	7350	20	.245
			B		1.987	16.12		24	7453	25	.188
			C		1.912	19.29	140°	*	879	17	.207
			D		1.935	18.31		*	515	30	.190
7	20	FM255 Layer B Sand Asphalt	A	2.317	1.790	22.74	77°	32	1016	17	
			E		1.795	22.53		35	1063	17	
			F		1.768	23.65	140°	25	1176	20	
			J		1.786	22.92		26	875	15	
8	20	FM255 Layer C Sand Asphalt 877-1-8	H	2.447	1.883	23.05	77°	33	112	22	.080
			D		1.891	22.72		35	354	15	.143
			F		1.908	22.03	140°	25	568	20	
			C		1.968	19.57		26	1002	19	.276

Table 7. cont.

Section No.	Dist.	Material I. D. Highway	Sample#	Ave. Rice Sp. Gr.	Core Sp. Gr.	Air Voids	Test Temp	Hveem Stability	Marshall Stability	Marshall Flow	Resilient Modulus, Psix10^3 68°
9	20	FM255	B	2.453	1.903	22.23	77°	25	680	20	
		3179-3 Sand	H		1.888	22.84		25	501	19	
		Asphalt	J		1.897	22.48	140°	18	17	35	
			K		1.892	22.68		21	17	20	
10	15	IH35	A-480	2.386	2.316	2.93	77°	49	10670	19	
		Black Base River	B-380		2.327	2.47		25	10608	22	
		Gravel	B-480		2.333	2.22	140°	28	1144	16	
		1½ max. sizw	C-380		2.312	3.10		21	1407	15	
11	15	IH57	2-890	2.446	1.960	19.86	77°	28	2175	26	.054
		Limestone Rock	6-912		1.995	18.44		42	3360	16	.204
		Asphalt	4-830		2.037	16.72	140°	35	395	7	.217
			5-1012		1.989	18.68		23	229	21	.151

*too low to measure.

adequately tested to determine its load carrying ability and its resistance to the action of water.

As noted on Figure 115, both the sand asphalt materials and the marginal materials for District 15 and 18 are within the range of data from black base field cores. Thus, it appears reasonable that this marginal materials may be suitable for use under certain loading and environmental conditions.

Figure 116 shows a comparison of stability values for laboratory molded sand asphalt mixes and materials from Districts 15 and 18 together with sand asphalt field cores. Laboratory molded sand asphalts have stability values within the range of values obtained from the field cores. Stability values of laboratory molded materials from Districts 15 and 18 were above those of the sand asphalt materials.

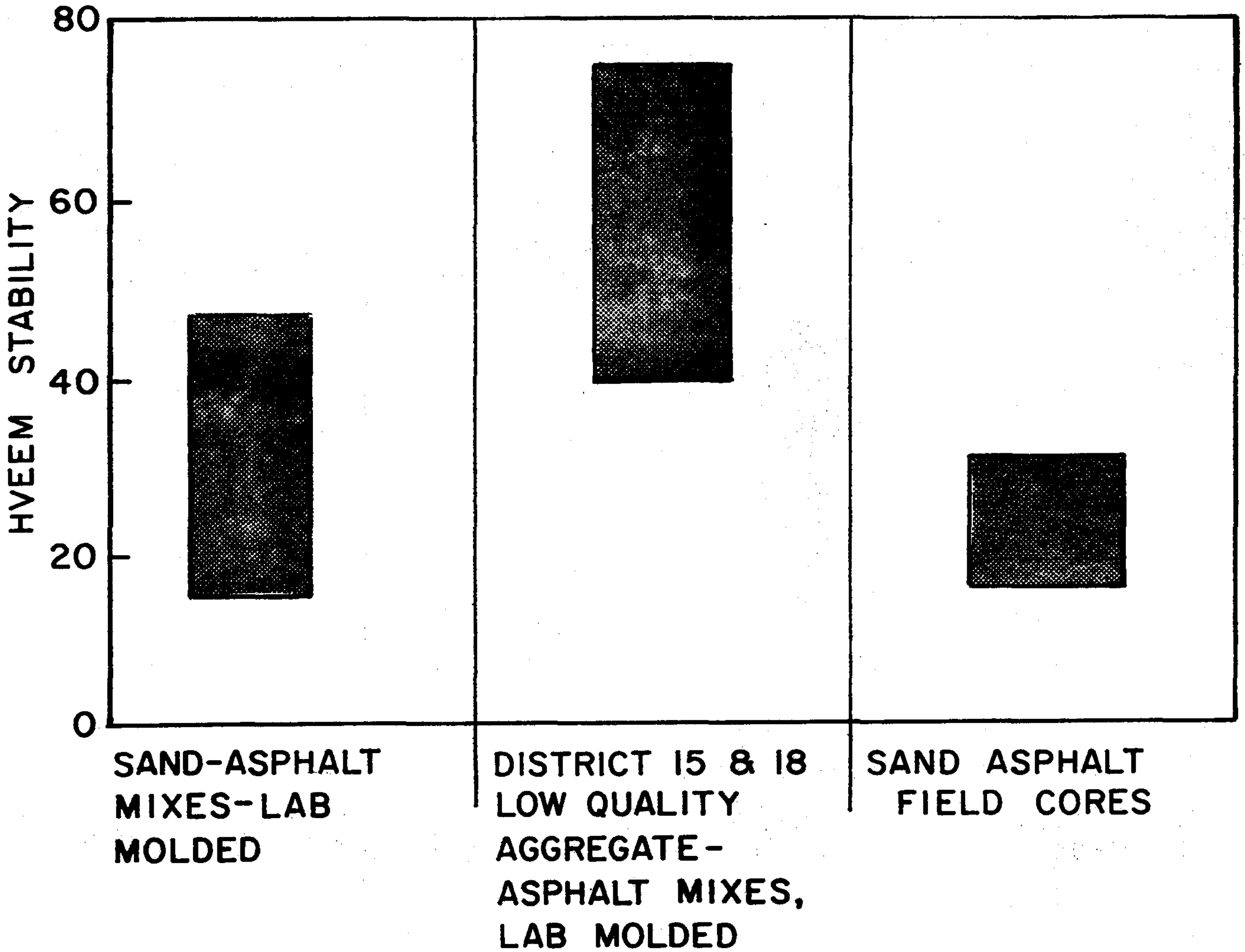

Figure 116. Comparison of typical Hveem stability values ranges for the sand aggregate, low quality aggregates and sand asphalt field cores.

ECONOMICS

The performance of black base pavements in Texas has been favorable and many Texas highway personnel are interested in using this type of pavement system provided it can be economically justified. Due to recent price increases in asphaltic concrete, flexible bases are in many cases more economical than asphalt treated bases (based on initial construction costs). A review of cost information in 1972 indicates that the price of asphalt concrete was between six and eight dollars per ton, black base between five and seven dollars per ton, and quality flexible bases between three and six dollars per ton (5). Using an equivalency of one inch of black base to replace one and one-half to two-inches of flexible base as is commonly practiced (47), figures indicate that black bases were acceptable economic substitutes for flexible bases.

Bituminous treated base costs have escalated much more rapidly than untreated flexible base courses. The cost of asphalt cement alone has increased from thirty dollars per ton in 1971 to approximately ninety dollars per ton today. Table 8 indicates that material costs comprise approximately fifty percent of the cost of producing hot mixed asphalt concrete (48).

In an attempt to reduce black base pavement costs, the following design was investigated (49):

Layer 1 - Thin high quality bituminous surface.

Layer 2 - Thick moderate quality bituminous base.

Layer 3 - Thin high quality bituminous subbase.

Layer 4 - Subgrade.

Table 8. Component cost of asphaltic concrete.

Component Cost Item	Percent of In-Place Cost
Plant Labor	4.05
Plant Fuel	0.19
Plant Expense	15.06
Dryer Fuel	2.32
General Overhead	1.35
Laydown Cost	11.58
Materials (Aggregate and Asphalt)	50.97
Haul to Job	14.48

The pavement design was investigated using fatigue life criteria.

To determine the fatigue life, a 9 kip dual wheel load (18 kip single axle load) was assumed and Chevron's multilayered elastic theory computer program (45) was used to determine the critical strain in each layer. Using the critical tensile strain in each bituminous layer and the critical compressive strain in the subgrade, the number of 18 kip equivalent axle loads to failure, N, in each layer was determined using the criteria proposed in March 1975 by Santucci (50).

The thickness and moduli of each of the three layers above the subgrade may be varied to achieve a specified design life. The critical strain in each layer for each of seventy-seven designs was computed. Using a log model regression analysis, an estimated design life for each design was determined.

A computer program was written which would solve any one of the design parameters given a design N and the other five design parameters. Assuming an N equal to 2,000,000 18 kip equivalent axle loads*, the design thicknesses for vaious pavement sections is illustrated in Figures 117-124. The following designs illustrate the use of the computer program.

Design 1. (Figure 117) It is assumed that the modulus of elasticity of the surface course, E_1, is 800,000 psi, the thickness of the surface course, D_1, is 2.0 inches, the thickness of the third layer (high quality bituminous subbase), D_3, is 2.0 inches, and the modulus of elasticity of the subgrade, E_S, is 5,000 psi. By using a bituminous base course with a modulus of elasticity, E_2, of 200,000 psi and a bituminous subbase with a

*Many existing black base sections in District 5 and 25 have expected N's of the order of 2,000,000; then is, 100,000 18 Kip equivalent 18 Kip axle loads per year for 20 years.

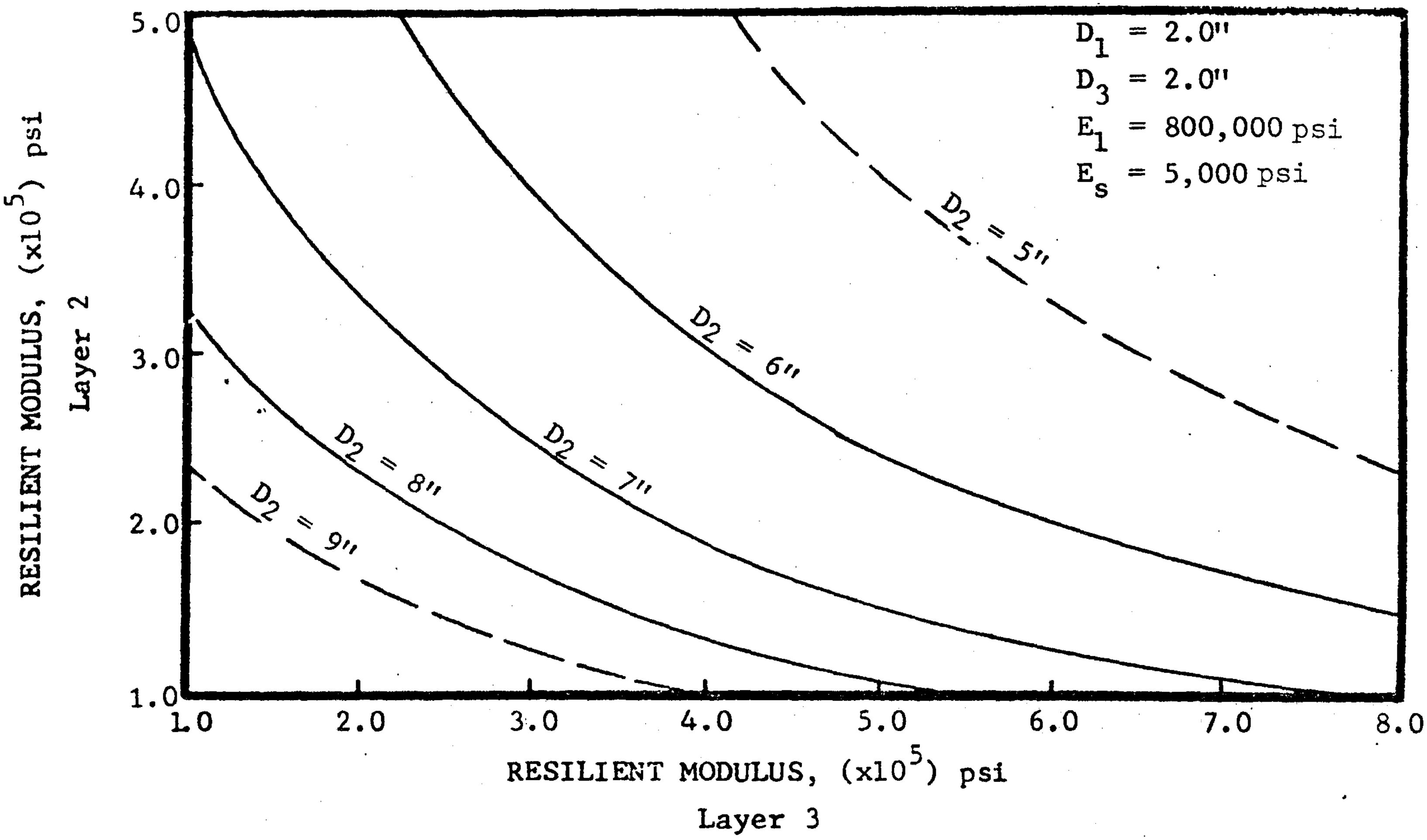

Figure 117. Thickness design chart for pavements containing asphalt stabilized bases (D_1=2.0", D_3=2.0", E_1=800,000 psi).

