ANALYSIS OF BITUMINOUS MIXES INCORPORATING SILICEOUS AGGREGATES

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CHAPTER I

INTRODUCTION

The design of bituminous pavements embodying both adequate stability to resist deformation due to traffic and sufficient resistance to skidding of vehicles under adverse conditions has long been studied by various agencies involved in highway engineering. One problem of combining high stability with good skid resistance has been the fact that those aggregates capable of resisting the polishing action of traffic may have inherent properties that are detrimental to the stability of the compacted asphalt-aggregate mixture. Many investigators have studied the effects on stability and skid resistance by incorporating various kinds of aggregates into pavement mixes.

The purpose of this study was to analyze bituminous mixes incorporating various siliceous aggregates available from sources in Oklahoma. The analysis was based on results from the Hveem stabilometer and cohesiometer tests on compacted asphalt-aggregate mixtures. In other words, the effects of the siliceous aggregates on the stability and cohesion of the bituminous paving mixes were studied.

A standard mix, composed of limestone as the coarse fraction and river sand as the fine aggregate fraction, was designed based on the Type B surface or base course gradation specifications from the Oklahoma Highway Department. The Type B mix was selected because it specified a coarse aggregate size up to and including the minus 3/4" sieve. Nine different siliceous aggregates were incorporated into the coarse aggregate fraction of the standard mix in three percentages, 20, 30, and 40%, based on the acid-insoluble residue content of the siliceous aggregates.

The density of the compacted mixes containing the siliceous aggregates was also evaluated. Three different methods were employed in determining the percent density of the compacted specimen, one method based on the bulk impregnated specific gravity of the combined aggregate, a calculated method using the bulk specific gravity (SSD) of the aggregate, and a vacuum saturation test method to determine the maximum specific gravity of a voidless sample of bituminous paving mixture.

CHAPTER II

LITERATURE REVIEW

Mix Design Method and Tests

The method of mix design and testing procedures used in this study conform to those employed by the Oklahoma Highway Department (1) and the Texas Highway Department (2). These methods and test procedures are a modification of those of the standard Hveem Method of Mix Design as outlined by the Asphalt Institute (3). The study utilized stability and cohesion values to determine whether the use of siliceous aggregates in standard limestone mixtures had any detrimental effect.

Stability and cohesion values have been used since the early 1930's to evaluate bituminous concrete mixtures. Stanton and Hveem (4) performed extensive research to determine the role of the laboratory in the investigation and control of the materials used in the construction of bituminous concrete pavements. From this research evolved the concept of "stability" and "cohesion" of a bituminous mix.

Stability is defined as that property of a bituminous pavement which tends to resist plastic deformation due to applied wheel loads. Results indicated that almost any aggregate gradation may develop sufficient stability to withstand the traffic loads, provided that, due to its surface characteristics, the aggregate itself has a high inherent stability. It was observed that unstable conditions predominated in pavements containing aggregates with a hard glassy surface texture. On the other hand, when aggregates of a rough irregular surface texture were used, fewer cases of instability were noticed. In fact, the surface characteristics of the mineral aggregates were determined to be the most important single quality affecting the stability of bituminous mixtures.

According to Hveem, cohesion is defined as that property of a bituminous mixture which tends to resist material flow with time. It is a measure of the cohesive resistance or tensile strength of the compacted mixture. Cohesion exists due to the adhesion of the asphalt binder to the aggregate particles and the coherent strength of the asphalt films. It also reflects the consistency of the asphalt binder and the fineness of the aggregate. Results indicated that high tensile strength, i.e., high cohesion, was not necessarily essential for resistance to the distorting effects of traffic. Discussion of the procedure followed by the author for the mix design and the stability and cohesion tests is found in Chapter IV.

Specimen Compaction

Unlike the compaction device used by Hveem in his mix design procedure, the device used in this study was a gyratory compaction apparatus styled after the device currently used by the Texas Highway Department (2) and the Oklahoma Highway Department (1). The gyratory apparatus evolved from a study conducted by the Texas Highway Department to find a mechanical method for field molding bituminous concrete mixtures into test specimens that could be tested in the Hveem stabilometer and cohesiometer.

The Texas Highway Department established the following requirements for the test specimens and the molding apparatus. The test specimen must be cylindrical in shape, four inches in diameter, and approximately two inches in height. The physical characteristics of the specimen must be as nearly identical as possible with the physical characteristics of the pavement. The specimen must have a density approximately equal to the density of the pavement produced from the mix. The density of the test specimen was selected to be 94%, that is, the total volume of solids divided by the total volume of the specimen. The aggregate degradation obtained from the actual molding must be approximately equal to the degradation obtained from the laydown and compaction operation used in the field. The molding apparatus must be a simple device preferably employing a hydraulic press already found in the field laboratory. Each test specimen must be molded separately to allow greater flexibility of the preparation and testing of specimens from many different mix designs.

Since preparation of the test specimen is an integral part of a testing procedure, the method of making the test specimen must be adaptable both to the design and control of the mix. For example, an excellent procedure for preparing and testing a specimen for design purposes requiring a week to perform could not be used on the job site to control the mix because of the long time involved.

Philippi (5) described nine mechanical devices studied by the Texas Highway Department. The ninth device, the gyratory apparatus, was finally selected as best fitting the requirements stated above. The apparatus included a hydraulic press and a cylindrical mold with two detachable long handles. A manual gyratory shearing motion was imparted to the molding cylinder using the long handles at a low compressive pressure. This allowed for the proper aggregate particle orientation which in turn controlled the amount of aggregate degradation. A high compressive pressure was then applied by the hydraulic press to obtain a required specimen density. The amount of low and high compressive pressures and the number and amplitude of rotations was then determined to approximate the actual laydown and compaction operation employed in the field. One man was capable of operating this apparatus successfully and with excellent reproducibility.

The apparatus used in this study was a motorized gyratory shear compaction device (see Figure 6), an outgrowth of the manually operated device, and the method of molding is identical to that stipulated by the Texas and Oklahoma Highway Departments. Dimensions of the apparatus and the molding pressures used in this project are outlined in Appendix A.

Skid Resistance of Pavements

The degree of slipperiness of a pavement surface is a matter of grave concern to everyone engaged in the design, construction, and maintenance of highways. One characteristic common to all types of pavement surfaces, either bituminous or portland cement concrete, lies in some coefficient of friction between the surface and the tires of a moving vehicle. Several factors affecting the mix also affect the measured skid resistance. Surface texture of the pavement is controlled by the grading and the maximum size of the aggregate particles. Although, it generally is considered that the fine graded aggregate mix, which gives a so called "sandpaper" finish to the surface, is the most skid resistant of all bituminous surfaces, research results do not bear this out in every case. Coarser mixes, in many cases, give equally good skid resistant surfaces.

Particle shape is also a factor affecting skid resistance. In many instances, the shape of the coarse particles is different than the shape of the finer particles in the same aggregate. Wholly irregular aggregate particles give the greatest degree of skid resistance. Much research has been conducted using rounded particles with one or more flattened sides or fractured faces in a bituminous mix. Under rolling operations, the particles tend to orient themselves with the flat faces parallel to the surface. When the traffic wears off the asphalt coating, the surface slipperiness is increased greatly, and, if the aggregate polishes easily, the surface becomes dangerous particularly when wet.

The toughness or abrasion resistance of the aggregate influences the slipperiness of a bituminous pavement. Test results indicate that skid resistance decreases with the Los Angeles abrasion loss. However, much practical experience indicates that the reverse is true. The occurrence of excess asphalt on the pavement surface will result in a low resistance to skidding. One cause of this problem is the ascension of asphalt from the bottom of the pavement to the surface due to stripping (6). Another cause is due either to the addition of too much asphalt to the mix or to the breakdown of the aggregate under heavy traffic loads, which reduces the original percent voids.

Other factors not related to the mix design affect the skid resistance of pavements. The presence of varying quantities of water on the surface, the material from which the tire tread is made, the smoothness of the tread, the condition of the car brakes, and many other factors contribute to the degree and character of the skid (7).

A bituminous concrete mix must meet certain design criteria such as stability in order to qualify as a usable pavement mixture. However, very few highway departments specify a mix design criterion based upon skid resistance of the pavement. Obviously, a pavement with an adequate stability but a negligible ability to resist the skidding of vehicle tires is not a desirable pavement.

A study of skid resistance of bituminous surfaces in New York was conducted by Burnett, Gibson, and Kearney (8) in 1968. They used a New York skid trailer on pavements subjected to five million equivalent vehicle passes. They found that adding up to 50% siliceous sand into a limestone coarse aggregate mix did not increase the skid resistance. Apparently, skid resistance is governed to a large extent by the coarse aggregate present in the mix. One pavement tested contained only siliceous sand (90% feldspar and silica content), as aggregate. It retained excellent skid resistance after the required vehicle passes. By adding only 5% crushed dolomitic limestone to the mix, the measured skid resistance was reduced. Results showed that pavements containing limestone had coefficients of less than 0.32 while pavements containing either traprock, sandstone, crushed gravel, iron ore tailings, or granite had coefficients greater than 0.40. Skid resistance was primarily controlled by the ability of the larger aggregates to resist the polishing action of traffic. Also, the coefficient of friction increased with the increase in percent acid-insoluble residue up to a point, as shown in Figure 1.

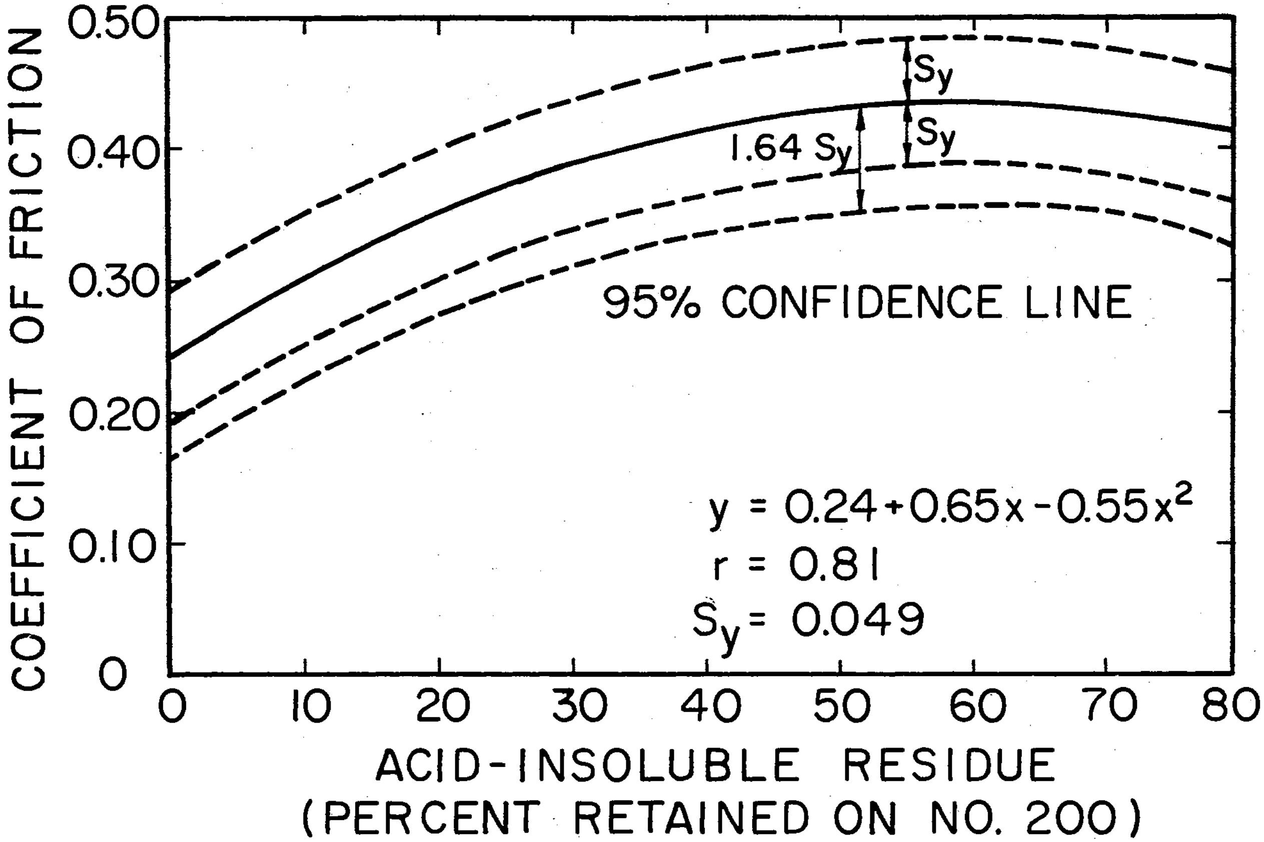

Figure 1. Correlation of Pavement Skid Resistance with Acid-Insoluble Residue of Coarse Aggregate (After Burnett, et al., 8)

An existing pavement with low skidding resistance can be rejuvenated by various methods, plant mix seals, chip seals, etc., to improve its skid resistance, but this involves extra cost. Nichols, Dillard, and Alwood (9) conducted an investigation in Virginia to design a mix with built-in high skid resistance thus eliminating the extra cost of surface rejuvenation. Results of skid tests indicated that the polishing of certain limestone aggregates in the mix was the major cause of Virginia's poor skid resistance pavements. Therefore, they incorporated abrasive aggregate into the mix without altering the gradation specifications. In the first part of the investigation, a polishing resistant fine aggregate (20% to 25% silica sand) was added to the mix but practically no change in skid resistance resulted. Next, they added a polishing resistant coarse aggregate (10%, 20%, and 30% granite or crushed gravel) and this resulted in some improvement of pavement skid resistance.