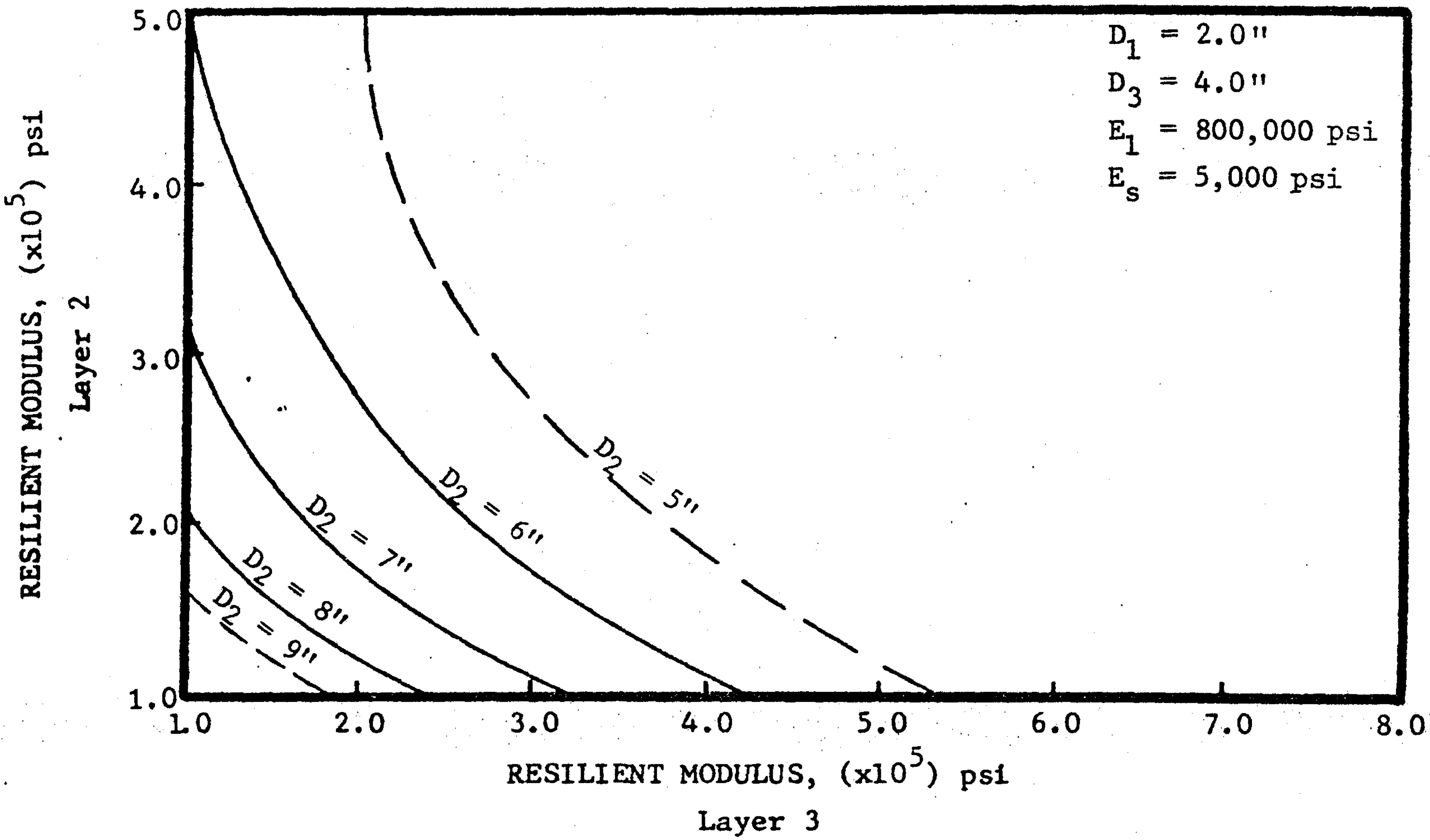

Figure 118. Thickness design chart for pavements containing asphalt stabilized bases (D_1=2.0", D_3=4.0", E_1=800,000 psi).

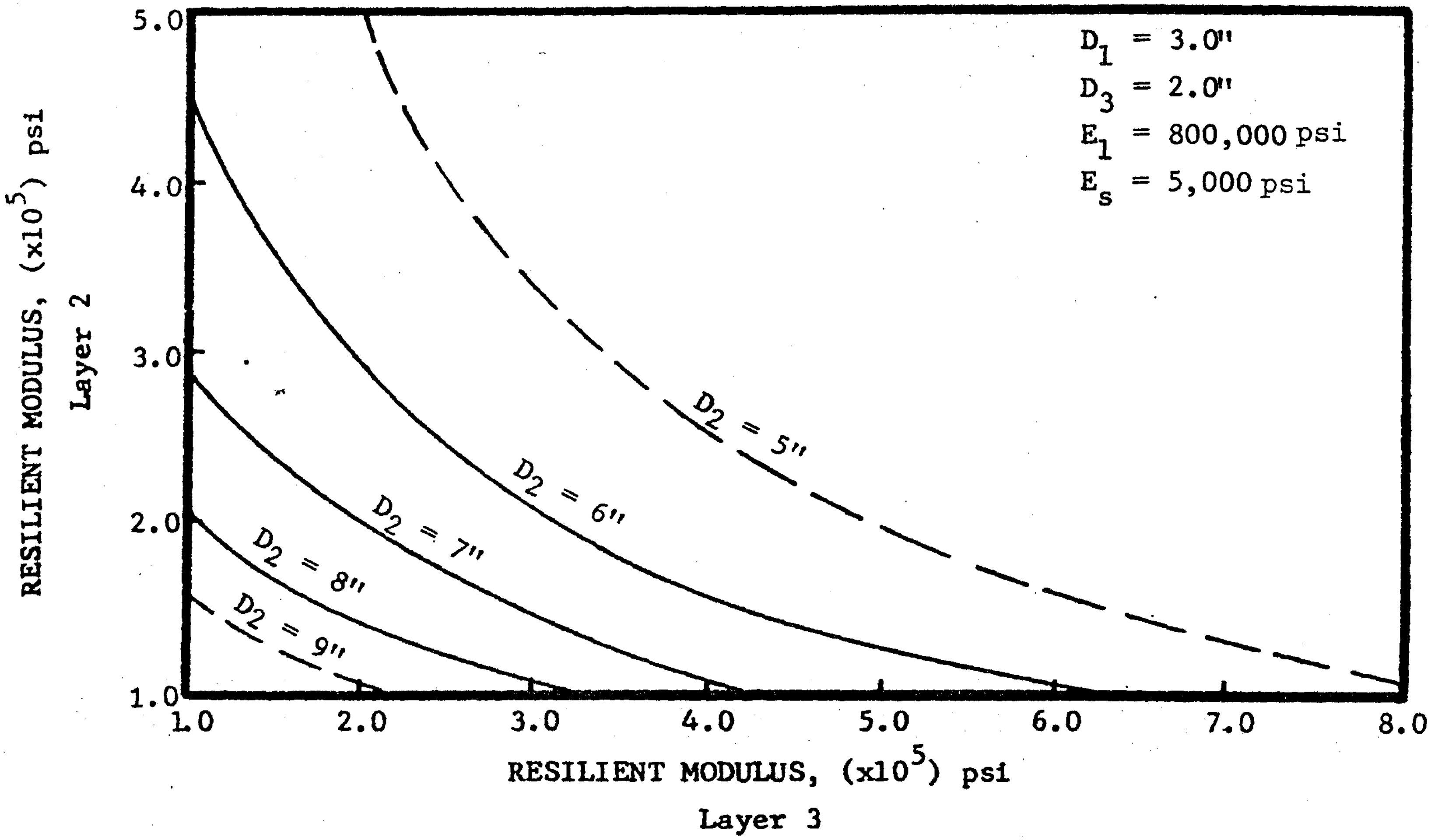

Figure 119. Thickness design chart for pavements containing asphalt stabilized bases (D_1=3.0", D_3=2.0", E_1=800,000 psi).

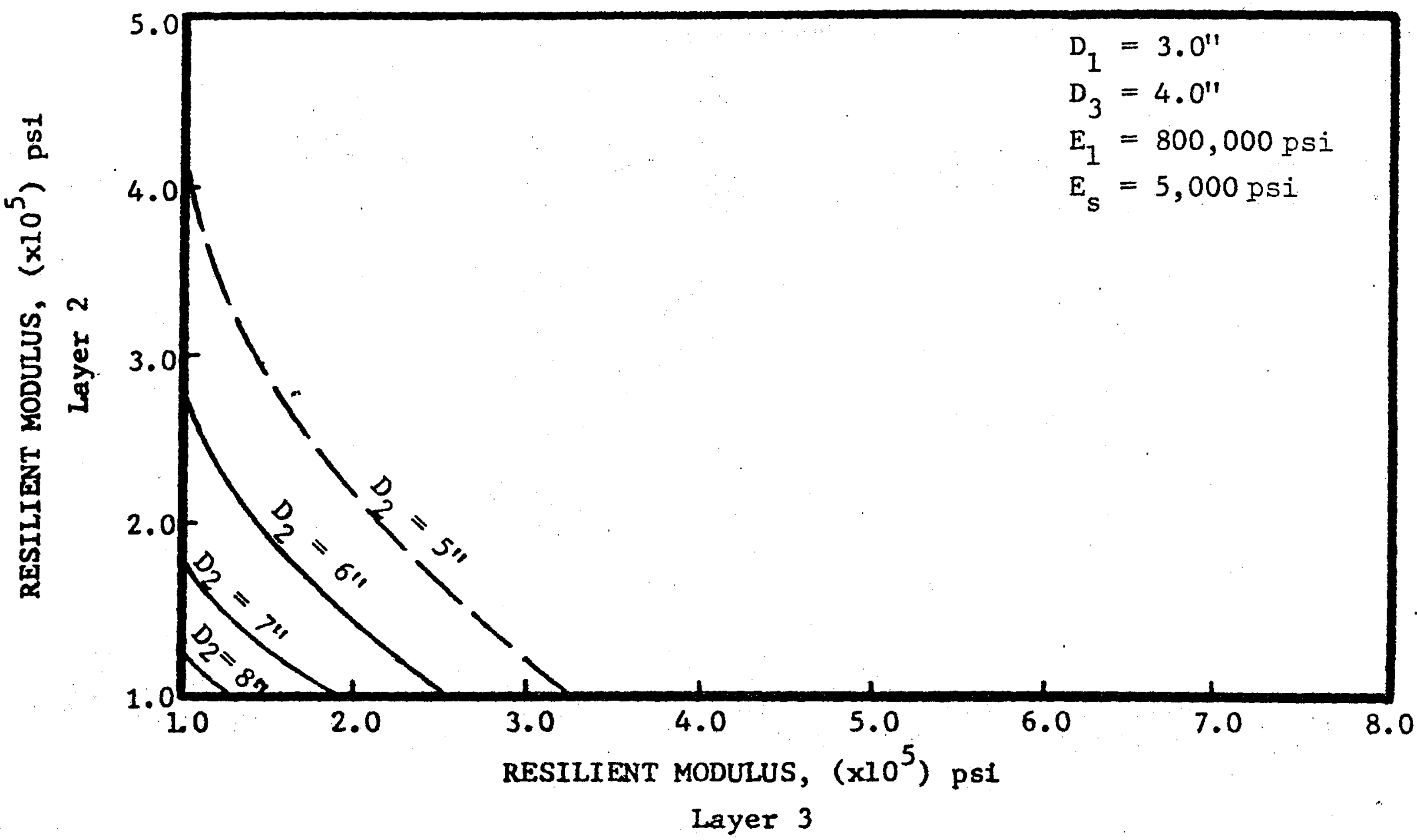

Figure 120. Thickness design chart for pavements containing asphalt stabilized bases (D_1=3.0", D_3=4.0", E_1=800,000 psi).