Polishing Characteristics of Aggregates

Aggregates used in the construction of bituminous pavements are normally described by various physical characteristics or properties such as shape, surface texture, resistance to abrasion, soundness, etc. Within the last 20 years, another physical property, polishing tendency, has been researched quite extensively. Polishing tendencies of various kinds of aggregates have been studied in connection with pavement skid resistance research. It is well substantiated that pavement skid resistance is a function of the kind of aggregate incorporated in the surface mix. Therefore, pavement skid resistance is directly related to the polishing property of the aggregate.

This polishing property is primarily dependent upon the mineralogical content of the aggregate. Maclean and Shergold (10) in a recent report stated the following conclusion, ". . . it is suggested that one important characteristic of rocks that remain rough is the presence of two minerals that have a considerable difference in the resistance to wear." In 1960, Knill (11) investigated the dependence of an aggregate's ability to resist polishing based on its mineral composition. In general, she concluded the following: 1) In the igneous rocks, the variation of the hardness between the minerals and the proportions of the soft minerals present caused the resistance to polishing to be high, 2) In the metamorphic rocks, an intermediate resistance to polishing was observed for the Quartzite group due to the presence of altered feldspars and shattered quartz grains. The Hornfels group exhibited low resistance due to the high proportion of hard minerals present, 3) In the sedimentary rocks, variable resistance was observed. The Gritstones (sandstones) gave a high resistance because of the hard crystals present in the soft and friable matrix. Flint, consisting of basically one hard mineral, had a low resistance. Higher resistances to polishing were obtained from certain limestones composed of calcium carbonate and an insoluble residue (especially if the residue was quartz) than from limestones not containing an insoluble residue. This idea of differential hardness could possibly be a controlling factor with regard to polishing of aggregate and pavement skid resistance.

Gray and Renninger (12) conducted skid resistance tests on bituminous concrete surfaces to determine the polishing characteristics of several dissimilar carbonate aggregates. Sections of pavement were constructed using the different aggregates and were tested by the

National Crushed Stone Association Slipperiness Testing Apparatus, a calibrated spinning bicycle wheel, at different intervals of traffic wear. Results of these tests were varied but several interesting effects did occur. The amount of acid-insoluble residue influenced but did not control the skid resistance. However, the sand-size grains of the residue effectively contributed to the skid resistance of the pavement. It was noted that two limestone pavements composed of arenaceous limestone had the highest skid resistance. Figure 2 shows the relationship of skid resistance to the amount of residue. Portland cement pavements were also constructed using the same carbonate aggregates. Results showed that the type of binder, i.e., asphalt or Portland cement, had little if any effect on the ultimate skid resistance developed by a highway pavement.

It is not economical to construct actual sections of pavement to test skid resistance of various kinds of aggregates. Therefore, a method of determining the skid resistance or polishing tendencies of an aggregate in the laboratory is most desirable. Sherwood and Mahone (13) established tentative guidelines for the use of Virginia limestones in pavement surfaces. These guidelines were based on two measurable parameters, the acid-insoluble constituents of the aggregate and the traffic volume on the pavement. Results of numerous tests indicated that the polish susceptibility of the Virginia limestones was related to the amount of non-carbonate or acid-insoluble residue. Results of years of pavement evaluations indicated that skid resistance was also related to the type and amount of traffic on the pavement. A stopping distance number was used to evaluate the pavement surfaces. They found that as

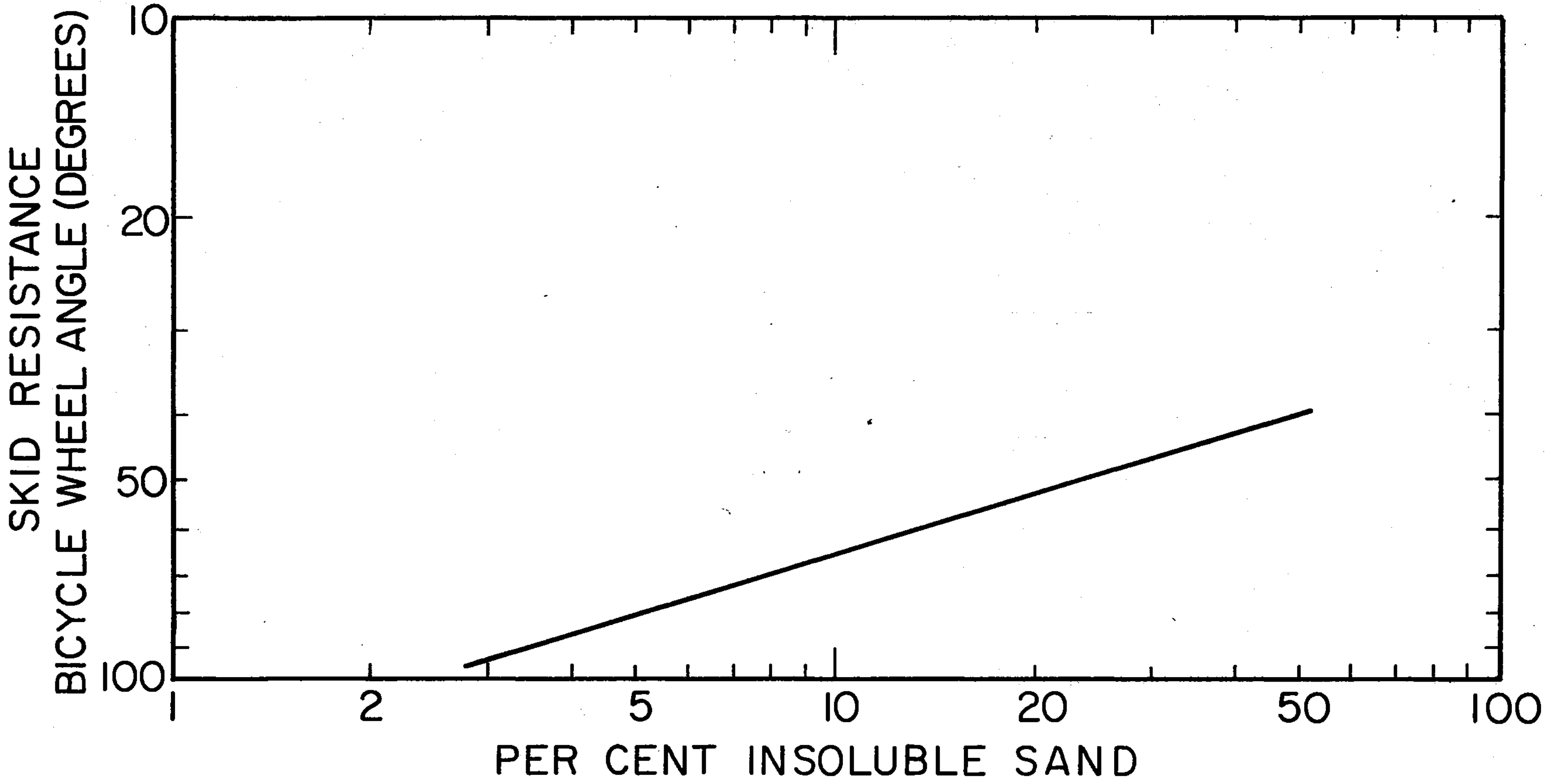

Figure 2. Relationship Between Skid Resistance and Amount of Insoluble Sand (After Gray and Renninger, 12)

the percent total insoluble residue of the aggregate decreased, the stopping distance number decreased, in other words, the skid resistance decreased. As a result of these tests, acid-insoluble tests and specification limits were proposed and are now in use in Virginia. Blends of limestone and non-polishing aggregate were used in surface courses and these proved highly successful in increasing pavement skid resistance.

Oklahoma Highway Department Report on Skid Resistance

The Oklahoma Highway Department conducted an extensive research project on the subject of skid resistance of highway pavements constructed of various kinds of aggregate. A part of this research contained a study by McCasland (14) of the polishing tendencies of aggregates available in and around Oklahoma. He used the British Pendulum Testing apparatus (15) to measure a "polished stone value" and an accelerated polishing method, the British Wheel Test (15), to actually polish the aggregate. Polished stone value is defined as a frictional value expressed in British Pendulum numbers (BPN) derived from (aggregate) specimens subjected to nine hours of polishing on the British Wheel Test. In general, the procedure followed by McCasland was to place the aggregate passing the 1/2" sieve but retained on the #4 sieve into curved molds, 3.5 inches by 1.75 inches in size, having a thickness of a single layer of aggregate using a polyester glue. These aggregate molds were tested on the pendulum apparatus to determine the before polish value, in other words, the amount of frictional resistance that the aggregate has in its natural state. The aggregate molds were then placed on a large wheel and subjected to a controlled polishing to simulate traffic wear. The type and size of the grit abrasive and the number of revolutions were known. After the accelerated polishing, the aggregate molds were again tested on the pendulum apparatus to obtain an after polish value or the polished stone value. Results of the study, given in Table I, showed that the sandstones had the least difference in the before and after polish stone values because of the faster wearing away or loss of the soft cementing agent exposing the individual grains, whereas the limestones and the gravels and cherty gravels had the higher differences in the polished stone values. Also, the sandstones had the higher before and after polished stone values than did the other kinds of aggregates. Figure 3 shows a relation of the polish stone value (BPN) to the percent acid-insoluble residue of the aggregate. Insoluble residue, as defined by McCasland, is the residue remaining after subjecting a sample (of aggregate) to dilute hydrocloric acid (HCl). The leaching process dissolves the carbonate fraction and leaves the noncarbonate fraction in the form of a residue.

Much research has been conducted to test skid resistance of pavements incorporating many different kinds and blends of aggregates. Test sections have been constructed in the lab and in the field to actually measure a coefficient of friction. Possibly, an assumption made by many authors was that an adequate pavement resulted regardless of the kinds of aggregates incorporated in the mixture. This study was conducted to determine if any detrimental effects, loss of stability and/or cohesion, resulted from the blending of these Oklahoma siliceous aggregates into a standard limestone-asphalt mixture.

TABLE I

POLISHED STONE VALUES OF AGGREGATES
EMPLOYED IN MIX DESIGN STUDY

Aggregate	Before Polish Value (BPN)[1]	Polished Stone Value (BPN)	Polishing Difference
Onapa Siliceous Sandstone	50.4	49.4	1.0
Keota Siliceous Sandstone	49.9	44.4	5.5
Cyril Dolomitic Sandstone	46.1	39.3	6.8
Hugo Chert Gravel	35.5	27.5	8.0
Broken Bow Chert Gravel	47.3	38.5	8.8
Gore Gravel	40.3	31.0	9.3
Miami Chert	43.1	33.1	10.0
Cooperton Limestone	42.4	31.8	10.6
Asher Chert Gravel	42.9	32.2	10.7
Hartshorne Limestone	48.3	37.2	11.1
Stringtown Siliceous Limestone	43.3	31.8	11.5

[1]British Pendulum Number (After McCasland, 14).

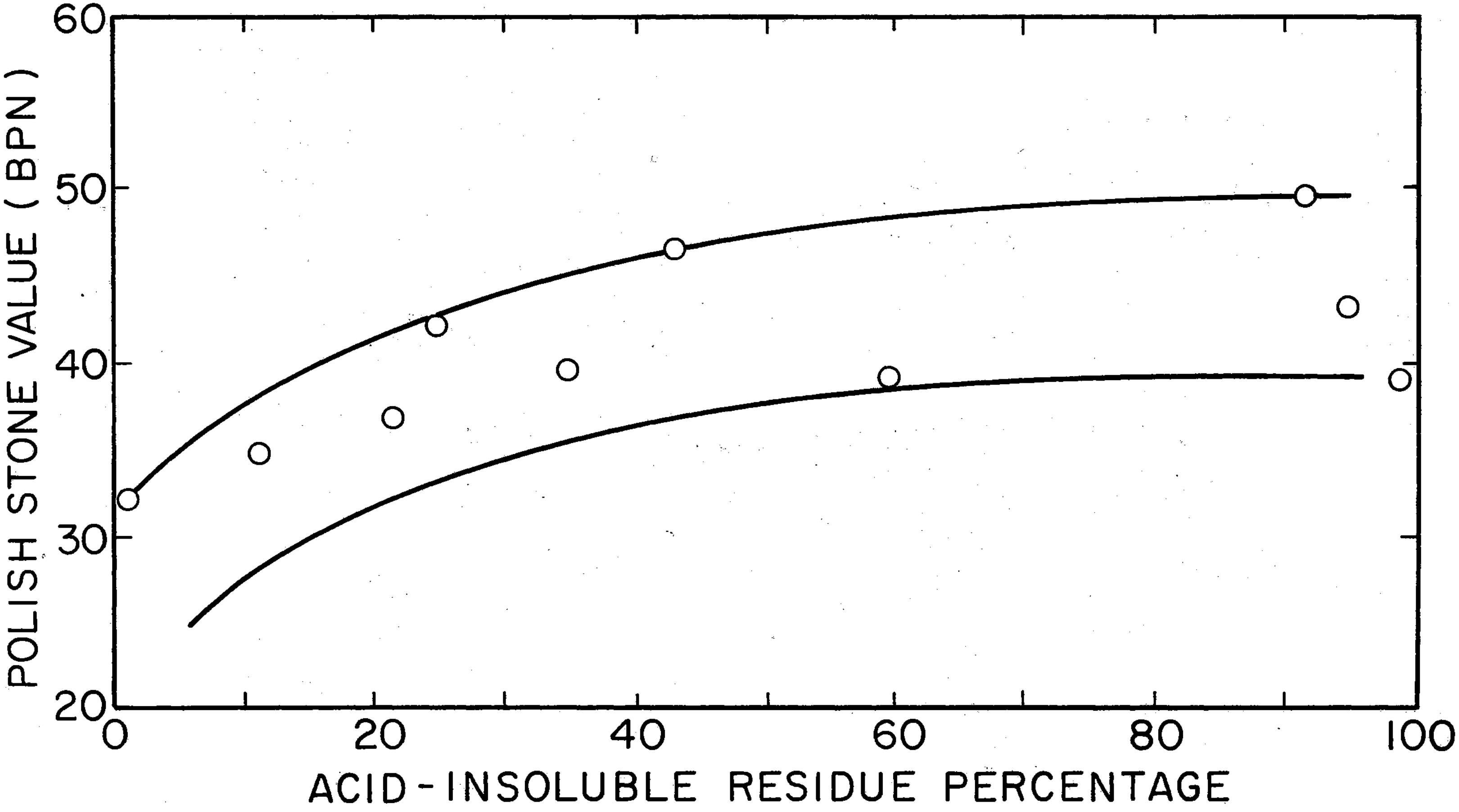

Figure 3. Relation of Polish Stone Value to Acid-Insoluble Residue (After McCasland, 14)

CHAPTER III

AGGREGATE DESCRIPTION

The aggregates used in this study were obtained from various sources in the eastern and southern portions of Oklahoma. They were selected primarily on the basis of their acid-insoluble residue content and their potential usage as road construction materials.