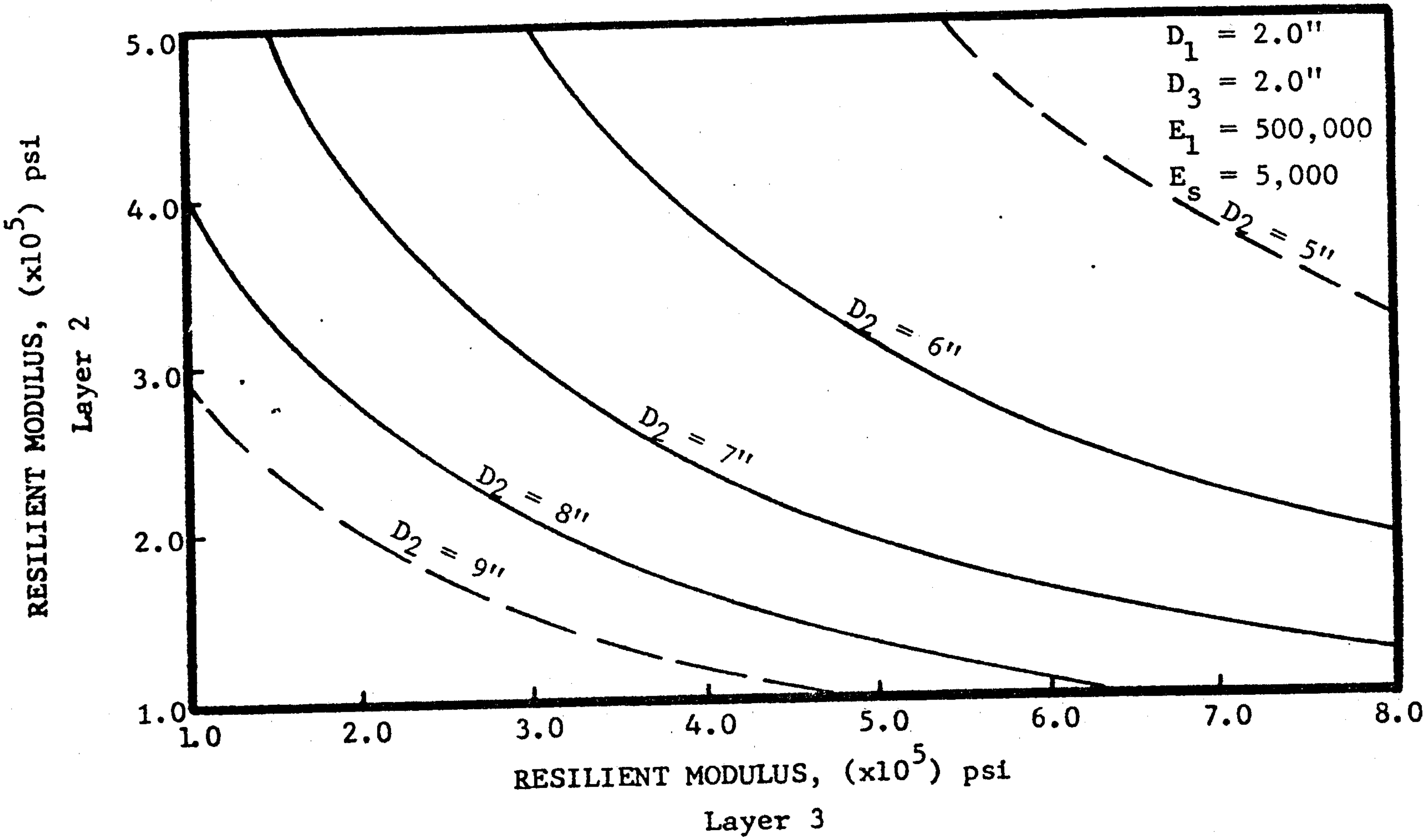

Figure 121 Thickness design chart for pavements containing asphalt stabilized bases (D_1=2.0", D_3=2.0", E_1=500,000 psi).

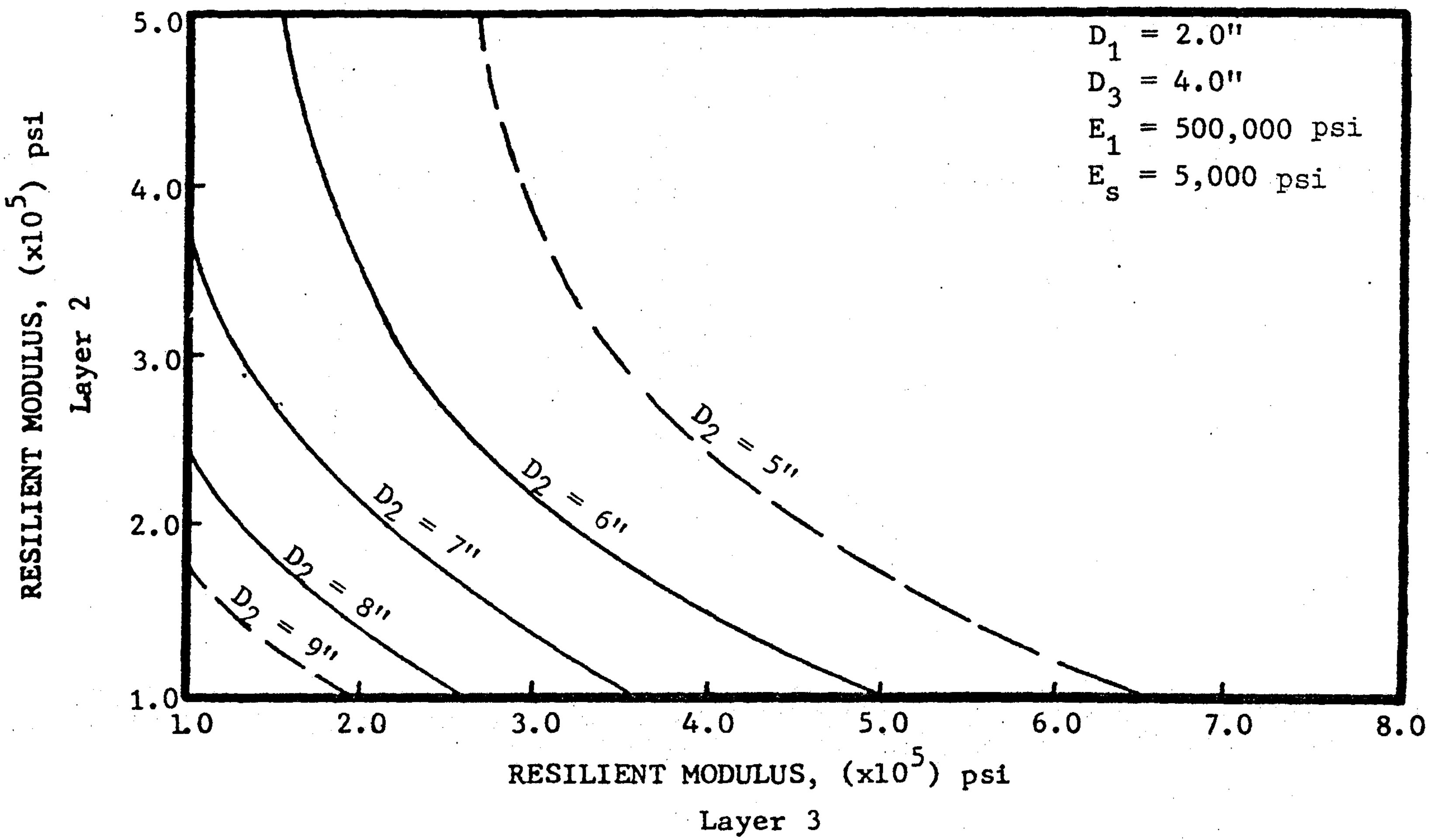

Figure 122. Thickness design chart for pavements containing asphalt stabilized bases (D_1=2.0",D_3=4.0", E_1=500,000 psi).

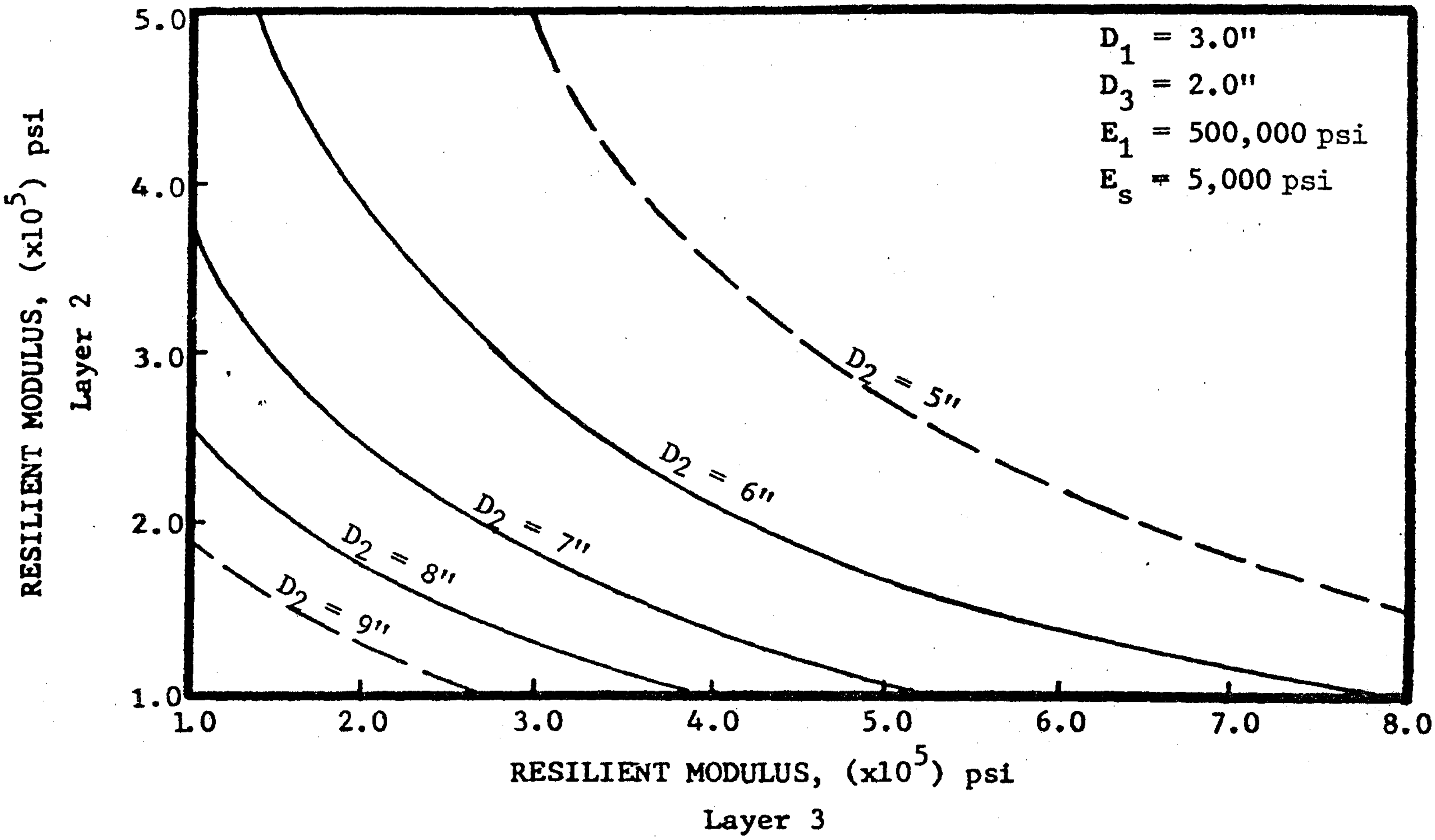

Figure 123. Thickness design chart for pavements containing asphalt stabilized bases (D_1=3.0", D_3=2.0", E_1=500,000 psi).

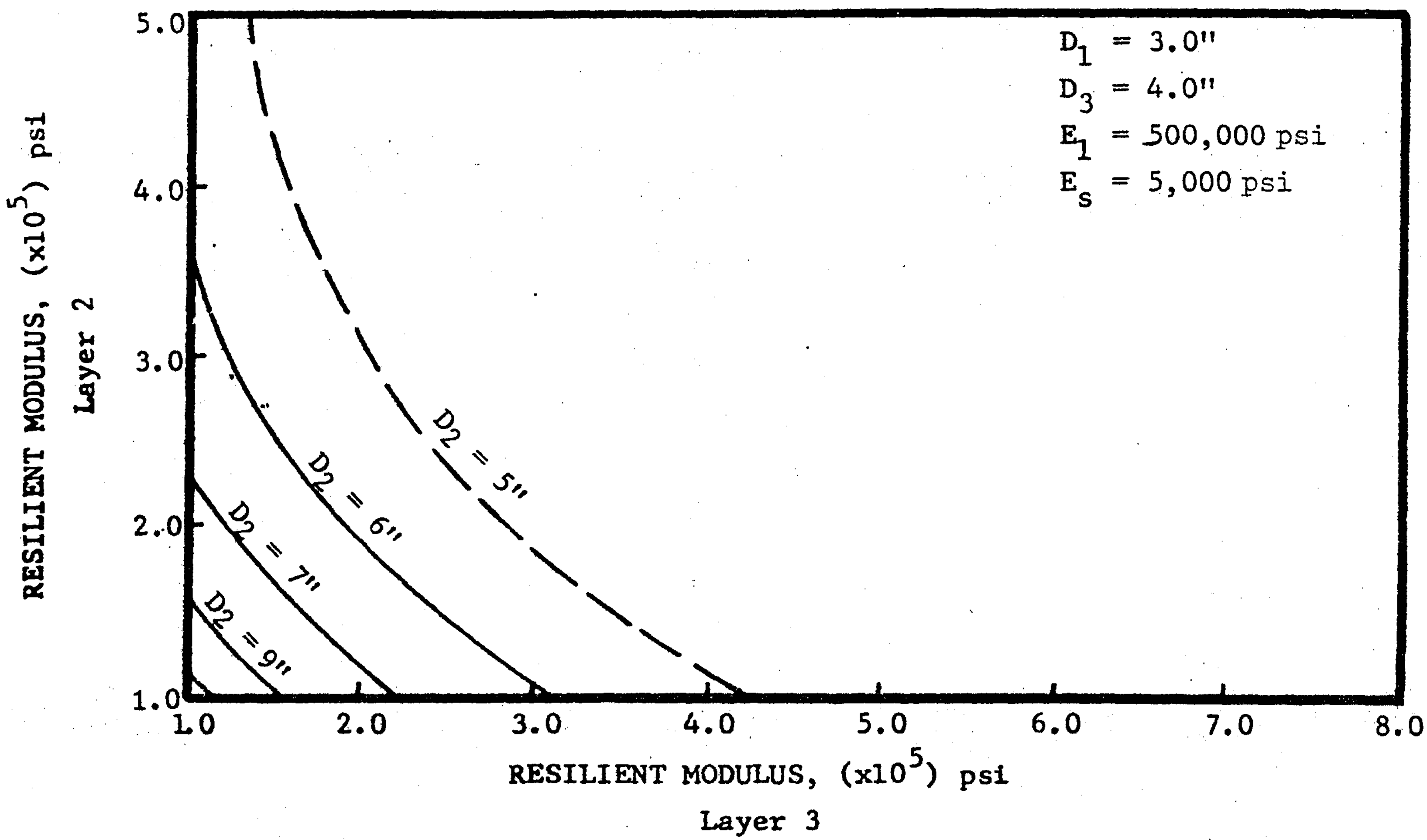

Figure 124. Thickness design chart for pavements containing asphalt stabilized bases (D_1=3.0", D_3=4.0", E_1=500,000 psi).

modulus of elasticity, E_3, of 400,000 psi, the design would require seven inches of bituminous base material, D_2.

Design 2. Using the assumed criteria in Design 1 but increasing E_2 to 300,000 psi would result in a base thickness of six inches and a savings of one inch of black base material.

Design 3. (Figure 119). Using the parameters in Design 1 and increasing the surface thickness, D_1, to 3.0 inches, results in D_2 approximately equal to 5.5 inches and a savings of 1.5 inches of base material. Thus, an economic comparison is necessary to determine whether or not it would be economically justifiable to increase D_1 by one inch in order to reduce D_2 by one and one-half inches.

Design 4. (Figure 121). Again using the parameters of Design 1 but reducing the modulus of elasticity of the surface course, E_1, to 500,000 psi, would result in a base thickness, D_2, of 7.5 inches. Thus, it would be possible to use a considerably lower quality surface material and increase the base layer thickness by only 0.5 inches.

From the four design examples above, it appears that an infinite number of workable solutions are obtainable. Undoubtably, the design engineer must acquaint himself with current highway cost information in order that the "optimum" pavement system be selected.

Asphalt concrete and black base prices (51, 52) began to increase rapidly in January of 1973; however, since May of 1975 these prices have somewhat levelled off (Figures 125, 128).

In September of 1975, the "in place" price of black base was fifteen dollars per ton (based on 6.0 percent asphalt by total weight of the mix), the price of asphalt concrete was eighteen dollars per ton (again based on

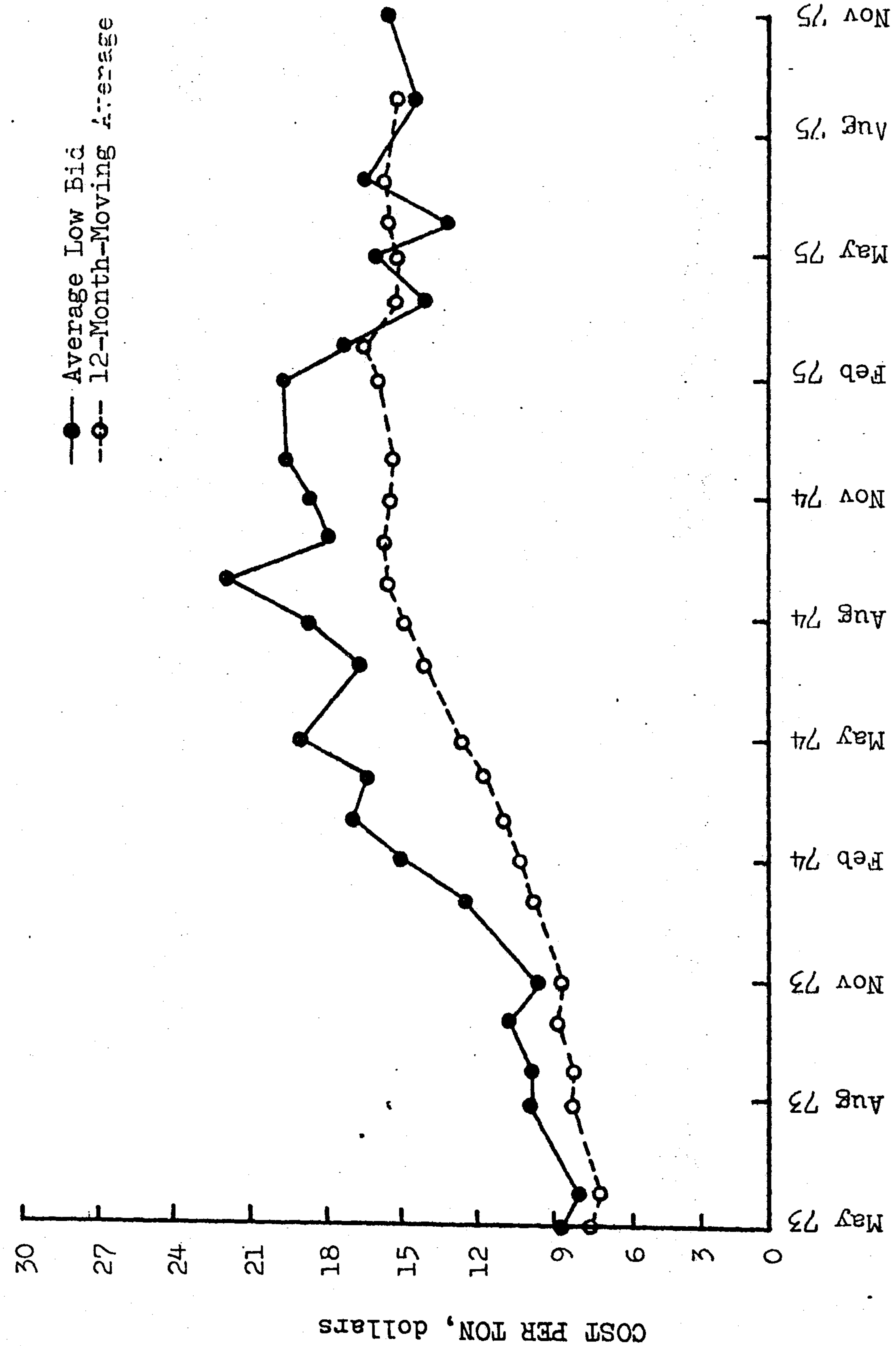

Figure 125. Black base cost information.

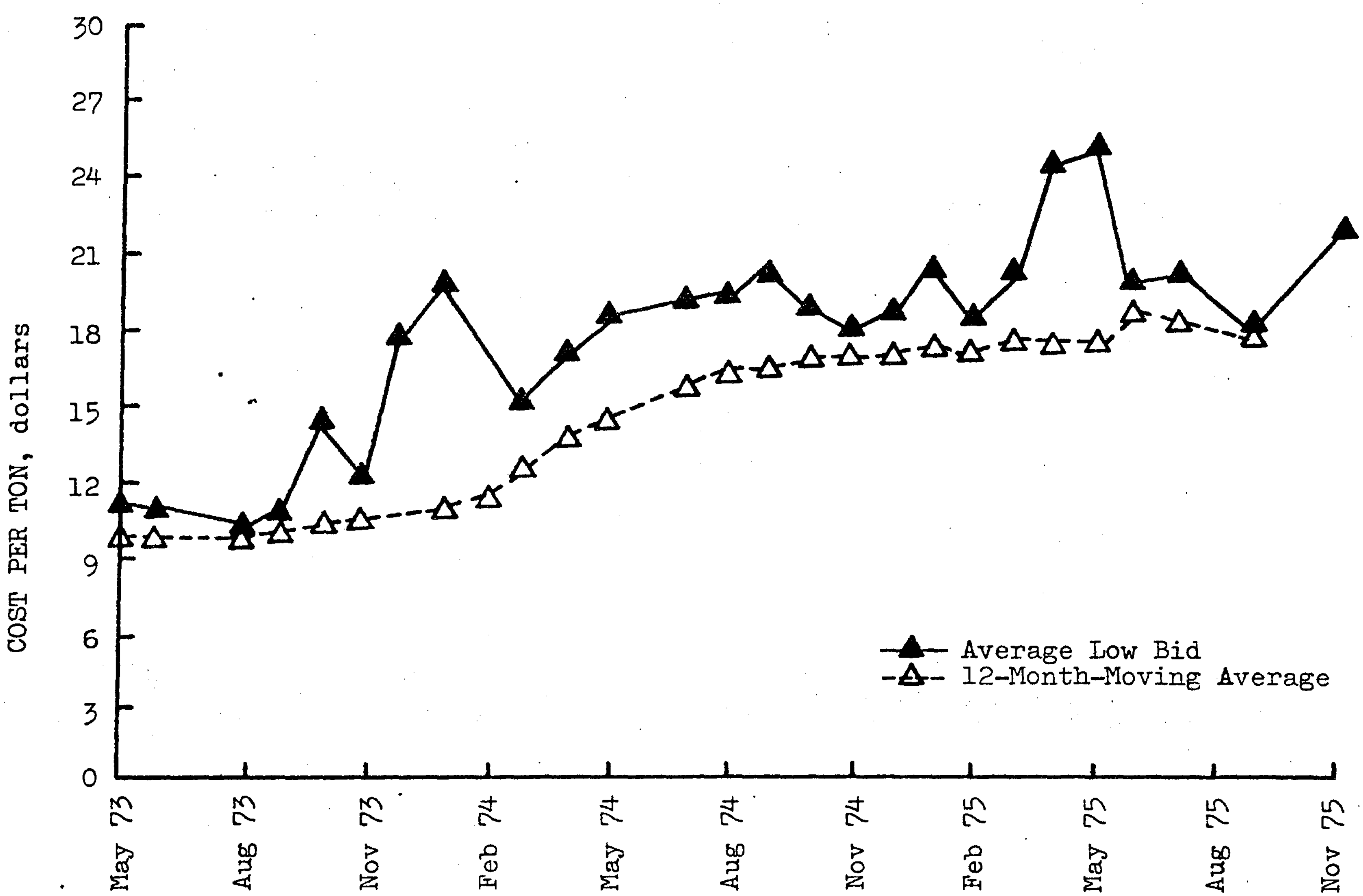

Figure 126. Asphalt concrete cost information.