In order to provide a more complete description of each of the aggregates employed, two methods are used to characterize them. The first is a general megascopic description including color, type of rock, geologic formation and age, and a petrological breakdown. The second is a tabular listing of a physical description which includes the percent insoluble factor, bulk specific gravity, percent water absorption, percent fractured faces, Los Angeles abrasion, and a chemical analysis. Polish stone values for the aggregates are shown in Table I in Chapter II.

Megascopic Description

Cooperton limestone is a gray, very uniform and pure limestone from the Upper Arbuckle formation of the Ordovician system. It was obtained from southwestern Oklahoma, just north of the Wichita Mountains. The aggregate consists of 98% limestone or calcium carbonate and 2% silica, magnesium, and iron.

Arkhola sand is a brown to red silica sand obtained from the Arkansas River north of Muskogee, Oklahoma. The finer sand, #80 to #200 sieve size, is from aeolian deposits on the bank of the Arkansas River near Ft. Gibson, Oklahoma. This sand is practically pure silica dioxide.

Asher gravel is a whitish pink to brown, fairly homogeneous, chert gravel obtained from a conglomerate of the Wellington Admire formation of the Permian system in the Prairie Plains Homocline in central Oklahoma. The aggregate consists of 94% banded chert and 6% cherty limestone and is coated with a red dust.

Miami chert is a whitish gray crushed chert obtained from the Boone formation of the Mississippian system. The aggregate consists of 92% chert, 6% limestone, and 2% dolomite, zinc, and iron. The aggregate comes from a zinc mine operation in northeastern Oklahoma.

Onapa sandstone is a gray siliceous sandstone from the Bluejacket formation of the Pennsylvanian system. It was obtained from the Arkhoma Basin in eastern Oklahoma. The aggregate consists of 68% quartz grains, 13% chert, and 1% miscellaneous material.

Stringtown limestone is a gray siliceous limestone from the Wapanucka formation of the Pennsylvanian system in southeastern Oklahoma. The aggregate consists of 60% siliceous limestone, 30% chert, and 10% sandy shale.

Cyril sandstone is a gray calcareous to dolomitic sandstone, very nearly a siliceous sandy limestone from the Wapanucka formation of the Pennsylvanian system. It was obtained from the Anadarko Basin region in southwestern Oklahoma. The aggregate consists of 53% calcite (dolomitic), 42% quartz, and 4% miscellaneous material.

Broken Bow gravel is a light brown heterogeneous siliceous chert gravel obtained from an alluvial deposit of the Quaternary system in southeastern Oklahoma. The aggregate consists of 50% quartz, 24% chert, 21% quartzitic sandstone, and 4% metamorphic rock.

Gore gravel is a multi-colored, heterogeneous, alluvial gravel of the Quaternary system from east central Oklahoma. The aggregate consists of 59% quartz, 22% chert, 9% granite, 8% feldspar, and 1% sandstone.

Hugo gravel is a brown, fairly homogeneous, chert gravel obtained from terrace deposits of the Quaternary system in southeastern Oklahoma. The aggregate consists of 94% chert and 6% quartzitic sandstone.

Keota sandstone is a gray siliceous sandstone obtained from the Arkhoma Basin in eastern Oklahoma. It is found in the Bluejacket formation of the Pennsylvanian system. The aggregate consists of 80% quartz, 18% chert, and 1% feldspar.

Physical Properties

The physical description of the aggregates are given in Table II. The average bulk specific gravity (SSD), the water absorption of the coarse aggregate, and the percent fractured faces were determined in the study; whereas, the acid-insoluble residue, the Los Angeles abrasion, and the chemical analysis were previously determined by the Oklahoma Highway Department.

TABLE II

PHYSICAL PROPERTIES OF AGGREGATES

Aggregate	Okla. Hwy. Dept. Number	Acid-Insoluble Residue Percentage[1]	Average Bulk Specific Gravity	Water Absorption of C.A.	Fractured Faces (%)	Los Angeles Abrasion[1]	Chemical Analysis (%)[1]				
							$CaCO_3$	$MgCO_3$	SiO_2	Al_2O_3	Misc.
Cooperton Limestone	38 - 01	1.2	2.67	1.14	--	25.8	91.0	2.4	4.5	Trace	Trace
Asher Chert Gravel	63 - 01	99.8	2.38	3.45	50.3	20.0			Not Available		
Miami Chert	58 - 01	95.4	2.53	1.86	--	21.2			Not Available		
Onapa Siliceous Sandstone	46 - 01	92.1	2.33	5.27	--	35.9	2.2	1.6	83.0	10.6	Trace
Stringtown Siliceous Limestone	03 - 01	72.8	2.52	1.42	--	19.8	30.5	Trace	62.4	2.4	Trace
Cyril Dolomitic Sandstone	08 - 01	59.2	2.63	1.01	--	31.7	22.1	15.4	59.2	2.6	Trace
Broken Bow Chert Gravel	45 - 01	98.3	2.53	1.61	64.7	25.0			Not Available		
Gore Gravel	68 - 01	97.9	2.46	2.20	68.5	19.0			Not Available		
Hugo Chert Gravel	12 - 01	99.0	2.53	1.35	51.1	25.0			Not Available		
Keota Siliceous Sandstone	31 - 01	96.3	2.37	3.19	--	34.5	Trace	Trace	90.6	6.2	Trace

[1]Source: Oklahoma Highway Department.

CHAPTER IV

LABORATORY MIX DESIGN PROCEDURE

Conducting an investigation based upon laboratory test results requires that a detailed procedure be outlined and followed explicitly for handling, preparing, and testing the specimens. The laboratory procedure followed in this study is presented in the following four parts: handling of the aggregate and asphalt, calculating the proposed mix design batch weights, preparing the test specimens, and testing the specimens.

Aggregate and Asphalt

The sources of aggregate for the study were selected by the Oklahoma Highway Department in accordance with their long-range investigation of highway pavement skid resistance. A relatively pure limestone from Cooperton, Oklahoma and an Arkansas River sand from a source near Muskogee, Oklahoma, were used as a standard aggregate mixture. Several other aggregates, primarily siliceous in nature, were selected for blending with the standard mix in specified percentages. These siliceous aggregates were obtained from Oklahoma sources presently furnishing large quantities of material for highway construction. The aggregates were taken from quarry stockpiles using accepted sampling methods to obtain representative samples and transported in plastic bags.

An 85-100 penetration grade asphalt cement was selected for use in the mixtures since this is the standard paving grade asphalt material specified by the Oklahoma Highway Department (16). The asphalt was obtained from a source in Stroud, Oklahoma, and was received in five gallon buckets. These buckets of asphalt were heated to approximately 250 F and poured into smaller containers to facilitate subsequent handling.

In the laboratory, the sampled aggregate was dried to constant weight in a large gas-fired oven. The dried aggregate was then sieved into specified sizes using an 8" Ro-Tap sieve shaker, an 8" vibratory sieve shaker, and a 15" by 24" Gilson screen shaker. The Cooperton limestone was separated into the following fractions: 3/4" - 1/2", 1/2" - 3/8", 3/8" - #4, #4 - #10, and minus #200. The Arkhola sand was separated into the following fine aggregate sizes: #10 - #40, #40 - #80, and #80 - #200. Since only the coarse fractions of the siliceous aggregates were used in the mixes, they were sized as follows: 3/4" - 1/2", 1/2" - 3/8", 3/8" - #4, and #4 - #10. The respective sized aggregate was placed in clean five gallon buckets and covered with a tight lid.

The specific gravity and water absorption of the aggregate was determined using a method of testing outlined by Manke (17). This method is a modification of the test procedure outlined by ASTM Designation: C 127 and C 128 (18) to determine the bulk and apparent specific gravity and the water absorption of that aggregate passing the 3/4" sieve but retained on the #80 sieve and the apparent specific gravity of the minus #80 plus #200 sieve size aggregate. The bulk specific gravity and water absorption of the various aggregates are listed in Table II in Chapter III.

The specific gravity of the asphalt cement was determined using the pycnometer method, ASTM Designation: D 70 (19). This specific gravity was used to calculate the theoretical maximum specific gravity of the asphalt-aggregate mixture as described in Chapter VI.

Mix Design

The aggregate gradation used for the mixes was based on the Oklahoma Highway Department specifications for the Type B surface or base course mixture. The upper and lower limits of the specifications and the mid-point gradation used for the mixes are given in Table III. Figure 4 shows a plot of the specification limits and the mid-point gradation. The Type B mix has a coarser gradation than the Type C surface course normally used for highway construction and was selected so that the larger sizes of the siliceous aggregates could be incorporated into the mix. Results of several studies have indicated that the coarse aggregate in the pavement surface governs, to a large extent, the skid resistance of the pavement (8, 9).

Specifications for the Type B mix stipulate an asphalt content range of 5 to 7 1/2%, by weight of the total mix. However, the range used in this study was from 4 to 6 1/2%. Selection of the exact mid-point gradation of the Type B mixture resulted in a very dense aggregate conbination and this necessitated a reduction of the asphalt content to obtain adequate stability of the mix. Molded specimens containing more than 5 1/2% asphalt appeared extremely rich and many of them slumped under their own weight during cooling. Also, excessive deformations of these specimens, in many cases, prevented the determination of stability.

TABLE III

OKLAHOMA HIGHWAY DEPARTMENT SPECIFICATIONS AND MID-POINT GRADATION OF TYPE B MIX[1]

Sieve Size	Per Cent by Weight Passing	
	Specification	Mid Point Gradation
3/4"	100	100
1/2"	80 - 100	90
3/8"	70 - 90	80
#4	50 - 70	60
#10	35 - 50	42.5
#40	15 - 30	22.5
#80	10 - 20	15
#200	3 - 9	6

[1]Sec. 708.01 of Standard Specifications, (16).

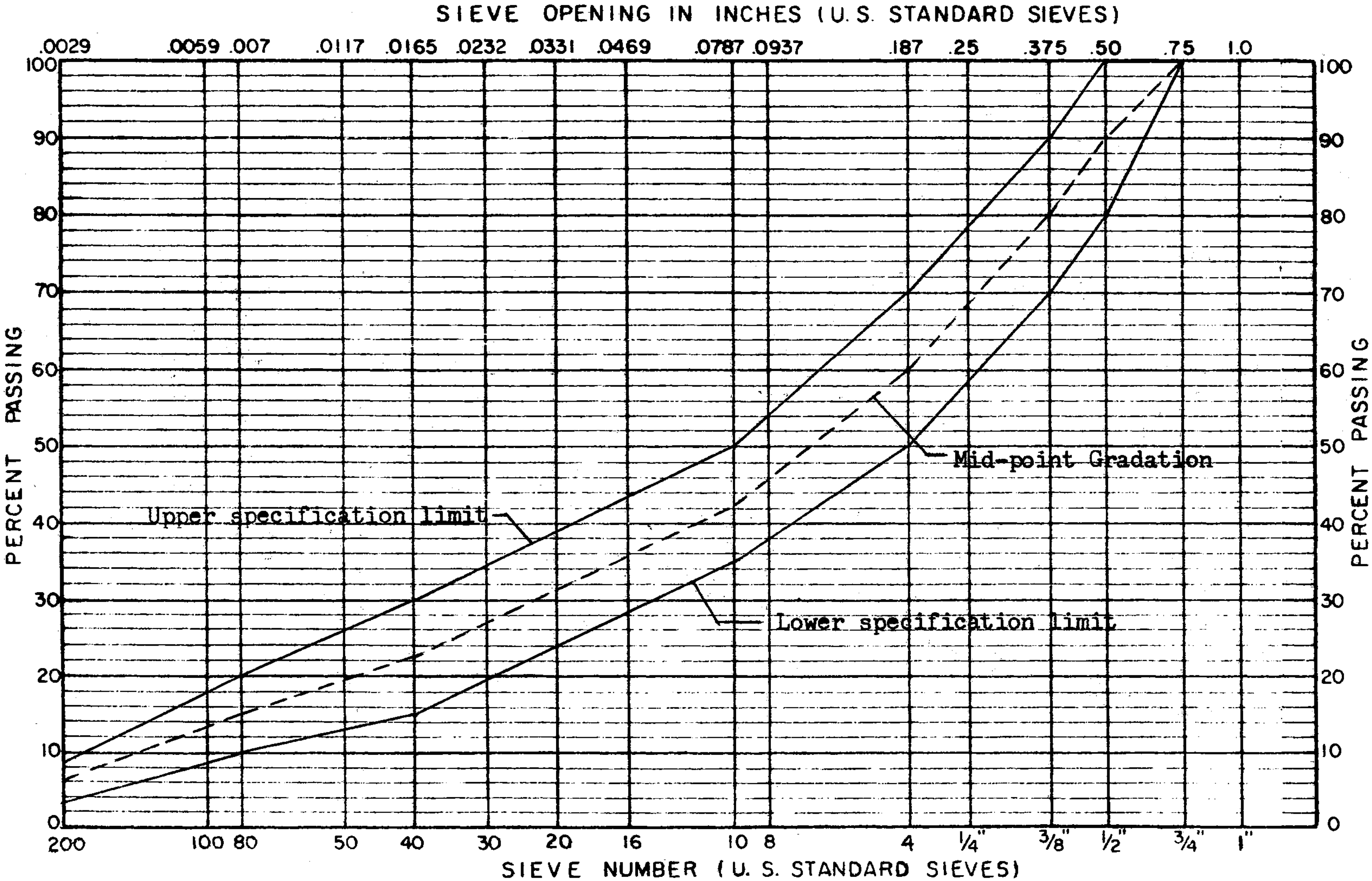

Figure 4. Type "B" Specifications and Mid-Point Gradation (16)

Mixtures containing 4, 4 1/2, 5, 5 1/2, 6, and 6 1/2% asphalt by total weight were prepared for each aggregate or aggregate blend used in the study. Four test specimens were molded at each asphalt content. These molded specimens were 4 inches in diameter, approximately 2 inches in height, and contained 1000 grams of graded aggregate.