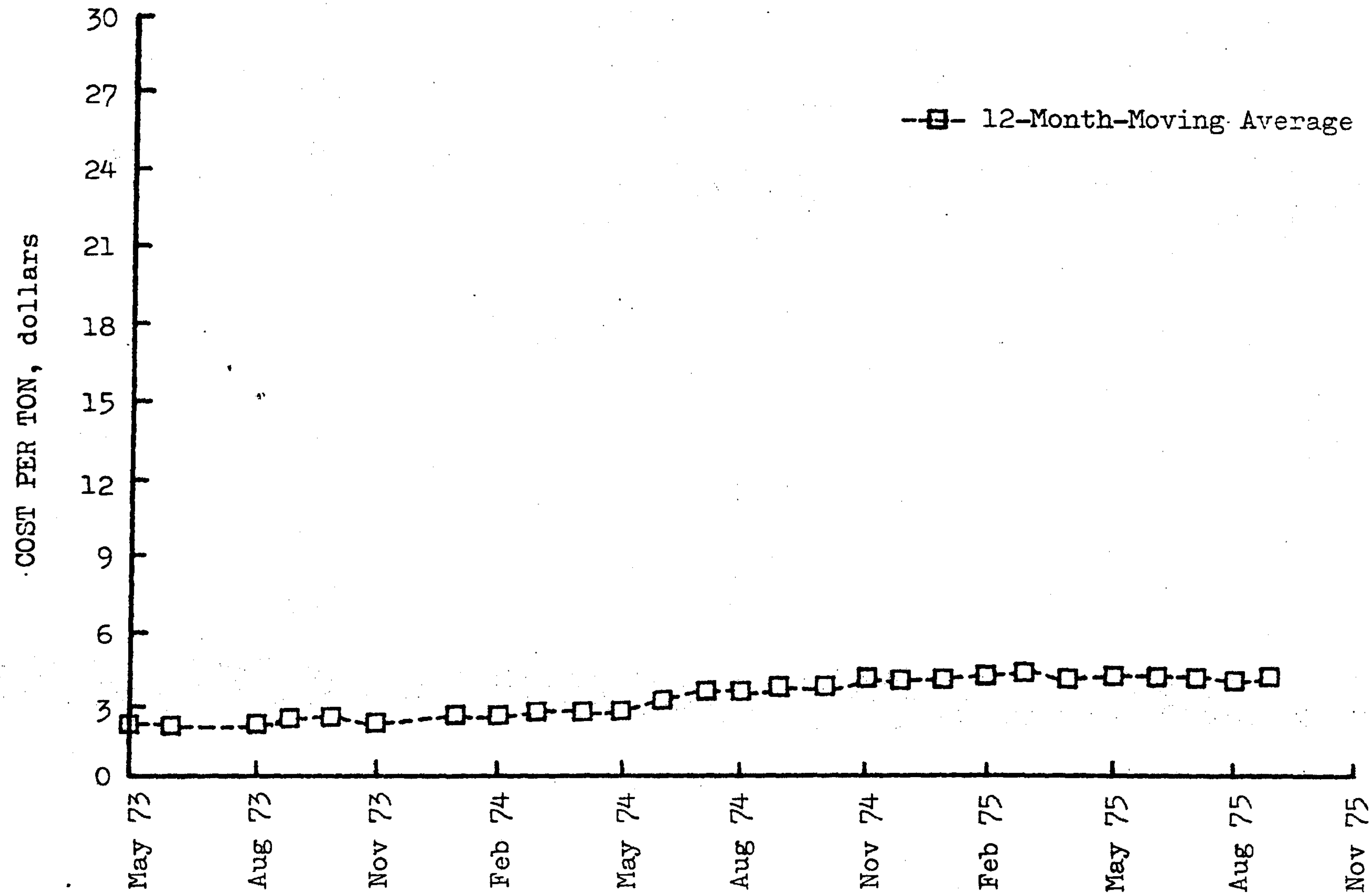

Figure 127. Flexible base course cost information

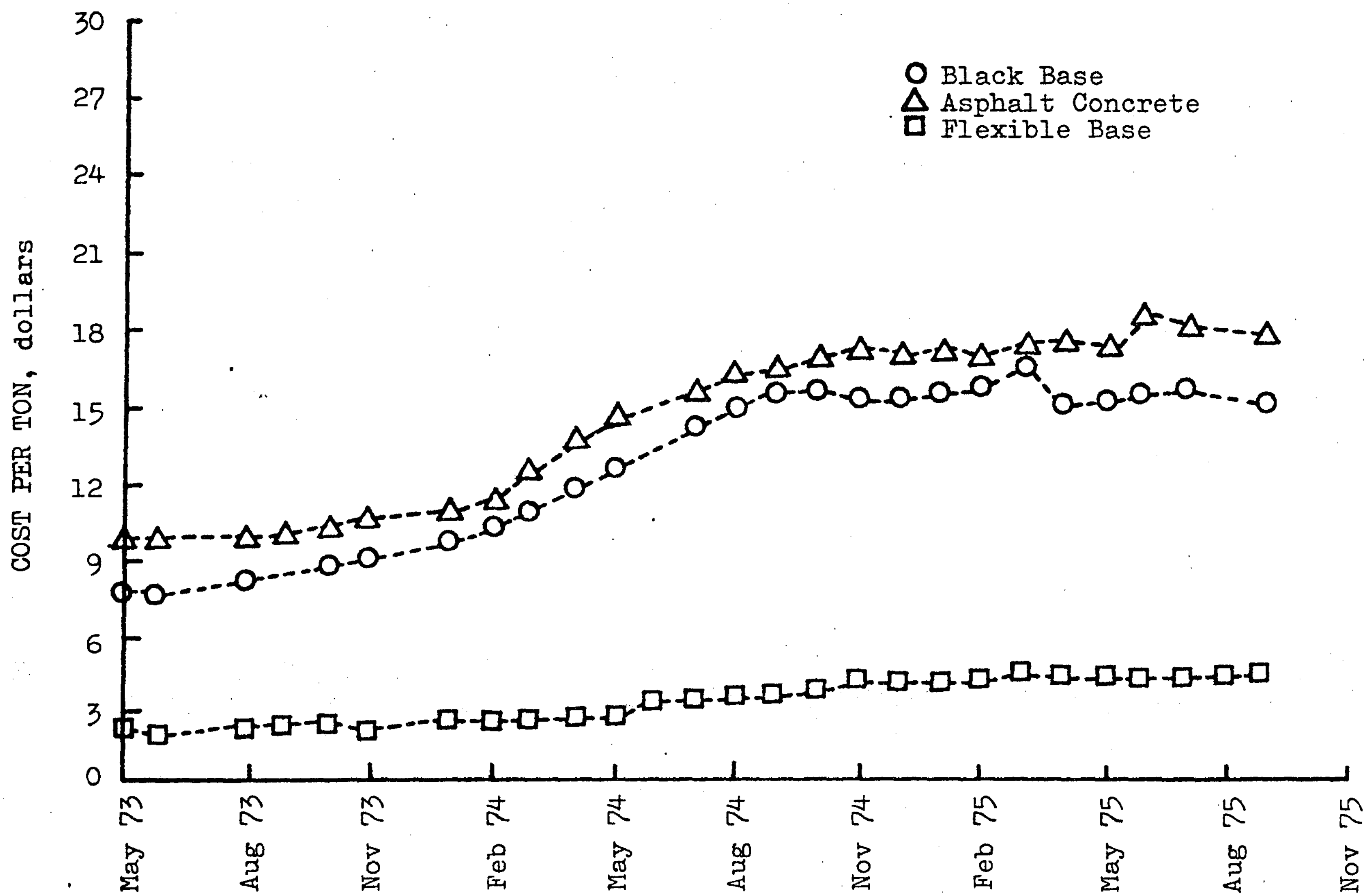

Figure 128. Base course cost information (12-Month-Moving Average).

6.0 percent asphalt), and high-quality flexible base material was five dollars per ton. Using these prices and the test results previously presented a cost analysis of the laboratory molded black base mixtures was conducted.

The purpose of the analysis was to determine the cost effect of reducing the asphalt content in the black base layer. The following assumptions were made:

1. The resilient moduli values for the various materials are those obtained after vacuum saturation. The resilient modulus of the black base mixutres varies with asphalt content.

2. Using Figure 121, the following pavement section was evaluated.

$D_1 = 2.0''$	$E_1 = M_{R1} = 500{,}000$ psi
$D_2 = ?$	$E_2 = M_{R2} =$ varies with asphalt content
$D_3 = 2.0''$	$E_3 = M_{R3} = 500{,}000$ psi
XXXXX	

3. The following "in place" prices were assumed:
   a. The asphalt concrete surface course and the high quality bituminous subbase costs were 18 dollars per ton each and at two-inch lift thicknesses the costs were 1.96 dollars per square yard for each (Table 9).

   b. While the aggregate cost was 10.50* dollars per ton and the asphalt cement price was 86.90 dollars per ton, the price of the black base (layer 2) was 15.00 dollars per ton based on a 6.0 percent asphalt content (by total weight of the mix). The price per ton of black base, of course, changes with asphalt content.

The next set of designs are for the Garner-Ross sandstone mixtures and are designed based on the assumptions previously listed.

*for marginal aggregates, the cost of aggregates may be reduced substantially.

Table 9. Costs per square yard of asphalt concrete pavement courses* (53).

Cost per ton	Thickness of Pavement Course, in.												
	1/2	1	2	3	4	5	6	7	8	9	10	11	12
0.50	0.01	0.03	0.05	0.08	0.11	0.14	0.16	0.19	0.22	0.24	0.27	0.30	0.33
1.00	0.03	0.05	0.11	0.16	0.22	0.27	0.33	0.38	0.44	0.49	0.54	0.60	0.65
2.00	0.05	0.11	0.22	0.33	0.44	0.54	0.65	0.76	0.87	0.98	1.09	1.20	1.30
3.00	0.08	0.16	0.33	0.49	0.65	0.82	0.98	1.14	1.30	1.47	1.63	1.79	1.96
4.00	0.11	0.22	0.44	0.65	0.87	1.09	1.30	1.52	1.74	1.96	2.18	2.39	2.61
5.00	0.14	0.27	0.54	0.82	1.09	1.36	1.63	1.90	2.18	2.45	2.72	2.99	3.26
7.00	0.19	0.38	0.76	1.14	1.52	1.90	2.28	2.66	3.04	3.43	3.81	4.19	4.57
8.00	0.22	0.44	0.87	1.30	1.74	2.18	2.61	3.04	3.48	3.92	4.35	4.78	5.22
9.00	0.24	0.49	0.98	1.47	1.96	2.45	2.94	3.43	3.92	4.40	4.89	5.38	5.87
10.00	0.27	0.54	1.09	1.63	2.18	2.72	3.26	3.81	4.35	4.89	5.44	5.98	6.52
11.00	0.30	0.60	1.20	1.79	2.39	2.99	3.59	4.19	4.78	5.38	5.98	6.58	7.18
12.00	0.33	0.65	1.30	1.96	2.61	3.26	3.92	4.57	5.22	5.87	6.52	7.18	7.83
13.00	0.35	0.71	1.41	2.12	2.83	3.53	4.24	4.95	5.66	6.36	7.07	7.78	8.48
14.00	0.38	0.76	1.52	2.28	3.04	3.81	4.57	5.33	6.09	6.85	7.61	8.37	9.14
15.00	0.41	0.82	1.63	2.45	3.26	4.08	4.89	5.71	6.52	7.34	8.16	8.97	9.79
16.00	0.44	0.87	1.74	2.61	3.48	4.35	5.22	6.09	6.96	7.83	8.70	9.57	10.44
17.00	0.46	0.92	1.85	2.77	3.70	4.62	5.55	6.47	7.40	8.32	9.24	10.17	11.09
18.00	0.49	0.98	1.96	2.94	3.92	4.89	5.87	6.85	7.83	8.81	9.79	10.77	11.74
19.00	0.52	1.03	2.07	3.10	4.13	5.17	6.20	7.23	8.26	9.30	10.33	11.36	12.40
20.00	0.54	1:09	2.18	3.26	4.35	5.44	6.52	7.61	8.70	9.79	10.88	11.96	13.05
21.00	0.57	1.14	2.28	3.43	4.57	5.71	6.85	7.99	9.14	10.28	11.42	12.56	13.70
22.00	0.60	1.20	2.39	3.59	4.79	5.98	7.18	8.37	9.57	10.77	11.96	13.16	14.36
23.00	0.63	1.25	2.50	3.75	5.00	6.25	7.50	8.75	10.00	11.26	12.51	13.76	15.01
24.00	0.65	1.30	2.61	3.92	5.22	6.52	7.83	9.14	10.44	11.74	13.05	14.36	15.66
25.00	0.68	1.36	2.72	4.08	5.44	6.80	8.16	9.52	10.88	12.23	13.59	14.95	16.31
30.00	0.82	1.63	3.26	4.89	6.52	8.16	9.79	11.42	13.05	14.68	16.31	17.94	19.58
35.00	0.95	1.90	3.81	5.71	7.61	9.52	11.42	13.32	15.22	17.13	19.03	20.93	22.84
40.00	1.09	2.18	4.35	6.52	8.70	10.88	13.05	15.22	17.40	15.58	21.75	23.92	26.10
45.00	1.22	2.45	4.89	7.34	9.79	12.23	14.68	17.13	19.58	22.02	24.47	26.92	29.36
50.00	1.36	2.72	5.44	8.16	10.88	13.59	16.31	19.03	21.75	24.47	27.19	29.91	32.62

*Assumed density = 145 lb per ft^3

Design 5. Evaluate the cost of the entire pavement section when the black base layer is at optimum asphalt content. At asphalt content = 9.0 percent, M_{R2} = 500,000 psi. From Figure 121, the required thickness of the black base layer D_2, is 5.2 inches.

Asphalt cost = 0.09 x \$86.90/ton = \$7.82/ton

Aggregate cost = 0.91 x \$10.50/ton = \$9.56/ton

Total cost of black base layer = \$17.38/ton

The cost of 5.2 inches of black base at \$17.38 per ton is \$4.92 per square yard (53). The cost of 2.0 inches of hot-mixed asphalt concrete plus 2.0 inches of high quality bituminous subbase both at \$18.00 per ton is: \$1.96 per square yard + \$1.96 per square yard = \$3.92 per square yard. The total cost of the pavement section is: (\$4.92 + \$3.92) per square yard = \$8.84 per square yard.

Design 6. Reducing the asphalt content to 8.5 percent reduces the M_{R2} to 400,000 psi and D_2, therefore, is equal to 5.6 inches.

Asphalt cost = 0.085 x \$86.90/ton = \$7.39/ton

Aggregate cost = 0.915 x \$10.50/ton - \$9.61/ton

Total cost of black base layer = \$17.00/ton

The cost of 5.6 inches of black base at \$17.00 per ton is \$5.18 per square yard. As previously noted, the cost of the surface layer and subbase layer was assumed to remain constant at \$3.92 per square yard. Therefore, the total cost of the pavement section is: (\$5.18 + \$3.92) per square yard = \$9.10 per square yard.

Design 7. Reduce the asphalt content to 8.0 percent; therefore, M_{R2} = 200,000 psi and D_2 = 6.8 inches.

Asphalt cost = 0.08 x \$86.90/ton = \$6.95/ton

Aggregate cost = 0.92 x \$10.50/ton = \$9.66/ton

Total cost black base layer = \$16.61/ton

The cost of 6.8 inches of black base at \$16.61 per ton is \$6.10 per square yard. Therefore, the total cost of the pavement section is: (\$6.10 + \$3.92) per square yard = \$10.02 per square yard.

<u>Design 8</u>. Reduce the asphalt content to 6.0 percent; therefore, M_{R2} = 100,000 psi and D_2 = 9.0 inches.

Asphalt cost = 0.06 x \$86.90/ton = \$5.21/ton

Aggregate cost = 0.94 x \$10.50/ton = \$9.87/ton

Total cost of black base layer = \$15.08/ton

The cost of 9.0 inches of black base at \$15.08 per ton is \$7.35 per square yard. Therefore, the total cost of the pavement section is: (\$7.35 + \$3.92) per square yard = \$11.27 per square yard.

<u>Design 9</u>. Evaluate the cost at 10.4 percent asphalt. At 10.4 percent asphalt, the M_{R2} = 200,000 psi and D_2 = 6.9 inches.

Asphalt cost = 0.104 x \$86.90/ton = \$9.04/ton

Aggregate cost = 0.896 x \$10.50/ton = \$9.41/ton

Total cost of black base layer = \$18.45/ton

The cost of 6.9 inches of black base at \$18.45 per ton is \$7.02 per square yard. Therefore, the total cost of the pavement section is: (\$7.02 + \$3.92) per square yard = \$10.94 per square yard.

Table 10 illustrates the results obtained in Designs 5 through 9. Obviously, the most economical pavement section for the Garner-Ross sandstone mixtures is the design at optimum asphalt content (Design 5). The above

Table 10. Cost analysis of Garner-Ross sandstone mixtures.

Design Number	Percent Asphalt, by weight of mix	M_R After Vacuum Saturation, psi (73°F)	Required Thickness for Black Base Layer D_2, inches	Total Cost per Square Yard for Black Base Pavement
5	9.0 (optimum)	500,000	5.2	$8.84
6	8.5	400,000	5.6	$9.10
7	8.0	200,000	6.8	$10.02
8	6.0	100,000	9.0	$11.27
9	10.4	200,000	6.9	$10.94

results may be further illustrated by evaluating the costs of another laboratory molded black base mixture.

The following set of designs illustrate the costs of the "74" Ranch pit, crushed sandstone mixtures. The assumptions used in the analysis of the sandstone mixtures also apply to the crushed sandstone analysis.

Design 10. Evaluate the cost of the pavement section for the case where the black base material is at optimum asphalt content. At 9.5 percent asphalt, M_R = 200,000 psi and D_2 = 7.0 inches.

Asphalt cost = 0.095 x \$86.90/ton = \$8.26/ton

Aggregate cost = 0.905 x \$10.50/ton = \$9.50/ton

Total cost of black base layer = \$17.76/ton

The cost of 7.0 inches of black base at \$17.76 per ton is \$6.76 per square yard. Therefore, the total cost of the pavement section is: (\$6.76 + \$3.92) per square yard = \$10.68 square yard.

Design 11. Reduce the asphalt content to 9.0 percent; therefore, M_R = 150,000 psi and D_2 = 7.7 inches.

Asphalt cost = 0.09 x \$86.90/ton = \$7.82/ton

Aggregate cost = 0.91 x \$10.50/ton = \$9.56/ton

Total cost of black base layer = \$17.38/ton

The cost of 7.7 inches of black base at \$17.38 per ton is \$7.15 per square yard. Therefore, the total cost of the pavement section is: (\$7.15 + \$3.92) per square yard = \$11.07 per square yard.

Design 12. Reduce the asphalt content to 8.0 percent; therefore, M_R = 100,000 psi and D_2 = 9.0 inches.

Asphalt cost = 0.08 x \$86.90/ton = \$6.95/ton

Aggregate cost = 0.92 x \$10.50/ton = \$9.66/ton

Total cost of black base layer = \$16.61/ton

Table 11. Cost analysis of "74" Ranch pit crushed sandstone mixtures.

Design Number	Percent Asphalt, by weight of mix	M_R After Vacuum Saturation, psi (73°F)	Required Thickness for Black Base Layer D_2, inches	Total Cost per Square Yard for Black Base Pavement
10	9.5 (optimum)	200,000	7.0	$10.68
11	9.0	150,000	7.7	$11.07
12	8.0	100,000	9.0	$12.02

The cost 9.0 inches of black base at \$16.61 per ton is \$8.10 per square yard. Therefore, the total cost of the pavement section is: (\$8.10 + \$3.92) per square yard = \$12.02 per square yard.

Table 11 illustrates the results obtained in Designs 10 through 12. As noted of the sandstone mixtures, the most economical black base pavement design for the crushed sandstone mixtures appears to be at the optimum asphalt content.

The cost effect of reducing the asphalt content in the blabk base layer appears to be minimal when compared to the increase in required black base thickness. On a per-unit basis, for example, the effect of a one percent change in asphalt content on the cost of a ton of black base may be determined as follows:

$$(\text{1 ton of black base}) \times \left(\frac{2000\ \text{lb}}{\text{ton}}\right) \times \left(\frac{0.01\ \text{asphalt}}{\text{content}}\right) =$$

20 lb of asphalt cement in a ton of black base (1.0% asphalt content).

The cost of asphalt cement on a nation-wide basis is assumed to be \$75.00 per ton. Consequently, the black base cost difference associated with a one percent change in asphalt content is:

$$\left(\frac{20\ \text{lb}}{\frac{2000\ \text{lb}}{\text{ton}}}\right) \times \left(\frac{\$75.00}{\text{ton}}\right) = \$0.75$$

Therefore, a decrease in one percent asphalt content will result in a decrease of \$0.75 per ton of black base (at asphalt cement price of \$75.00 per ton).

Summary of Economics

A method of economic analysis has been developed which involves the use of a thickness design procedure together with the cost of asphalt and aggregate refined to produce the desired asphalt stabilized base. This method allows the engineer to select the mixture required to carry the imposed traffic and resist the environment in which it is placed. The effect of asphalt content and aggregate type and asphalt type can be studied from an economic view point.

CONCLUSIONS

1. Five marginal aggregates from Districts 15 and 18 were investigated through an extensive laboratory testing program. The five aggregates were generally considered to be low-quality, marginal aggregates which may be found in many areas of Texas. Results indicate that several of the marginal aggregate mixtures may possess adequate load carrying characteristics provided that strict quality control measures be employed. At least three of the marginal aggregate mixtures tested have resilient modulus values consistent with those of conventional black base mixtures currently in use.

2. Three sands and a sand-gravel aggregate were also subjected to a series of laboratory tests. Results from these tests are similar to those obtained from cores of pavements presently in service in Texas. The use of sand asphalt bases in Texas appears to be a reasonable approach for providing low cost paving materials in areas where these are economically available.

3. An economic analysis method has been presented which allows the engineer to consider a number of factors including the mixture properties, the effect of the asphalt content and the cost of the asphalt and aggregate.

4. Materials have been identified which appear to be suitable for use of these materials under traffic and environmental conditions compatible with the characteristics of these materials.

5. Mixture design and pavement design must be considered simultaneously if the engineer is to economically use marginal materials as base courses.

6. The use of "sandwich" design for the use of marginal black base has been suggested. This design places the marginal material in a pavement

where tensile and shear stresses are reduced as compared to the bottom and the top of the asphalt treated layer. Additionally, the resistance to the action of water is not as critical a factor for the marginal material when used in this application. This assumes that the marginal material is sufficiently protected from excess moisture or that it is resistant to water to an extent that its performance is acceptable.

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Appendix A

Laboratory Test Results

Districts 15 and 18 Materials

Table A-1. Results of laboratory testing program.

Sample Number	Biscuit Label	Specimen Thickness inches	Asphalt, % by wgt. of mix	Compacted Specific Gravity	Rice Specific Gravity	Air Voids, percent	Hveem Stability bef vacuum saturation	Hveem Stability aft vacuum saturation
SS-1			3.9	1.99	2.450	18.9		
	SS-1A	1.874	3.9	2.04	2.46	16.7		
	SS-1B	2.069	3.9	1.93	2.44	21.2		
	SS-1C	1.935	3.9	1.99		18.8		
	SS-1D	2.004	3.9	1.99		18.8		
SS-2			6.2	2.10	2.360	11.0	61	60
	SS-2A	2.064	6.2	2.12	2.39	10.2	53	50
	SS-2B	1.830	6.2	2.11	2.33	10.6	70	70
	SS-2C	1.848	6.2	2.06		12.7	62	
	SS-2D	2.073	6.2	2.11		10.6	60	
SS-3			8.3	2.19	2.277	4.1	42	
	SS-3A	2.007	8.3	2.18	2.33	4.4	31	
	SS-3B	1.783	8.3	2.22	2.23	2.6	52	
	SS-3C	1.866	8.3	2.18	2.27	4.4	57	
	SS-3D	1.929	8.3	2.17		4.8	28	

Table A-1. (continued)

Sample Number	Biscuit Label	Specimen Thickness inches	Asphalt, % by wgt. of mix	Compacted Specific Gravity	Rice Specific Gravity	Air Voids, percent	Hveem Stability bef vacuum saturation	Hveem Stability aft vacuum saturation
SS-4			8.8	2.20	2.235	1.9	22	26
	SS-4A	2.053	8.8	2.19	2.22	2.2	11	
	SS-4B	1.737	8.8	2.19	2.25	2.2	26	40
	SS-4C	1.887	8.8	2.21		1.3	39	
	SS-4D	1.999	8.8	2.20		1.6	12	11
SS-5			5.7	2.15	2.360	8.8	56	60
	SS-5A	1.916	5.7	2.15	2.35	8.8	43	61
	SS-5B	1.742	5.7	2.12	2.37	10.2	54	
	SS-5C	1.951	5.7	2.17		8.1	62	60
	SS-5D	1.757	5.7	2.17		8.1	64	
SS-6			10.4	2.17	2.185	0.6	7	8
	SS-6A	2.009	10.4	2.17	2.18	0.7	7	
	SS-6B	1.858	10.4	2.17	2.19	0.7	7	8
	SS-6C	1.899	10.4	2.18		0.5	7	8
	SS-6D	1.572	10.4	2.18		0.5	6	

Table A-1. (continued)

Sample Number	Biscuit Label	Specimen Thickness inches	Asphalt, % by wgt. of mix	Compacted Specific Gravity	Rice Specific Gravity	Air Voids, percent	Hveem Stability bef vacuum saturation	Hveem Stability aft vacuum saturation
SS-7			9.8	2.19	2.210	1.4	7	10
	SS-7A	1.817	9.8	2.17	2.23	1.8	6	10
	SS-7B	1.716	9.8	2.21	2.19		7	
	SS-7C	1.954	9.8	2.20		0.5	7	10
	SS-7D	2.031	9.8	2.17		1.8	8	
SS-8			9.0	2.21	2.255	2.1	7	10
	SS-8A	2.037	9.0	2.25	2.25	0.5	8	
	SS-8B	1.840	9.0	2.21	2.26	2.2	9	14
	SS-8C	1.897	9.0	2.21		2.2	7	
	SS-8D	1.673	9.0	2.18		3.5	4	6
SS-10			7.3	2.11	2.315	8.9	57	64
	SS-10A	1.674	7.3	2.13	2.30	8.2	35	
	SS-10B	1.718	7.3	2.12	2.33	8.6	65	
	SS-10C	1.779	7.3	2.09		9.9	65	65
	SS-10D	1.827	7.3	2.11		8.9	64	63

Table A-1. (continued)

Sample Number	Biscuit Label	Specimen Thickness inches	Asphalt, % by wgt. of mix	Compacted Specific Gravity	Rice Specific Gravity	Air Voids, percent	Hveem Stability bef vacuum saturation	Hveem Stability aft vacuum saturation
LS-1			7.4	2.31	2.325	0.8	5	8
	LS-1A	1.837	7.4	2.30	2.32	1.1	7	
	LS-1B	1.801	7.4	2.30	2.33	1.1	7	10
	LS-1C	1.559	7.4	2.31		0.6	3	5
	LS-1D	1.532	7.4	2.32	2.32	0.2	3	
LS-2			5.7	2.38	2.395	0.8	15	21
	LS-2A	1.960	5.7	2.36	2.39	1.5	10	
	LS-2B	1.972	5.7	2.39	2.40	0.2	14	19
	LS-2C	1.853	5.7	2.37		1.0	18	
	LS-2D	1.867	5.7	2.38		0.6	16	23
LS-3			4.8	2.37	2.450	3.2	63	64
	LS-3A	2.070	4.8	2.37	2.44	3.3	49	
	LS-3B	1.885	4.8	2.36	2.46	3.7	70	
	LS-3C	1.642	4.8	2.36		3.7	68	70
	LS-3D	1.711	4.8	2.40		2.0	66	57

Table A-1. (continued)

Sample Number	Biscuit Label	Specimen Thickness inches	Asphalt, % by wgt. of mix	Compacted Specific Gravity	Rice Specific Gravity	Air Voids, percent	Hveem Stability bef vacuum saturation	Hveem Stability aft vacuum saturation
LS-4			3.8	2.30	2.475	7.2	60	69
	LS-4A	2.005	3.8	2.34	2.47	5.5	32	
	LS-4B	1.900	3.8	2.26	2.48	8.7	70	67
	LS-4C	1.612	3.8	2.28		7.9	70	
	LS-4D	1.678	3.8	2.31		6.7	68	70
LS-5			4.3	2.33	2.455	5.1	63	70
	LS-5A	1.778	4.3	2.35	2.45	4.3	52	
	LS-5B	1.726	4.3	2.31	2.46	5.9	62	
	LS-5C	1.882	4.3	2.30		6.3		
	LS-5D	1.607	4.3	2.36		3.9	75	70
LS-6			5.2	2.39	2.420	1.33	27	22
	LS-6A	1.842	5.2	2.37	2.42	2.1	25	20
	LS-6B	1.938	5.2	2.39	2.42	1.2	39	
	LS-6C	1.923	5.2	2.40		0.8	18	23
	LS-6D	1.498	5.2	2.39		1.2	26	