The laboratory batch weights for the standard Cooperton limestone-Arkhola sand mixture are shown in Table IV. The coarse aggregate fractions of the various siliceous materials were incorporated in this standard mixture in amounts based on the acid-insoluble residue percentage (IRP) of each respective aggregate.

The acid-insoluble residue percentage was determined by subjecting a known weight of aggregate to a dilute hydrochloric acid (HCl) solution. The HCl dissolves the carbonate fraction in the aggregate and leaves the non-carbonate fraction in the form of a residue. Therefore, a pure carbonate aggregate would have an IRP = 0% and a pure silica aggregate would have an IRP = 100%. The method for determining the acid-insoluble residue percentage is outlined in test method OHD-L-25 of the Materials Division's Laboratory Testing Procedures Manual (1). The acid-insoluble residue values for the various aggregates are listed in Table II in Chapter III.

Asphalt-aggregate mixtures containing 20, 30, and 40% (by weight of aggregate) acid-insoluble material were studied. These percentages included the acid-insoluble residue contained in the Cooperton limestone. Sample calculations used to determine the percentage of siliceous aggregate to be incorporated in a mixture are illustrated below.

TABLE IV

LABORATORY BATCH WEIGHTS OF STANDARD MIX

Aggregate	Sieve Fraction	Weight of Each Sieve Fraction (grams)	Cumulative Weight of Sieve Fractions (grams)
Cooperton Limestone	3/4" - 1/2"	100	100
	1/2" - 3/8"	100	200
	3/8" - #4	200	400
	#4 - #10	175	575
Arkhola Sand	#10 - #40	200	775
	#40 - #80	75	850
	#80 - #200	90	940
Cooperton Limestone	minus #200	60	1000

Given: Onapa Sandstone IRP = 92.1%

Cooperton Limestone IRP = 1.2%

For: 20% acid-insoluble residue in aggregate mixture

Find: % Onapa (by weight of aggregate) to be used in mixture

1. 20% - 1.2% = 18.8%

2. $\% \text{ Onapa} = \frac{18.8\%}{92.1\% - 1.2\%} \times 100 = 20.68\%$

Using the above example, 20.68% of the coarse fractions of the Cooperton limestone was replaced by like fractions of the Onapa sandstone to obtain 20% insolubles in the coarse aggregate portion of the mixtures. Similar calculations were used for the 30% and 40% mixtures. The limestone-sand-siliceous aggregate mixtures were combined according to the sample batch weights given in Table V.

Preparation of Specimens

Pans containing 1000 grams of the sized aggregates were placed in a large gas-fired oven at 250 F ± 10 F for a period of 4 hours. The asphalt cement was placed in a large forced-air oven at 250 F ± 10 F for a similar period. Using a Mettler P-3 balance, the hot asphalt was weighed into the hot aggregate. Mixing of the asphalt-aggregate was accomplished using a Hobart C-100 mixer with a wire whip attachment (see Figure 5). The mixer bowl and whip were preheated in a 250 F oven to minimize heat loss during mixing and to prevent the mixture from sticking. During mixing, a Bunsen burner flame was passed beneath the mixer bowl to keep the mixture at the proper temperature until all the aggregate particles were coated. From 1 to 4 minutes of mixing was

TABLE V

SAMPLE LABORATORY BATCH WEIGHTS OF
LIMESTONE--SILICEOUS AGGREGATE MIX

Aggregate	Sieve Fraction	Weight of Each Sieve Fraction (grams)	Percentage of Aggregate	Adjusted Weight of Each Sieve Fraction (grams)	Cumulative Weights of Sieve Frictions (grams)
Cooperton Limestone	3/4" - 1/2"	100	79.32	79.32	79.32
	1/2" - 3/8"	100		79.32	158.64
	3/8" - #4	200		158.64	317.28
	#4 - #10	175		138.81	456.09
Onapa Sandstone	3/4" - 1/2"	100	20.68	20.68	476.77
	1/2" - 3/8"	100		20.68	497.45
	3/8" - #4	200		41.36	538.81
	#4 - #10	175		36.19	575.00
Arkhola Sand	#10 - #40	200	100	200	775.00
	#40 - #80	75		75	850.00
	#80 - #200	90		90	940.00
Cooperton Limestone	minus #200	60	100	60	1000.00

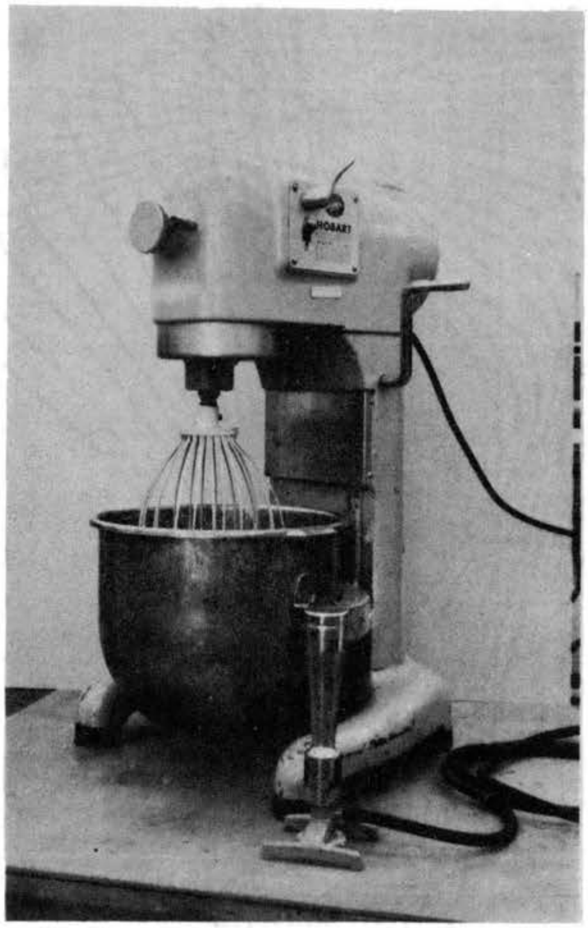

Figure 5. Hobart C-100 Mixer With
Wire Whip Attachment

required to achieve good coating of the particles. The asphalt-aggregate mixture was then placed in a holding oven (250 F ± 5 F) to await molding.

The method used in this study to mold or compact the asphalt-aggregate mixtures was essentially the same as that used by the Texas Highway Department, Tex-206-F (2). The actual procedure that was followed is outlined in Appendix A. The compactor was a motorized gyratory shear apparatus similar to that currently used by the Texas Highway Department (see Figure 6).

In general, the procedure was to remove the hot asphalt-aggregate mixture from the holding oven and place it into the gyratory mold in three approximately equal lifts or layers. The mold and base plate were heated in an oven to approximately 250 F to prevent loss of heat of the mixture. The mold (and mixture) were placed on the rotating platen of the compactor and an initial low pressure of 50 psi was applied to the mixture. The platen was rotated, forcing the mold through three complete gyrations, and the low pressure was applied again by the press. This was continued until one stroke of the pump handle gave an indicated reading of 150 psi on the mixture. Then, a leveling pressure of 2500 psi was applied to the mixture to complete the compaction. The mold was then removed from the compactor and the molded specimen extruded from the mold using an arbor press. The specimen was placed on a Masonite square and allowed to cool to room temperature. The mold, base plate, rotating platen, and press ram face were cleaned after each specimen was molded.

This procedure for molding the specimens conforms closely to that described in the test method, OHD-L-8 (1), except that the Oklahoma

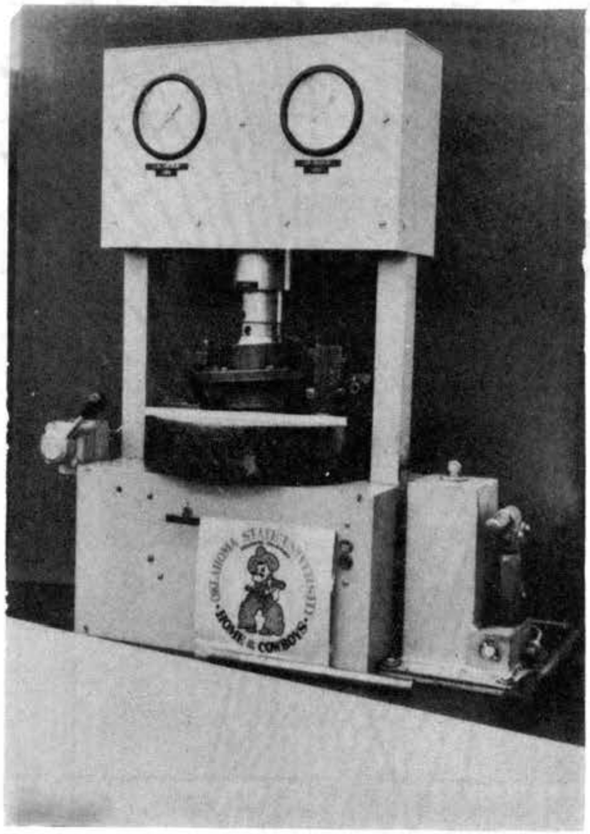

Figure 6. Motorized Gyratory Shear Compaction Device

Highway Department uses a compaction apparatus in which gyratory shearing action is applied manually. The motorized compactor was designed to duplicate, uniformly, the manual gyratory action applied to a specimen. While manual gyration is operator dependent and can result in wide variation of applied compactive effort, the amount of compactive force delivered by the motorized compactor is more nearly constant and results in more uniformly compacted specimens.

Testing the Specimens

After the specimens had cooled to room temperature, the height of each specimen was determined using the device shown in Figure 7. Five measurements were taken, at the center and at the ends of two orthogonal diameters, and averaged.

The bulk specific gravity of the compacted specimens was determined using the method outlined in test procedure, OHD-L-14, Method B (1). Briefly, the procedure was to weigh a specimen in air and weigh it suspended in water, and the bulk specific gravity was then calculated from the following equation:

$$G_b = \frac{A}{A - B}$$

where: G_b = bulk specific gravity of compacted specimen

A = weight of compacted specimen in air (grams)

B = weight of compacted specimen in water (grams).

The stabilometer test, ASTM Designation: D 1560 (19), was used to determine the stability or resistance to deformation exhibited by the various mixes. The Hveem stabilometer, a triaxial compression device, is used to measure the transmitted horizontal pressure developed in a

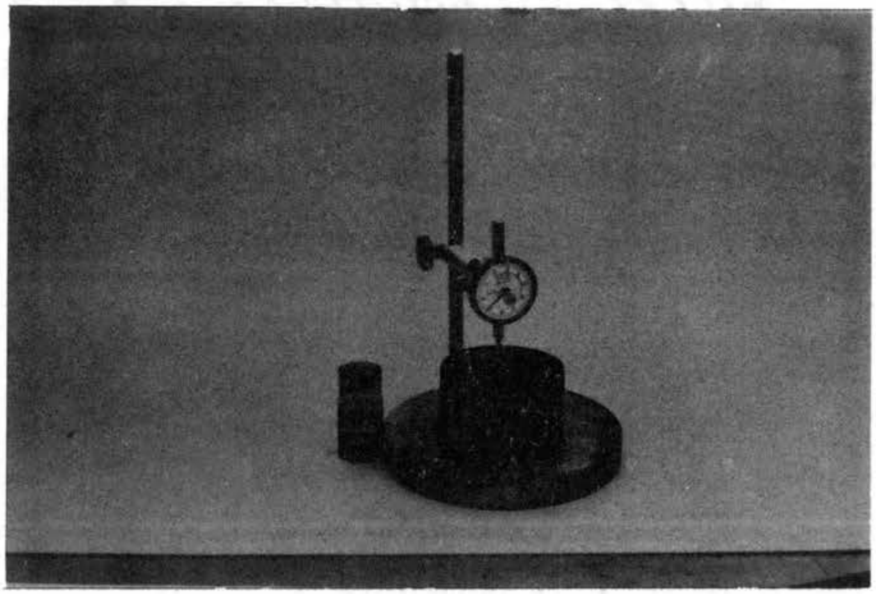

Figure 7. Device to Measure the Height of Specimen

compacted asphalt-aggregate specimen subjected to a given vertical pressure. The test values indicated the relative stability of a pavement constructed from the test materials to resist plastic deformation under the action of traffic.

Prior to testing, the molded specimens were brought to the test temperature of 140 F ± 5 F, the stabilometer calibration was checked and adjusted, and the head speed of the compression testing machine was set at 0.05 inches per minute. The specimen was placed in the stabilometer with a steel follower on top of the specimen and the entire assembly was then positioned in the compression machine. Figure 8 shows the stabilometer in position for testing on a Versa-Tester 30,000 pound testing machine. The specimen was loaded to 6000 pounds vertical load and the horizontal pressure was read from the stabilometer test gage at 1000 pound increments of vertical load.