Table A-1. (continued)

Sample Number	Biscuit Label	Specimen Thickness inches	Asphalt, % by wgt. of mix	Compacted Specific Gravity	Rice Specific Gravity	Air Voids, percent	Hveem Stability bef vacuum saturation	Hveem Stability aft vacuum saturation
CCG-1			4.8	2.03	2.400	15.6	44	
	CCG-1A	1.983	4.8	2.03	2.39	15.4	36	
	CCG-1B		4.8		2.41			
	CCG-1C		4.8					
	CCG-1D	1.962	4.8	2.02		15.8	52	
CCG-2			5.7	1.98	2.390	17.2	51	
	CCG-2A	2.031	5.7	1.98	2.39	17.2	45	
	CCG-2B	1.821	5.7		2.39			
	CCG-2C	2.004	5.7	1.91		20.1		
	CCG-2D	1.917	5.7	2.05		14.2	56	
CCG-3			6.1	2.21	2.380	7.1	54	59
	CCG-3A	1.913	6.1	2.24	2.38	5.9	49	
	CCG-3B	1.847	6.1	2.17	2.38	8.8	54	
	CCG-3C	1.821	6.1	2.15		9.7	51	
	CCG-3D	1.735	6.1	2.28		4.2	63	59

Table A-1. (continued)

Sample Number	Biscuit Label	Specimen Thickness inches	Asphalt, % by wgt. of mix	Compacted Specific Gravity	Rice Specific Gravity	Air Voids, percent	Hveem Stability bef vacuum saturation	Hveem Stability aft vacuum saturation
CCG-4			5.2	2.16	2.415	10.4	60	
	CCG-4A	1.929	5.2	2.18	2.40	9.5	60	
	CCG-4B		5.2		2.43			
	CCG-4C		5.2	2.14		11.2		
	CCG-4D		5.2					
CCG-5			7.0	2.29	2.335	2.1	39	38
	CCG-5A	2.049	7.0	2.30	2.34	1.5	25	25
	CCG-5B	1.893	7.0	2.20	2.33	2.4	43	51
	CCG-5C	1.809	7.0	2.26		3.4	62	
	CCG-5D	1.668	7.0	2.32		0.9	26	

Table A-1. (continued)

Sample Number	Biscuit Label	Specimen Thickness inches	Asphalt, % by wgt. of mix	Compacted Specific Gravity	Rice Specific Gravity	Air voids, percent	Hveem Stability bef vacuum saturation	Hveem Stability aft vacuum saturation
CSS-1			4.8	1.91	2.26	15.5	45	
	CSS-1A	2.019	4.8	1.91	2.26	15.5	49	
	CSS-1B		4.8		2.26			
	CSS-1C		4.8					
	CSS-1D	1.904	4.8	1.91		15.5	41	
CSS-2			5.7	1.90	2.235	15.3	58	
	CSS-2A	1.818	5.7	1.93	2.24	13.8	52	
	CSS-2B	1.952	5.7	1.84	2.23	17.9		
	CSS-2C	1.773	5.7	1.86		17.0		
	CSS-2D	1.903	5.7	1.96		12.5	63	
CSS-3			7.0	1.92	2.190	12.5	63	60
	CSS-3A	1.963	7.0	1.95	2.19	11.0	64	68
	CSS-3B	1.955	7.0	1.88	2.19	14.2	55	
	CSS-3C	1.829	7.0	1.88		14.2	61	53
	CSS-3D	1.649	7.0	1.96		10.5	70	

Table A-1. (continued)

Sample Number	Biscuit Label	Specimen Thickness inches	Asphalt, % by wgt. of mix	Compacted Specific Gravity	Rice Specific Gravity	Air Voids, percent	Hveem Stability bef vacuum saturation	Hveem Stability aft vacuum saturation
CSS-4			6.5	1.93	2.190	12.1	64	64
	CSS-4A	2.027	6.5	1.94	2.19	11.4	53	57
	CSS-4B	1.864	6.5	1.94	2.19	11.4	70	71
	CSS-4C	1.639	6.5	1.88		14.2	65	
	CSS-4D	1.690	6.5	1.94		11.4	69	
CSS-5			7.8	1.94	2.160	10.2	58	56
	CSS-5A	2.068	7.8	1.99	2.17	7.9	48	
	CSS-5B	1.980	7.8	1.88	2.15	13.0	52	50
	CSS-5C	1.873	7.8	1.91		11.6	63	
	CSS-5D	1.690	7.8	1.98		8.3	67	63
CSS-6			9.1	1.96	2.095	6.6	47	36
	CSS-6A	1.769	9.1	1.96	2.09	6.4	39	34
	CSS-6B	1.898	9.1	1.91	2.10	8.8	68	56
	CSS-6C	2.046	9.1	2.00		4.5	35	19

Table A-1. (continued)

Sample Number	Biscuit Label	Specimen Thickness inches	Asphalt, % by wgt. of mix	Compacted Specific Gravity	Rice Specific Gravity	Air Voids, percent	Hveem Stability bef vacuum saturation	Hveem Stability aft vacuum saturation
DAC-1			6.8	1.96	2.285	14.5	44	
	DAC-1A	2.071	6.8	2.00	2.28	12.5	51	
	DAC-1B	1.857	6.8	1.91	2.29	16.4		
	DAC-1C	1.749	6.8	1.91		16.4		
	DAC-1D	1.729	6.8	2.00		12.5	36	
DAC-2			5.7	1.97	2.300	14.5	38	
	DAC-2A	2.126	5.7	1.99	2.29	13.5	38	
	DAC-2B	1.997	5.7	1.93	2.31	16.1		
	DAC-2C		5.7					
	DAC-2D	1.885	5.7	1.98		13.9		
DAC-3			5.2	1.93	2.310	16.5		
	DAC-3A	2.039	5.2	1.93	2.21	16.5		
	DAC-3B		5.2		2.33			
	DAC-3C		5.2					
	DAC-3D		5.2					

Table A-1. (continued)

Sample Number	Biscuit Label	Specimen Thickness inches	Asphalt, % by wgt. of mix	Compacted Specific Gravity	Rice Specific Gravity	Air Voids, percent	Hveem Stability bef vacuum saturation	Hveem Stability aft vacuum saturation
DAC-4			6.1	1.95	2.280	14.5		
	DAC-4A	2.169	6.1	1.95	2.26	14.5		
	DAC-4B		6.1		2.30			
	DAC-4C		6.1					
	DAC-4D		6.1	1.95		14.5		
DAC-5			7.8	2.04	2.190	7.1	47	53
	DAC-5A	2.054	7.8	2.08	2.19	5.0	49	
	DAC-5B	1.768	7.8	2.01	2.19	8.2	46	
	DAC-5C	1.820	7.8	1.99		9.1	48	
	DAC-5D	1.962	7.8	2.06		5.9	45	53

Table A-2. Resilient moduli results of laboratory testing program.

Sample Number	Biscuit Label	Sample Thickness inches	M_R x10^6 psi 6"ϕx2" ht. 73° F	M_R x10^6 psi 4"ϕx2" ht. 73° F	M_R x10^6 psi aft vacuum saturation 73° F	M_R x10^6 psi bef vacuum saturation 34° F	M_R x10^6 psi bef vacuum saturation 100° F
SS-1			0.166				
	SS-1A	1.874	0.212				
	SS-1B	2.069	0.144				
	SS-1C	1.935	0.149				
	SS-1D	2.004	0.158				
SS-2			0.532	0.256	0.094	1.195	0.068
	SS-2A	2.064	0.613	0.368	0.114	1.622	0.090
	SS-2B	1.830	0.389	0.144	0.074	0.768	0.046
	SS-2C	1.848	0.349				
	SS-2D	2.073	0.778				
SS-3			0.624	0.496	0.157	1.740	0.082
	SS-3A	2.007	0.690				
	SS-3B	1.783	0.646	0.466	0.156	1.814	0.070
	SS-3C	1.866	0.652	0.526	0.158	1.665	0.093
	SS-3D	1.929	0.506				

Table A-2. (continued)

Sample Number	Biscuit Label	Sample Thickness inches	M_R x10^6 psi 6"øx2" ht. 73° F	M_R x10^6 psi 4"øx2" ht. 73° F	M_R x10^6 psi aft vacuum saturation 73° F	M_R x10^6 psi bef vacuum saturation 34° F	M_R x10^6 psi bef vacuum saturation 100° F
SS-4			0.548	0.407	0.446	1.717	0.060
	SS-4A	2.053	0.310				
	SS-4B	1.737	0.531	0.384	0.339	1.851	0.059
	SS-4C	1.887	0.718				
	SS-4D	1.999	0.634	0.430	0.553	1.583	0.061
SS-5			0.534	0.192	0.086	0.928	0.060
	SS-5A	1.916	0.594	0.246	0.120	1.060	0.075
	SS-5B	1.742	0.363				
	SS-5C	1.951	0.475	0.138	0.052	0.795	0.045
	SS-5D	1.757	0.705				
SS-6			0.198	0.167	0.203	1.545	0.026
	SS-6A	2.009	0.149				
	SS-6B	1.858	0.213	0.184	0.237	1.572	0.030
	SS-6C	1.899	0.214	0.150	0.168	1.517	0.022
	SS-6D	1.572	0.217				

Table A-2. (continued)

Sample Number	Biscuit Label	Sample Thickness inches	M_R x10^6 psi 6"ø x2" ht. 73° F	M_R x10^6 psi 4"ø x2" ht. 73° F	M_R x10^6 psi aft vacuum saturation 73° F	M_R x10^6 psi bef vacuum saturation 34° F	M_R x10^6 psi bef vacuum saturation 100° F
SS-7			0.259	0.219	0.309	2.222	0.038
	SS-7A	1.817	0.267	0.255	0.337	1.997	0.045
	SS-7B	1.716	0.295				
	SS-7C	1.954	0.256	0.182	0.281	2.447	0.031
	SS-7D	2.031	0.219				
SS-8			0.347	0.201	0.317	1.791	0.039
	SS-8A	2.037	0.288				
	SS-8B	1.840	0.355	0.211	0.328	1.691	0.044
	SS-8C	1.897	0.451				
	SS-8D	1.673	0.294	0.191	0.306	1.891	0.034
SS-10			0.636	0.315	0.080	0.886	0.068
	SS-10A	1.674	0.803				
	SS-10B	1.718	0.454				
	SS-10C	1.779	0.497	0.206	0.061	0.951	0.047
	SS-10D	1.827	0.789	0.424	0.098	0.820	0.089

Table A-2. (continued)

Sample Number	Biscuit Label	Sample Thickness inches	M_R x10^6 psi 6"ϕx2" ht. 73° F	M_R x10^6 psi 4"ϕx2" ht. 73° F	M_R x10^6 psi aft vacuum saturation 73° F	M_R x10^6 psi bef vacuum saturation 34° F	M_R x10^6 psi bef vacuum saturation 100° F
LS-1			0.286	0.261	0.440	2.829	0.045
	LS-1A	1.837	0.303				
	LS-1B	1.801	0.302	0.275	0.435	3.562	0.046
	LS-1C	1.559	0.286	0.247	0.445	2.095	0.043
	LS-1D	1.532	0.251				
LS-2			0.482	0.558	0.755	4.036	0.090
	LS-2A	1.960	0.275				
	LS-2B	1.972	0.428	0.539	0.759	4.14	0.085
	LS-2C	1.853	0.647				
	LS-2D	1.867	0.577	0.576	0.750	3.932	0.094
LS-3			1.247	0.963	1.003	7.580	0.262
	LS-3A	2.070	1.039				
	LS-3B	1.885	1.500				
	LS-3C	1.642	1.343	0.830	0.896	11.751	0.226
	LS-3D	1.711	1.106	1.096	1.110	3.408	0.298

Table A-2. (continued)

Sample Number	Biscuit Label	Sample Thickness inches	M_R x10^6 psi 6"ϕx2" ht. 73° F	M_R x10^6 psi 4"ϕx2" ht. 73° F	M_R x10^6 psi aft vacuum saturation 73° F	M_R x10^6 psi bef vacuum saturation 34° F	M_R x10^6 psi bef vacuum saturation 100° F
LS-4			0.746	0.789	0.311	2.531	0.227
	LS-4A	2.005	0.991				
	LS-4B	1.900	0.566	0.698	0.222	2.251	0.193
	LS-4C	1.612	0.498				
	LS-4D	1.678	0.927	0.880	0.399	2.811	0.260
LS-5			1.184	1.309	1.007	3.569	0.357
	LS-5A	1.778	0.886				
	LS-5B	1.726					
	LS-5C	1.882	1.296	1.197	0.942	2.78	0.317
	LS-5D	1.602	1.371	1.421	1.071	4.358	0.397
LS-6			0.618	0.550	0.669	3.231	0.096
	LS-6A	1.842	0.568	0.586	0.664	3.312	0.101
	LS-6B	1.938	0.736				
	LS-6C	1.923	0.663	0.514	0.674	3.149	0.090
	LS-6D	1.498	0.506				

Table A-2. (continued)

Sample Number	Biscuit Label	Sample Thickness inches	M_R x10^6 psi 6"ϕx2" ht. 73° F	M_R x10^6 psi 4"ϕx2" ht. 73° F	M_R x10^6 psi aft vacuum saturation 73° F	M_R x10^6 psi bef vacuum saturation 34° F	M_R x10^6 psi bef vacuum saturation 100° F
CCG-1			0.296	0.200		0.485	0.053
	CCG-1A	1.983	0.383	0.333		0.770	0.082
	CCG-1B						
	CCG-1C						
	CCG-1D	1.962	0.208	0.066		0.200	0.024
CCG-2			0.189	0.190	0.012	0.741	0.055
	CCG-2A	2.031	0.159	0.279	0.012	1.184	0.075
	CCG-2B	1.821					
	CCG-2C	2.004	0.165				
	CCG-2D	1.917	0.244	0.101		0.297	0.035
CCG-3			0.521	0.546	0.019	1.329	0.114
	CCG-3A	1.913	0.825				
	CCG-3B	1.847	0.365	0.098	0.022	0.746	0.035
	CCG-3C	1.821	0.325				
	CCG-3D	1.735	0.569	0.994	0.015	1.911	0.192

Table A-2. (continued)

Sample Number	Biscuit Label	Sample Thickness inches	M_R x10^6 psi 6"ϕx2" ht. 73° F	M_R x10^6 psi 4"ϕx2" ht. 73° F	M_R x10^6 psi aft vacuum saturation 73° F	M_R x10^6 psi bef vacuum saturation 34° F	M_R x10^6 psi bef vacuum saturation 100° F
CCG-4			0.274	0.268		0.797	0.100
	CCG-4A	1.929	0.274	0.268		0.797	0.100
	CCG-4B						
	CCG-4C						
	CCG-4D						
CCG-5			0.461	0.603	0.264	2.475	0.100
	CCG-5A	2.049	0.407	0.653	0.498	2.764	0.102
	CCG-5B	1.893	0.690	0.553	0.029	2.186	0.097
	CCG-5C	1.809	0.374				
	CCG-5D	1.668	0.371				

Table A-2. (continued)

Sample Number	Biscuit Label	Sample Thickness inches	M_R x10^6 psi 6"øx2" ht. 73° F	M_R x10^6 psi 4"øx2" ht. 73° F	M_R x10^6 psi aft vacuum saturation 73° F	M_R x10^6 psi bef vacuum saturation 34° F	M_R x10^6 psi bef vacuum saturation 100° F
CSS-1			0.189	0.061		0.117	0.023
	CSS-1A	2.019	0.164	0.081		0.156	0.023
	CSS-1B						
	CSS-1C						
	CSS-1D	1.904	0.214	0.040		0.078	0.023
CSS-2			0.180	0.040		0.117	0.025
	CSS-2A	1.818	0.212	0.045		0.135	0.025
	CSS-2B	1.952	0.079				
	CSS-2C	1.773	0.123	0.035		0.098	
	CSS-2D	1.903	0.305				
CSS-3			0.383	0.125	0.035	0.920	0.050
	CSS-3A	1.963	0.465	0.167	0.040	1.040	0.065
	CSS-3B	1.955	0.272				
	CSS-3C	1.829	0.286	0.083	0.029	0.800	0.034
	CSS-3D	1.649	0.507				

Table A-2. (continued)

Sample Number	Biscuit Label	Sample Thickness inches	M_R x10^6 psi 6"ϕx2" ht. 73° F	M_R x10^6 psi 4"ϕx2" ht. 73° F	M_R x10^6 psi aft vacuum saturation 73° F	M_R x10^6 psi bef vacuum saturation 34° F	M_R x10^6 psi bef vacuum saturation 100° F
CSS-4			0.338	0.191	0.042	1.127	0.059
	CSS-4A	2.027	0.412	0.252	0.056	1.446	0.072
	CSS-4B	1.864	0.258	0.130	0.028	0.808	0.045
	CSS-4C	1.639	0.246				
	CSS-4D	1.690	0.435				
CSS-5			0.415	0.205	0.053	1.002	0.060
	CSS-5A	2.068	0.565				
	CSS-5B	1.980	0.269	0.106	0.028	0.891	0.033
	CSS-5C	1.873	0.269				
	CSS-5D	1.690	0.555	0.303	0.078	1.112	0.086
CSS-6				0.327	0.174	1.643	0.079
	CSS-6A	1.769		0.331	0.251	1.785	0.081
	CSS-6B	1.898		0.355	0.052	1.518	0.090
	CSS-6C	2.046		0.296	0.220	1.625	0.065

Table A-2. (continued)

Sample Number	Biscuit Label	Sample Thickness inches	M_R x10^6 psi 6"øx2" ht. 73° F	M_R x10^6 psi 4"øx2" ht. 73° F	M_R x10^6 psi aft vacuum saturation 73° F	M_R x10^6 psi bef vacuum saturation 34° F	M_R x10^6 psi bef vacuum saturation 100° F
DAC-1			0.449	0.197		0.611	0.095
	DAC-1A	2.071	0.732	0.296		0.837	0.135
	DAC-1B	1.857	0.174				
	DAC-1C	1.749	0.224				
	DAC-1D	1.729	0.666	0.097		0.385	0.055
DAC-2			0.192	0.125		0.328	0.057
	DAC-2A	2.126	0.286	0.125		0.328	0.057
	DAC-2B	1.997	0.098				
	DAC-2C						
	DAC-2D	1.885	0.187				
DAC-3			0.217	0.027		0.059	0.029
	DAC-3A	2.039	0.217	0.027		0.059	0.029
	DAC-3B						
	DAC-3C						
	DAC-3D						

Table A-2. (continued)

Sample Number	Biscuit Label	Sample Thickness inches	M_R x10^6 psi 6"ϕx2" ht. 73° F	M_R x10^6 psi 4"ϕx2" ht. 73° F	M_R x10^6 psi aft vacuum saturation 73° F	M_R x10^6 psi bef vacuum saturation 34° F	M_R x10^6 psi bef vacuum saturation 100° F
DAC-4			0.283	0.051		0.066	0.033
	DAC-4A	2.169	0.283	0.051		0.066	0.033
	DAC-4B						
	DAC-4C						
	DAC-4D						
DAC-5			0.604	0.525	0.179	1.892	0.206
	DAC-5A	2.054	0.749	0.658	0.274	2.409	0.257
	DAC-5B	1.768	0.419				
	DAC-5C	1.820	0.399				
	DAC-5D	1.962	0.848	0.392	0.083	1.375	0.154

Appendix B

Laboratory Test Results

Sand and Sand-Gravel Materials

Table B-1, Stability Data for Sand Aggregate Mixtures (Laboratory Molded).

Material	% Asphalt AC-10	Sample	Hveem Stab. @140°F Before Sat. (Dry)	Hveem Stab. @140°F After Sat. (Dry)	Marshall After Sat. Stability	Marshall (Dry) Flow
Dist 25	4	A	17*	24	556	15
FM3182		B	15*	17*	417	24
Wheeler		C	20*	22	660	17
Co.	5	A	16*	23	695	14
		B	19*	25	661	15
		C	15*	20*	334	22
	6	A	16*	20*	250	34
		B	16*	20*	294	18
		C	17*	22*	323	26
Dist 21	4	A	49	62	3405	27
Beck Pit		B	36	51	3336	27
Hidalgo		C	56	59	3276	18
Co.	5	A	47	52	2714	18
		B	45	50	2998	22
		C	43	40	2499	27
	6	A	38	18*	1867	30
		B	36	10*	2463	19
		C	36	26	2940	20
Dist 20	4	A	22	26	294	27
US 96		B	26	34	412	26
Jasper		C	22	28	338	27
Co.	5	A	29	39	1338	22
		B	31	35	1250	20
		C	32	36	1323	18
	6	A	47	34	1529	22
		B	51	34	1632	21
		C	51	31	1588	23
Dist 5	4	A	12*	14*	104	47
FM 168		B	10*	13*	104	48
Lamb		C	12*	15*	528	17
Co.	5	A	11*	14*	150	14
		B	12*	16*	568	16
		C	14*	22*	726	22
	6	A	9*	15*	111	15
		B	11*	18*	119	18
		C	12*	12*	139	13

Calculated value, sample to weak to reach 6000

Table B-2. Resilient Modulus Data for Sand Aggregate Mixtures (Laboratory Molded).

Material	% AC-10 Asphalt	Sample	Before Saturation -10°F $x10^6$,psi	32°F $x10^6$,psi	73°F $x10^6$,psi	100°F $x10^6$,psi	Saturated @ 68°F $x10^6$,psi	After Dry Back @ 68°F $x10^6$,psi
Dist. 25	4	A	2.71	0.802	.130		.071	.158
FM3182		B	2.33	0.906	.134		.077	.155
Wheeler		C	2.23	0.861	.114		.065	.162
Co.	5	A	2.20	0.522	.136	.018	.074	.107
		B	2.55	0.734	.111	.028	.067	.161
		C	2.68	0.539	.107	.013	.043	.087
	6	A	2.72	0.376	.0788		.062	.078
		B	3.19	0.444	.110		.068	.079
		C	2.45	0.349	.106		.063	.075
Dist. 21	4	A	1.68	0.379	.178	.050	.0495	.136
Beck Pit		B	2.54	0.595	.306	.078	.071	.147
Hidalgo		C	2.29	0.472	.296	.074	.054	.115
Co.	5	A	3.76	1.03	.516	.108	.158	.278
		B	3.84	1.08	.624	.120	.196	.286
		C	3.08	0.693	.391	.076	.153	.196
	6	A	4.45	2.06	.617	.122	.341	.316
		B	4.39	2.76	.748	.136	.655	.484
		C	4.75	1.59	.736	.109	.334	.405
Dist. 20	4	A	0.380	0.074	.086	.024	.033	.046
US96		B	0.530	0.166	.139	.039	.032	.074
Jasper		C	0.508	0.115	.125	.037	.0095	.056
Co.	5	A	2.20	0.905	.281	.138	.094	.333
		B	1.95	0.861	.281	.130	.137	.306
		C	2.10	0.893	.248	.133	.133	.325
	6	A	3.25	1.06	.343	.127	.370	.251
		B	3.42	1.20	.364	.125	.264	.289
		C	3.15	1.12	.351	.127	.281	.281
Dist. 5	4	A	2.00	0.549	.071		.060	.110
FM168		B	2.30	0.339	.092		.053	.055
Lamb Co.		C	1.83	0.594	.135		.102	.130
	5	A	2.69	0.595	.074	.007	.056	.091
		B	1.98	0.547	.119	.020	.083	.145
		C	1.84	0.666	.127	.023	.094	.222
	6	A	2.07	0.444	.088		.063	.061
		B	2.42	0.472	.074			.064
		C	2.26	0.468	.082		.062	.069

Table B-3. Properties of Sand Aggregate Mixtures (Laboratory Molded)

Sample I. D.	% Asph.	Sample	Before Satutation WT.,9m.	Saturated WT.,9m.	Finel WT. Dry,9m.	Ave Rice SP GR	Bulk SP GR	Rel Den,%	Air Voids,%	Ave Air Voids,%
Dist. 25	4	A	835	932	833		1.932	74.1	25.9	
FM3182		B	834	937	831	2.607	1.918	73.6	26.4	26.2
Wheeler		C	840	941	837		1.918	73.6	26.4	
Co.	5	A	834	924	838		1.944	79.6	20.4	
		B	823	911	824	2.440	1.952	80.0	20.0	20.1
		C	839	931	841		1.956	80.2	19.8	
	6	A	824	905	833		1.974	81.7	18.3	
		B	823	903	832	2.417	1.980	81.9	18.1	18.2
		C	826	909	836		1.975	81.7	18.3	
Dist. 21	4	A	912	979	907	2.410	2.193	91.0	9.0	
Beck Pit		B	912	992	911		2.120	88.0	12.0	10.4
Hidalgo		C	863	930	864		2.163	89.8	10.2	
Co.	5	A	885	941	893		2.210	92.9	7.1	
		B	906	961	916	2.380	2.219	93.2	6.8	7.1
		C	900	955	912		2.201	92.5	7.5	
	6	A	890	928	908		2.245	97.1	2.9	
		B	903	936	919	2.312	2.248	97.2	2.8	2.8
		C	910	950	922		2.249	97.3	2.7	
Dist. 20	4	A	815	888	793		1.990	79.9	20.1	
US96		B	812	886	798	2.491	2.005	80.5	19.5	19.7
Jasper		C	816	886	798		2.005	80.5	19.5	
Co.	5	A	832	911	829		2.027	82.1	17.9	
		B	832	910	829	2,468	2.012	81.5	18.5	18.2
		C	832	908	829		2.019	81.8	18.2	
	6	A	852	916	856		2.066	84.6	15.4	
		B	854	918	857	2.441	2.068	84.7	15.3	15.1
		C	850	912	854		2.083	95.3	14.7	
Dist. 5	4	A	801	960	804		1.864	74.0	26.0	
FB168		B	800	904	809	2.520	1.862	73.9	26.1	26.1
Lamb		C	829	935	832		1.858	73.7	26.3	
Co.	5	A	829	942	832		1.849	75.7	24.3	
		B	820	931	830	2.443	1.851	75.8	24.2	24.3
		C	826	935	826		1.845	75.5	24.5	
	6	A	822	919	837		1.895	80.6	19.4	
		B	820	922	837	2.352	1.876	79.8	20.2	19.6
		C	818	916	834		1.898	80.7	19.3	