The stability value, S, was then determined from a conversion chart, or graphical solution of Hveem's equation:

$$S = \frac{22.2}{\dfrac{P_h D_2}{P_v - P_h} + 0.222}$$

where: S = stabilometer or relative stability value

P_v = vertical pressure at 400 psi

P_h = horizontal or lateral pressure at 400 psi

D_2 = final displacement in inches multiplied by 10.

This mathematical expression does not take into consideration the height of the tested specimen. Because of the influence of the height on the relative stability value, the measured values (for specimens of

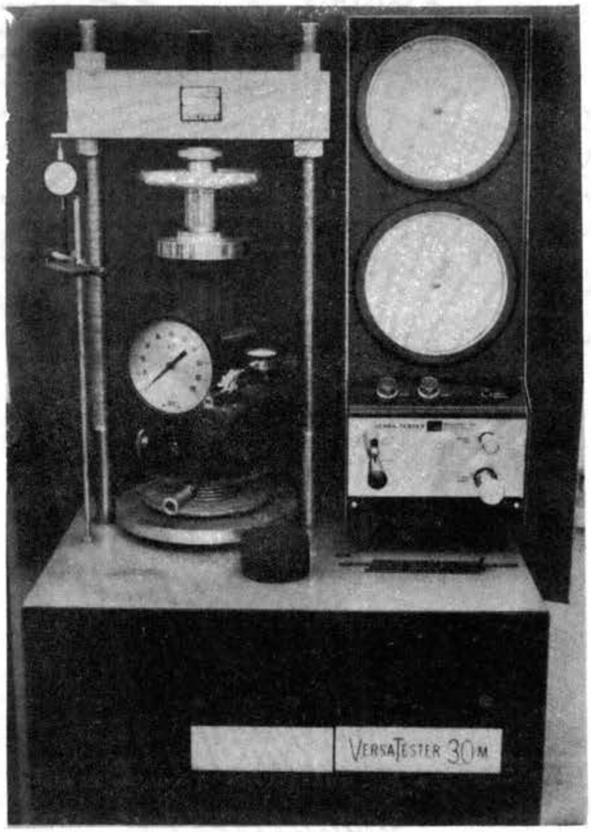

Figure 8. Stabilometer and Compression Testing Machine

various heights) were converted to equivalent stability values for a standard height specimen using a correction chart.

The cohesiometer test, ASTM Designation: D 1560 (19), was performed on the specimens previously tested for stability. This test provides a measure of the cohesive resistance or tensile strength of a compacted asphalt-aggregate mixture. The cohesion of a compacted specimen is determined by measuring the force required to break or bend the specimen as a cantilever beam by means of the Hveem cohesiometer. The cohesiometer value, C, is a numerical value expressed as weight in grams of lead shot required to break, in tension, a test specimen equivalent to 3 inches in height and 1 inch in width. Figure 9 shows the Hveem Cohesiometer.

Following the stability test, the compacted specimens were placed in an oven (140 F ± 5 F) for approximately two hours. The thermostat in the cohesiometer cabinet was adjusted to maintain a test temperature of 140 F ± 2 F and the shot release mechanism was calibrated to release 1800 ± 20 grams per minute of lead shot into the receiving bucket. The specimen was placed in the cohesiometer, the top plates were leveled and tightened, and the lid was closed. When the inside temperature reached 140 F, the loading arm was unlocked and the mechanism allowed to release the shot until the end of the loading arm moved vertically downward 1/2 inch. At this point, the shot mechanism was triggered to shut off the flow of shot and the weight of the shot in the bucket was determined. The cohesiometer value, C, was calculated according to the equation:

$$C = \frac{L}{W(0.2\ H + 0.044\ H^2)}$$

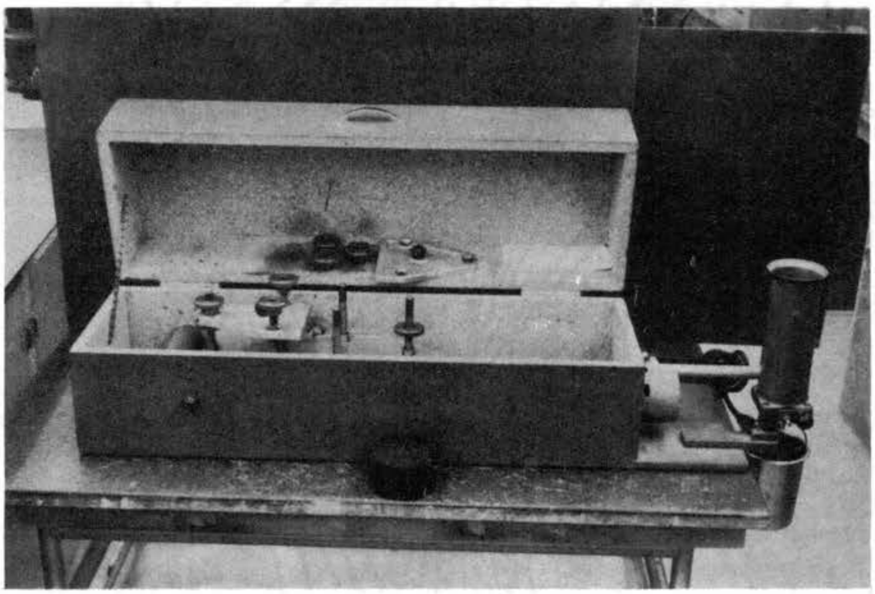

Figure 9. Cohesiometer

where: C = cohesiometer value (grams per inch width corrected to a 3 inch height)

L = weight of shot (grams)

W = diameter of specimen (inches)

H = height of specimen (inches).

The stability values and the cohesiometer values were plotted against the asphalt content of the respective mixtures. These plots were used to determine the optimum asphalt content of each mixture and to ascertain any trends or effects that occurred due to the incorporation of the siliceous aggregates into the standard limestone-sand mix.

The laboratory procedures followed for determining the percent crushed faces (fractured faces) of an aggregate, the bulk impregnated specific gravity, and Rice's Method for determining the maximum specific gravity of uncompacted mixtures are outlined in Chapter VI.

CHAPTER V

LABORATORY MIX DESIGN RESULTS

The purpose of this study was to investigate any effects that might develop by incorporating various siliceous aggregates into a standard limestone-sand aggregate mixture. The investigation involved the use of the stabilometer and cohesiometer test results to evaluate the asphalt-aggregate mixes. The evaluation included the determination of the optimum asphalt content based on the maximum stabilometer value and the percent density of the compacted mix. Each aggregate combination was analyzed based on the inherent properties of the blended aggregates and how they affected the stability and cohesion of the compacted asphalt-aggregate mixtures.

According to Hveem, "the surface characteristic of the mineral aggregates is the most important single quality affecting stability of bituminous pavements" (4). In general, results of much research have shown that crushed aggregate blends developed better stability than rounded or uncrushed aggregate blends.

Herrin and Goetz (20) conducted a study to determine any effects that aggregate shape might have on the properties of a bituminous mix. They tested dense-graded mixtures of natural sand as the fine aggregate fraction and varied the shape of the coarse aggregate fraction from 0 to 100% crushed gravel. Results showed that little measurable difference in stability occurred regardless of the percent crushed aggregate in the mix. The cohesion remained fairly constant as the amount of crushed aggregate incorporated in the mix was increased.

Hargett (21) also studied the effects of aggregate size, surface texture, and shape on bituminous mixtures. He observed that a rough surface texture of an aggregate particle, as reflected by the high angle of internal friction, effectively increased the stability of the mix. The interlocking of the aggregate, based on the shape of the particles, affected the stability. Fractured particles induced better interlocking which increased stability. On the other hand, the cohesive strength of the mix did not reflect any effects of the size, shape, or surface texture of the aggregate. Cohesion was affected by the inherent material properties of the asphalt binder.

Stability of a bituminous specimen is affected by the amount of effort used to compact the mixture. Interparticle friction can only occur if the particles are placed close enough together so that their movement past each other is retarded. Adequate compaction puts these aggregate particles in intimate contact with each other.

The tensile strength of a compacted bituminous specimen is reflected by the cohesion that exists between the asphalt and the aggregate. This cementing action can only occur if adequate compaction has placed these materials in close contact with each other and sufficient asphalt is available to entirely coat the particles.

Most laboratory mix design studies are based upon the fact that all the specimens are subjected to equal compactive efforts, in other words, the criterion of acceptability is constant density. It is intended that the density obtained in the laboratory will be reproduced in the field. However, field supervision of compaction to laboratory density leaves much to be desired. If the bituminous mix is not compacted to this same laboratory density in the field, the pavement cannot be expected to have the same characteristics as the specimen in the laboratory.

As was discussed earlier, the selection of the exact mid-point gradation of the Type B specification limits resulted in a very dense aggregate mixture. Results of voids in the mineral aggregate (VMA) calculations gave values ranging from 10.8% to 13.6%. Therefore, to obtain adequate stability, the asphalt content range was lowered to include 4 and 4 1/2% by weight of mix. Since the selected aggregate gradation proved very dense, some of the mixtures became what are called critical mixtures in that slight variations of the asphalt content resulted in large changes in the stabilometer test values.

However, results of the cohesiometer test failed to show any indication that the selected gradations had critical tendencies. Possibly, this test was insensitive to the density of the aggregate mixture. On the other hand, the cohesiometer test was very sensitive to the operator conducting the test, in that different operator techniques resulted in widely different cohesiometer values for like specimens. The cohesiometer test results tended to increase in magnitude toward the latter part of the study. However, this was attributed not so much to actual increased cohesion of the aggregate blends, as to operator experience and the more uniform manner in which the tests were performed.

Table VI shows the stabilometer value, the cohesiometer value, and the percent density, based on Rice's method, of each of the aggregate combinations at the selected optimum asphalt content.

TABLE VI

STABILOMETER VALUE, COHESIOMETER VALUE AND PERCENT DENSITY OF COMPACTED SPECIMEN AT OPTIMUM ASPHALT CONTENT

Aggregate	Insoluble Residue Percentage	Optimum Asphalt Content	Stabilometer Value at Optimum	Cohesiometer Value at Optimum	Percent Density of Compacted Specimen (Rice's Method)
Cooperton Limestone	na	4.25	42	192	96.5
Asher Chert Gravel	20	4.25	42	183	96.3
	30	4.25	37.5	187	96.2
	40	4.25	37	164	96.2
Miami Chert	20	4.5	39	163	96.5
	30	4.5	40	167	97.0
	40	4.5	37.5	185	96.7
Onapa Sandstone	20	4.5	41.5	200	96.5
	30	4.5	41	180	95.7
	40	4.5	43	156	96.1
Stringtown Limestone	20	4.25	42	177	96.3
	30	4.25	41	162	96.3
	40	4.25	40	175	96.1
Cyril Sandstone	20	4.25	39	196	96.4
	30	4.25	40.5	266	96.7
	40	4.25	38	204	96.6
Broken Bow Chert Gravel	20	4.0	37.5	190	96.5
	30	4.0	35	307	98.2
	40	4.0	40	296	97.5
Gore Gravel	20	4.0	40	311	97.6
	30	4.0	36.5	302	97.0
	40	4.0	41	323	97.7
Hugo Chert Gravel	20	4.0	38	330	97.7
	30	4.0	36	278	97.9
	40	4.0	37	301	97.8
Keota Sandstone	20	4.0	44	275	97.0
	30	4.0	44	343	96.6
	40	4.0	42	394	97.6

Standard Mix--Limestones

The standard aggregate mixture used in this study was composed of crushed, angular limestone from Cooperton, as the coarse fraction, and rounded sand from the Arkansas River, as the fine aggregate fraction, the limestone comprising 63.5% of the mix by weight. Results of the stability test, Figure 10, showed that at the optimum asphalt content, the stabilometer value was 42. Possible the rounded fine aggregate particles tended to lower the maximum stabilometer values of this standard mix. A bituminous mix composed entirely of limestone aggregate was not tested.

Results of the study conducted by Herrin and Goetz showed that by substituting a natural sand in the fine aggregate fraction, in place of limestone screenings, the stability of the mix was decreased substantially (20).

Cohesion in a bituminous mix, as discussed previously, exists due to the adhesion of the asphalt binder to the aggregate particles and the coherent strength of the asphalt films. In other words, if the asphalt fails to adhere adequately to the aggregate, the cohesive or tensile strength of the compacted mix is reduced. In general, asphalt adheres to the limestone aggregate quite well because limestone is, by nature, a hydrophobic particle. On the other hand, asphalt does not adhere too well to hydrophilic aggregates, such as sand or chert. Figure 11 shows the results of the cohesiometer test on the standard mix. Hveem suggested that a minimum cohesiometer value of 50 be used in designing bituminous pavements. Test values for the standard mix were well above this suggested minimum, at optimum asphalt content. In fact, all the aggregate blends tested above 50.

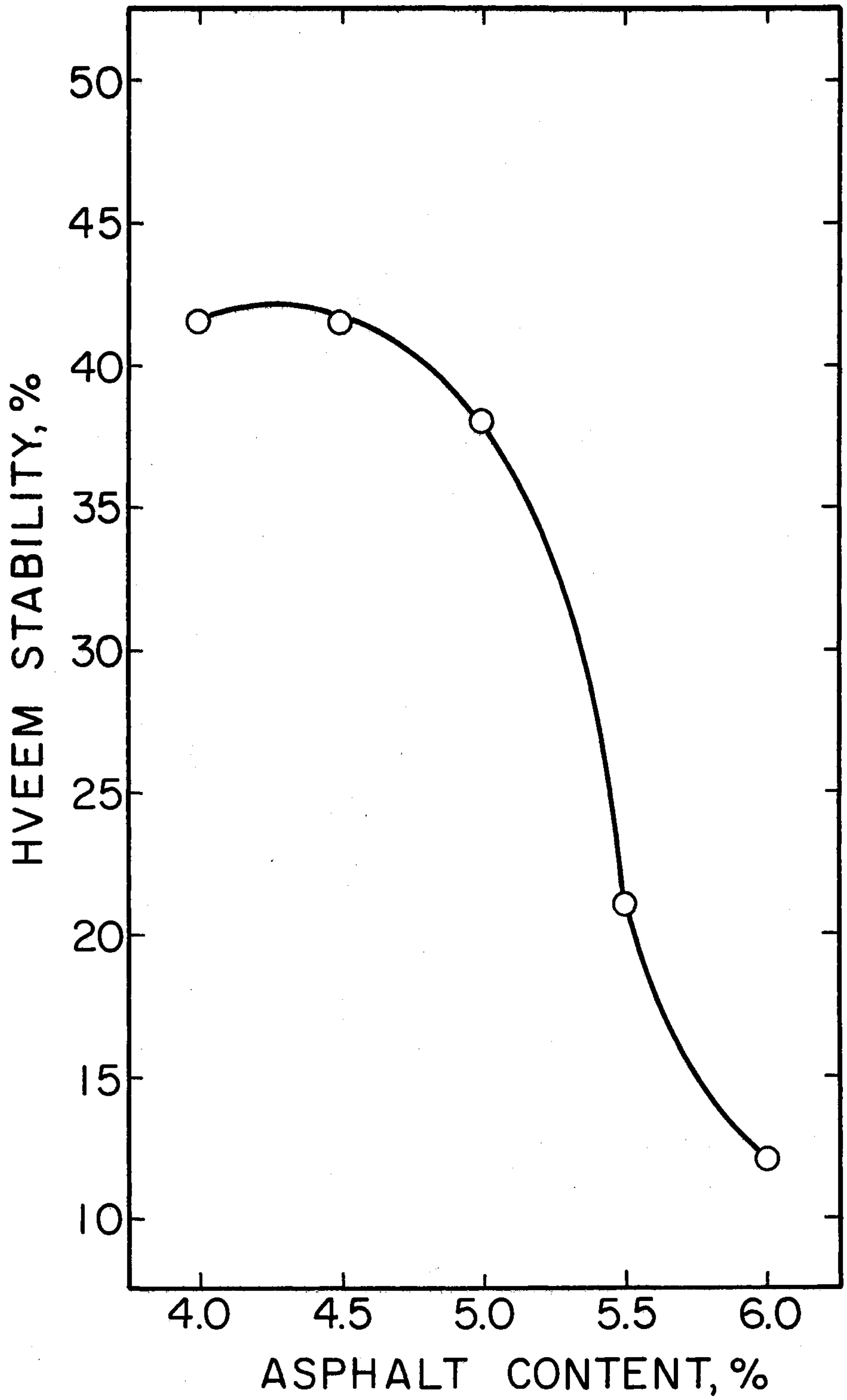

Figure 10. Hveem Stability Versus Asphalt Content
Cooperton Limestone

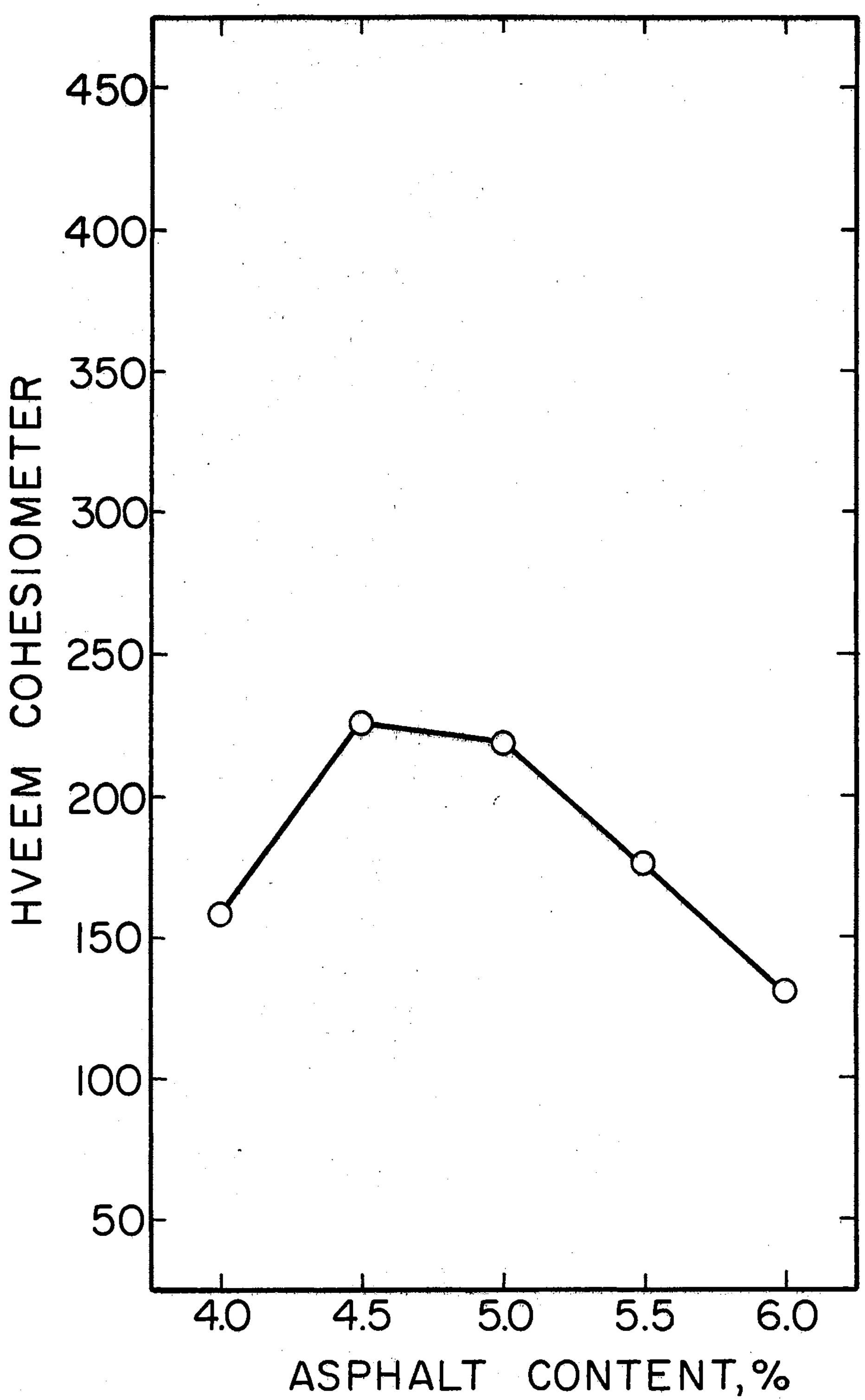

Figure 11. Hveem Cohesion Versus Asphalt Content
Cooperton Limestone

Again, Herrin and Goetz showed that the cohesion decreased when a dense-graded mix composed of natural sand fine aggregate was tested and compared to a mix composed entirely of crushed limestone (20).

A siliceous limestone from Stringtown was blended into the standard mix. Like Cooperton, Stringtown limestone is composed of crushed angular particles, the difference being that Stringtown contains more acid-insoluble residue. Results of the stabilometer test are shown in Figure 12 for the 20, 30, and 40% I.R. mixtures. The stability values at optimum asphalt content compared to the value for the standard mix; however, the three curves were slightly steeper indicating that the stabilities of these three blends were more susceptible to increases in the asphalt content.

Figure 13 gives the cohesiometer test results for the siliceous limestone mixes. Unlike the stability results, the plots of cohesion versus asphalt content for the three mixtures did not resemble the standard mix plot. The higher cohesiometer values corresponded to the higher asphalt contents indicating that more asphalt was required to adequately coat this hydrophilic aggregate to obtain maximum cohesion. However, the cohesiometer values at the optimum asphalt content were slightly less than the standard mix value.

Cherts

A fully-crushed chert from Miami was blended into the standard limestone mix. These chert particles exhibited sharp edges and very smooth, glassy faces. The results of the stabilometer test on the three blends are given in Figure 14. Adequate stability existed, possibly due

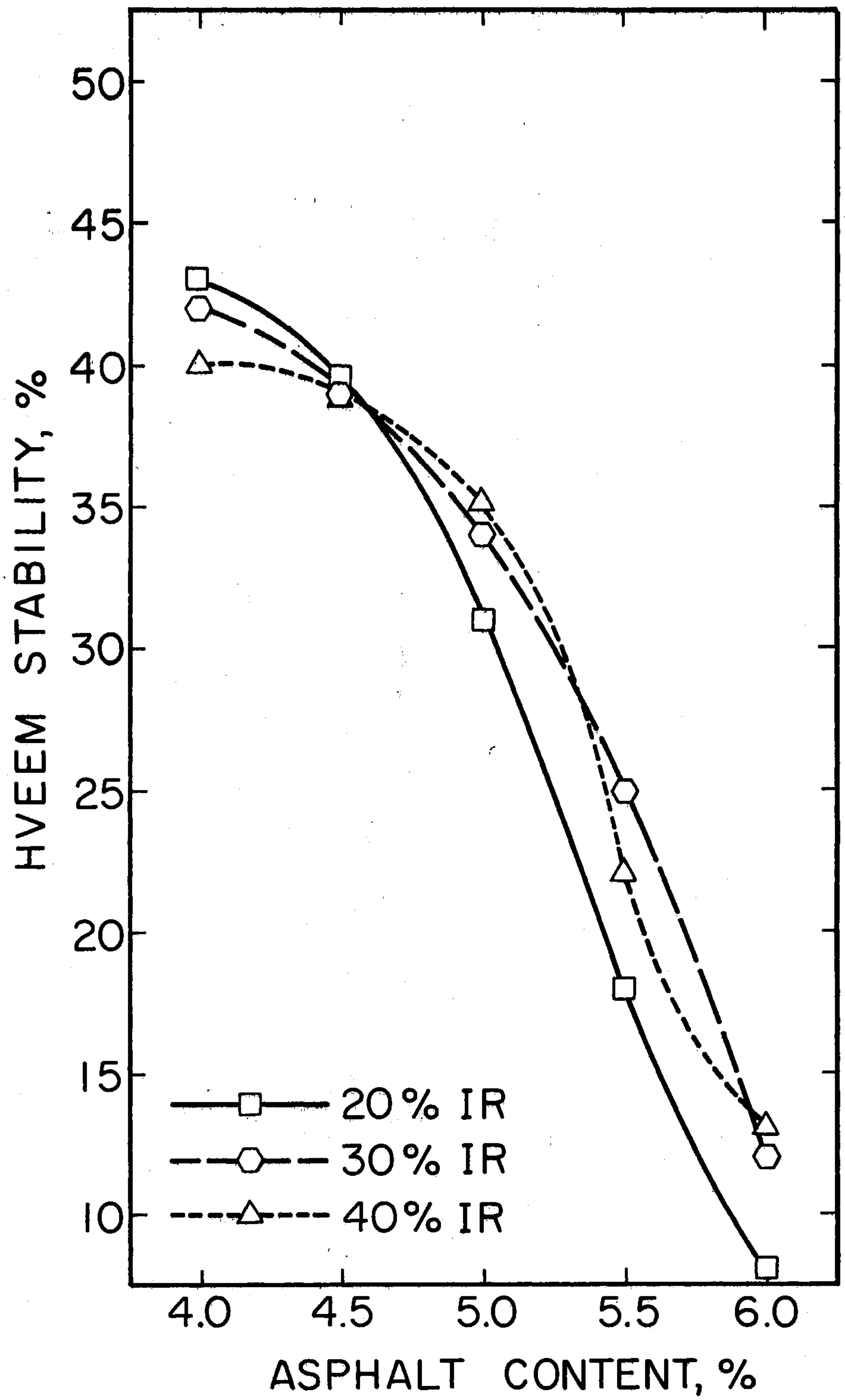

Figure 12. Hveem Stability Versus Asphalt Content
Stringtown Limestone

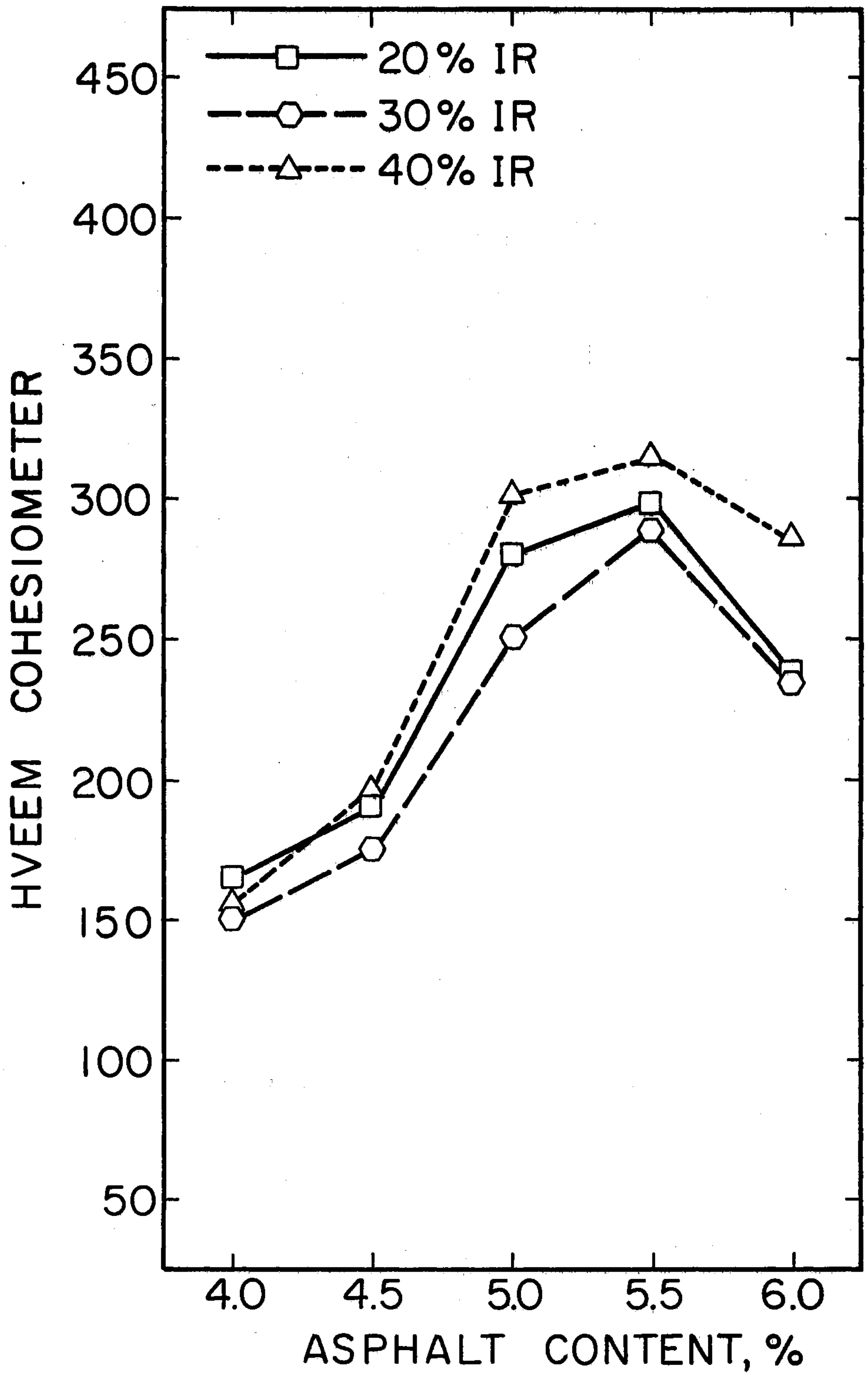

Figure 13. Hveem Cohesion Versus Asphalt Content
Stringtown Limestone

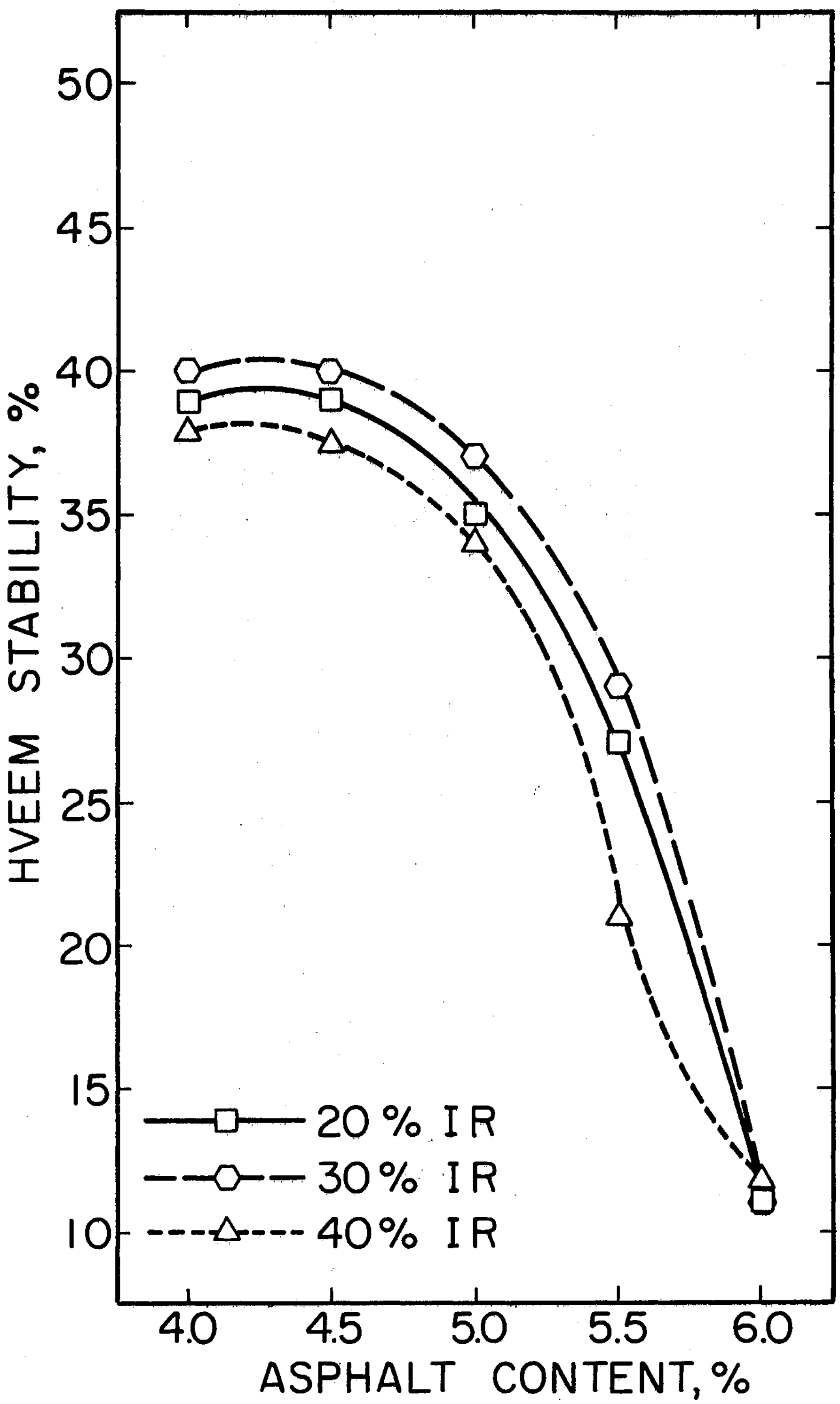

Figure 14. Hveem Stability Versus Asphalt Content
Miami Chert Gravel

to the sharp angularity of the chert which increased the interlocking of the particles enhancing the stability. However, the stabilometer values, at optimum asphalt content, were below the value for the standard mix. The stabilometer values at optimum for these crushed chert mixes were generally higher than those values for the partially crushed gravel mixes discussed later.

The cohesiometer values at optimum asphalt content were, in general, lower than the standard mix value. Figure 15 shows the cohesiometer test results. The asphalt possibly did not adhere as well with the hard glassy smooth faces of the chert particles. Also, the fact that chert is basically a hydrophilic aggregate could contribute to the lowering of the cohesion.

Sandstones

Three different sandstone aggregates were blended individually into the standard limestone mix. The Onapa and Cyril sandstones behaved similarly while the Keota material exhibited a different pattern. Onapa and Cyril stability values were high for asphalt contents from 4 to 5% as shown by Figures 16 and 17. The rough surface texture of the sandstone aggregate, indicating a high angle of internal friction, possibly accounted for the observed high stabilometer values. In fact, at optimum asphalt content, the three Onapa blends had stabilometer values comparable to that of the standard limestone mix.

The stability results for the Keota sandstone, given in Figure 18, indicated that these three blends were critical mixtures. Although Keota had the highest stabilometer values at optimum asphalt content

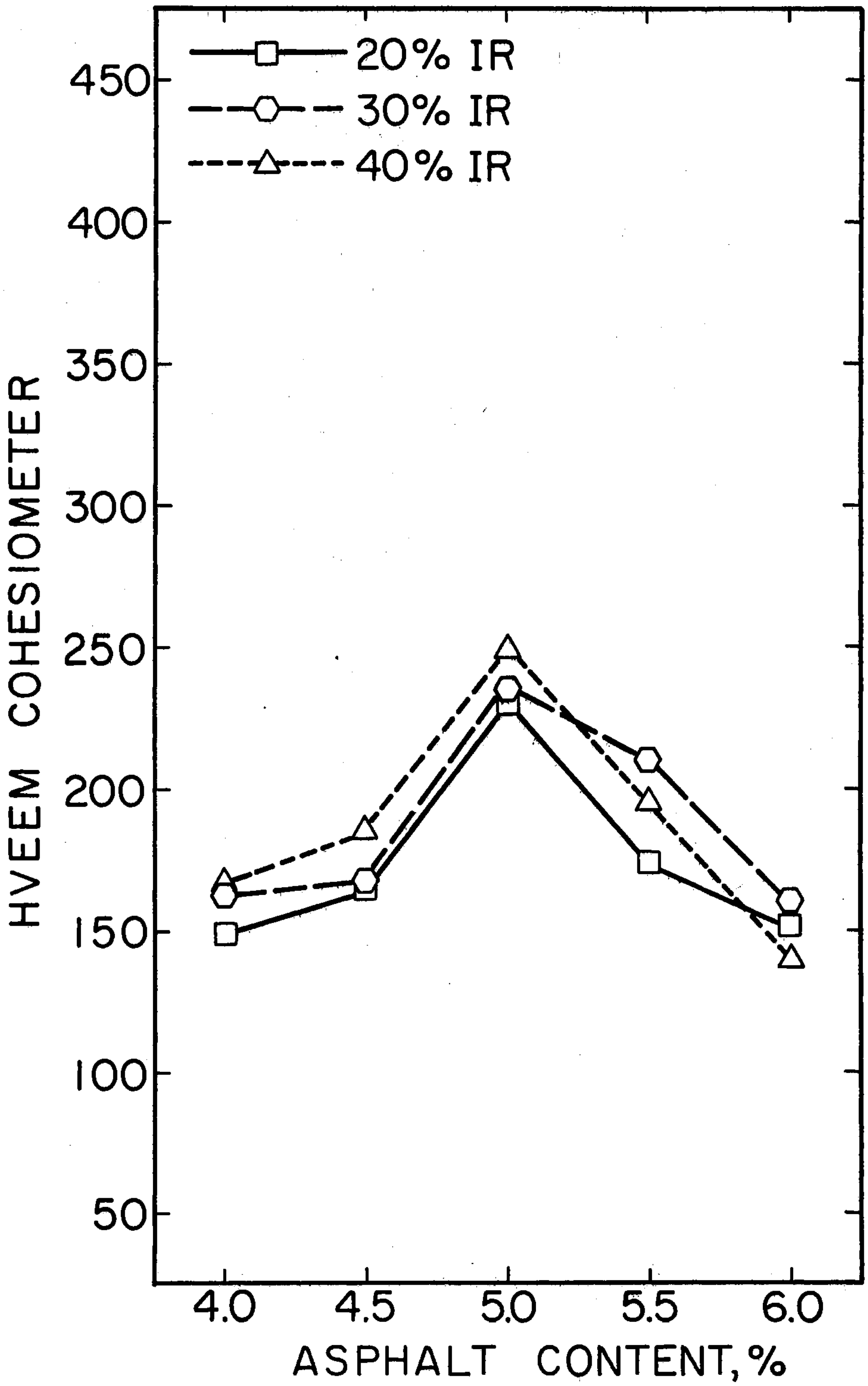

Figure 15. Hveem Cohesion Versus Asphalt Content
Miami Chert Gravel

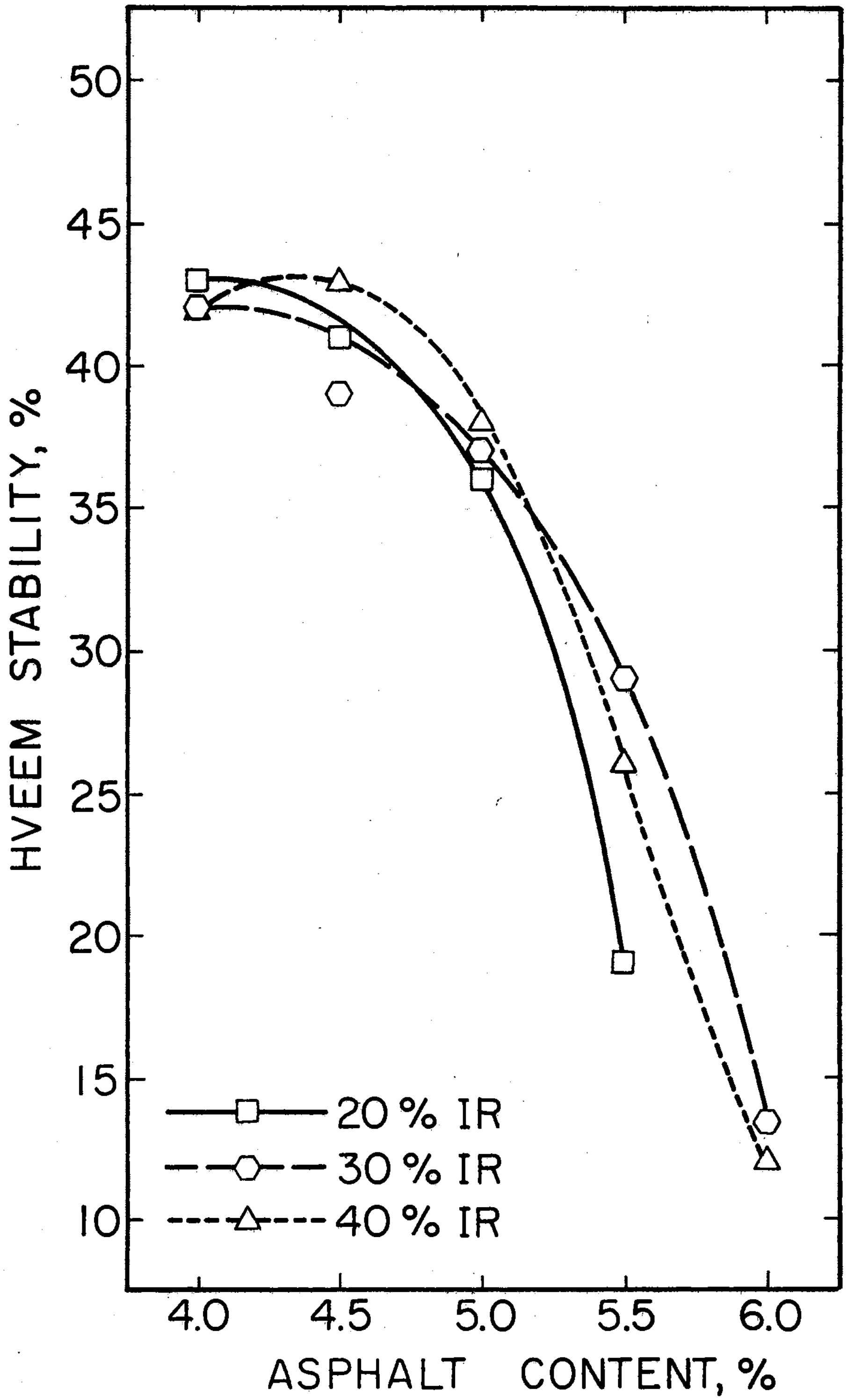

Figure 16. Hveem Stability Versus Asphalt Content
Onapa Sandstone

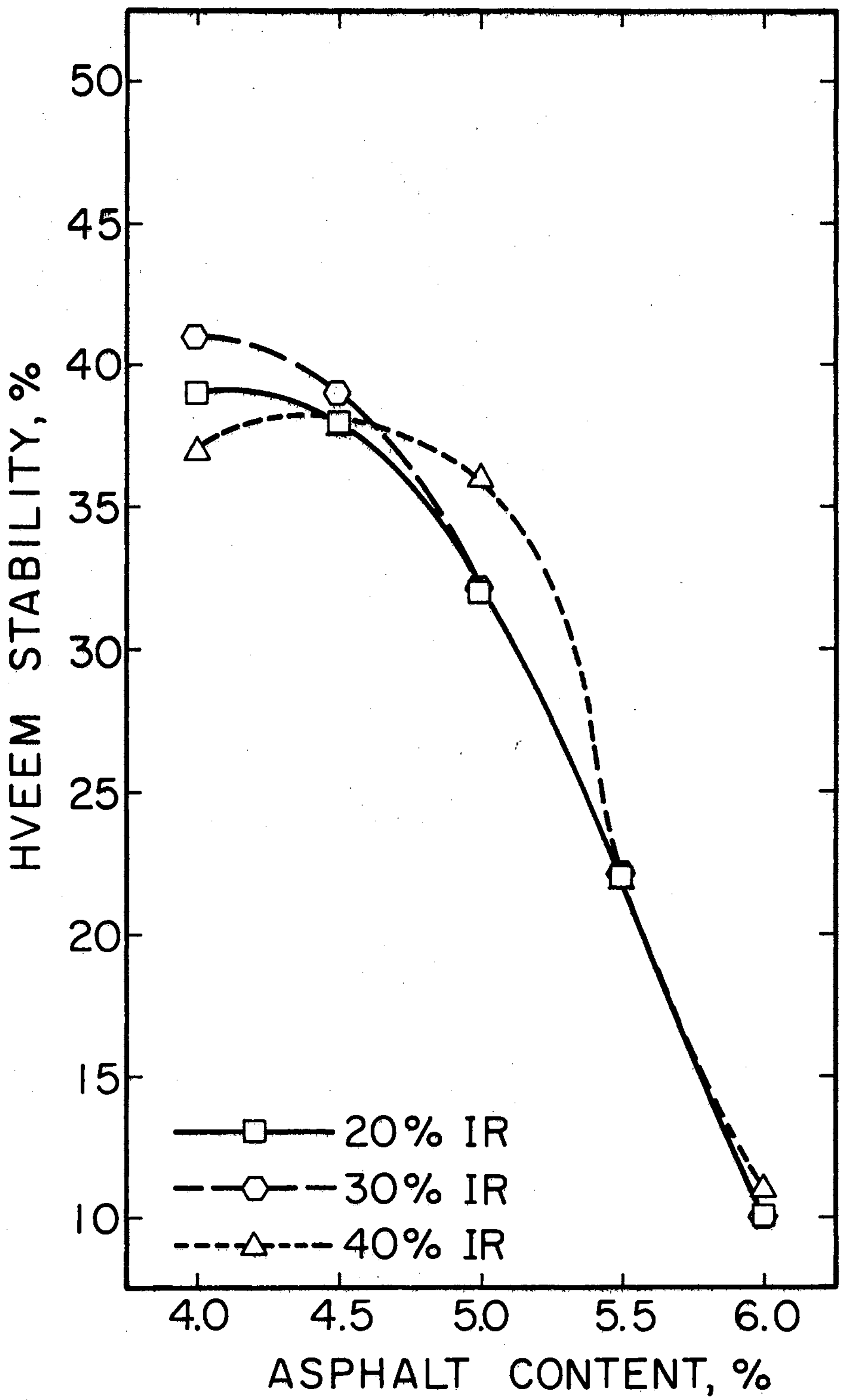

Figure 17. Hveem Stability Versus Asphalt Content
Cyril Sandstone

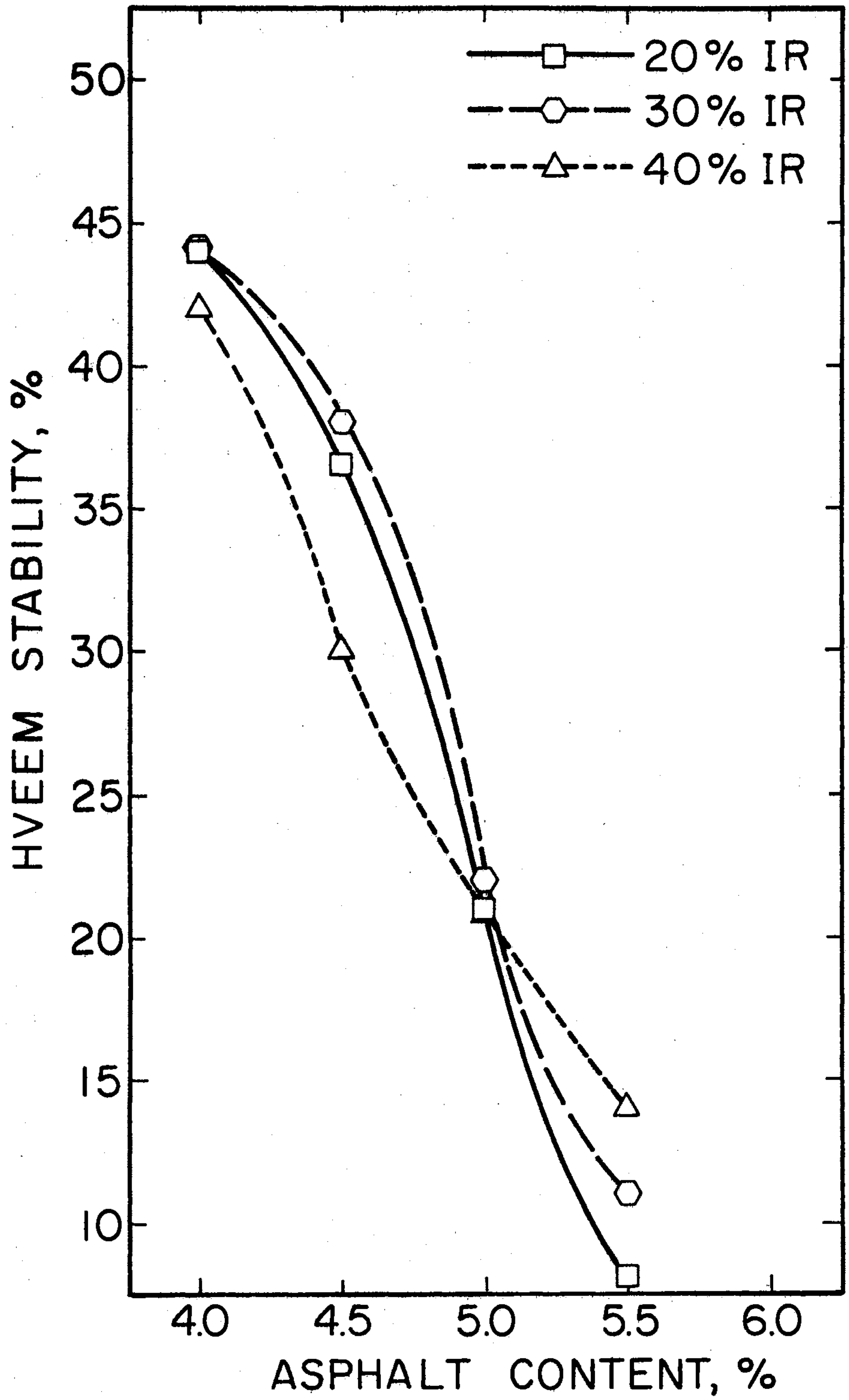

Figure 18. Hveem Stability Versus Asphalt Content Keota Sandstone

than all the other aggregate combinations, a 1% increase in asphalt lowered the stability drastically.

Cohesiometer results for the Onapa and Cyril sandstone blends were similar, whereas the Keota mixture again exhibited different trends. Figures 19 and 20 show the results for the three Onapa and three Cyril insoluble residue mixtures. In general, the maximum cohesiometer values occurred at the higher asphalt contents. Possibly, the more surface area available on the lighter weight sandstone effectively decreased the thickness of the asphalt film on the particles, therefore requiring more asphalt to develop maximum cohesion. Even though sandstone is classified as a hydrophilic aggregate, results showed that good adhesion of the asphalt to the sandstone particle existed.

However, for the three Keota blends, the maximum cohesiometer values occurred at lower asphalt contents. The results, as shown in Figure 21, again indicated that the Keota mixes were critical aggregate blends. Large differences in cohesion resulted with slight changes in asphalt content. The cohesiometer test was conducted on the Keota sandstone near the end of the study; therefore, the high cohesiometer values reflected the experience of the operator, not the inherent cohesion of the mix.

Gravels

Four different gravels were incorporated separately into the standard limestone mix. These gravels consisted of rounded particles that had been partially crushed, in other words, each particle had one or more freshly crushed or fractured faces. Results of tests determining the percent by weight of fractured particles, given in Chapter VI, showed

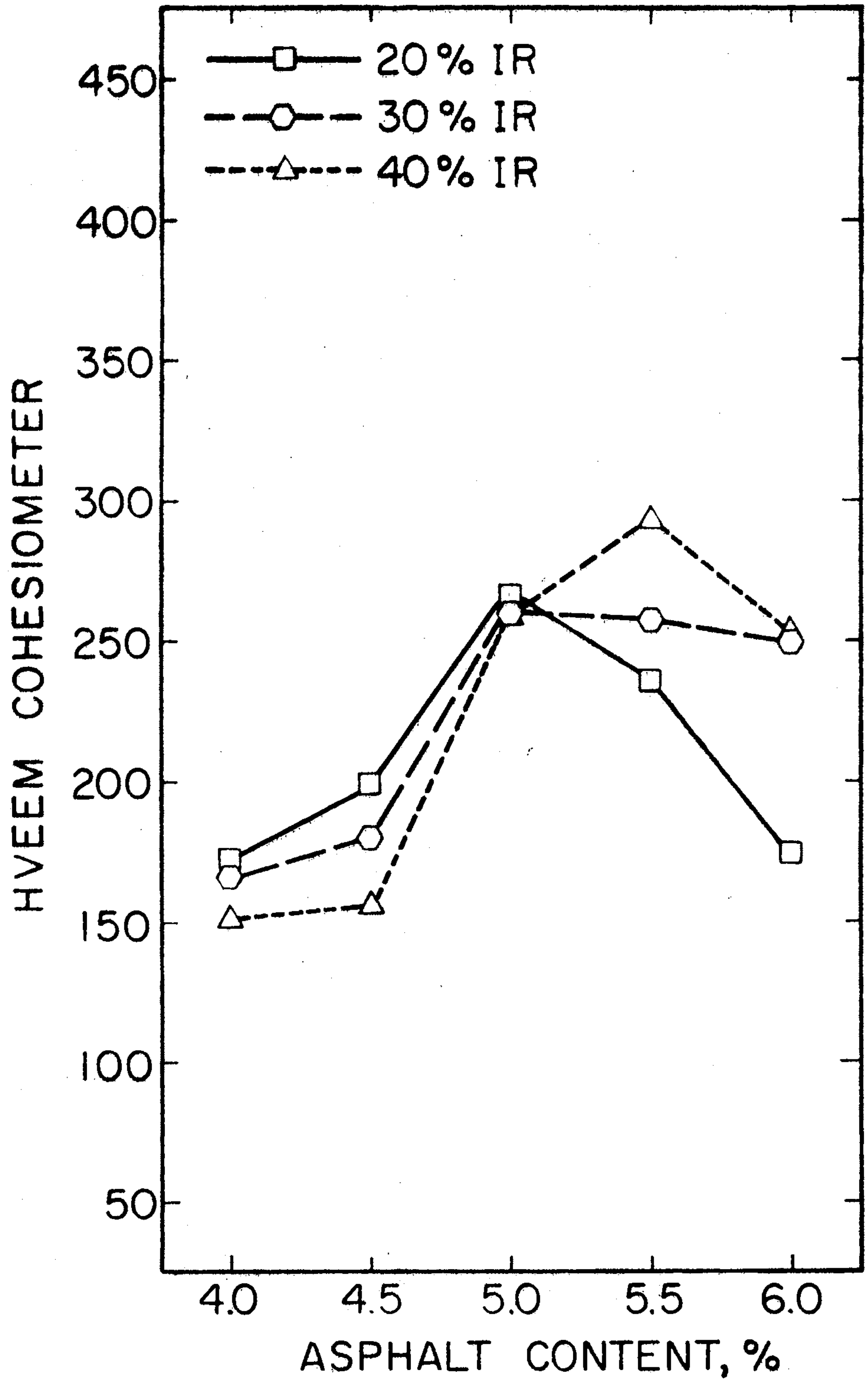

Figure 19. Hveem Cohesion Versus Asphalt Content
Onapa Sandstone

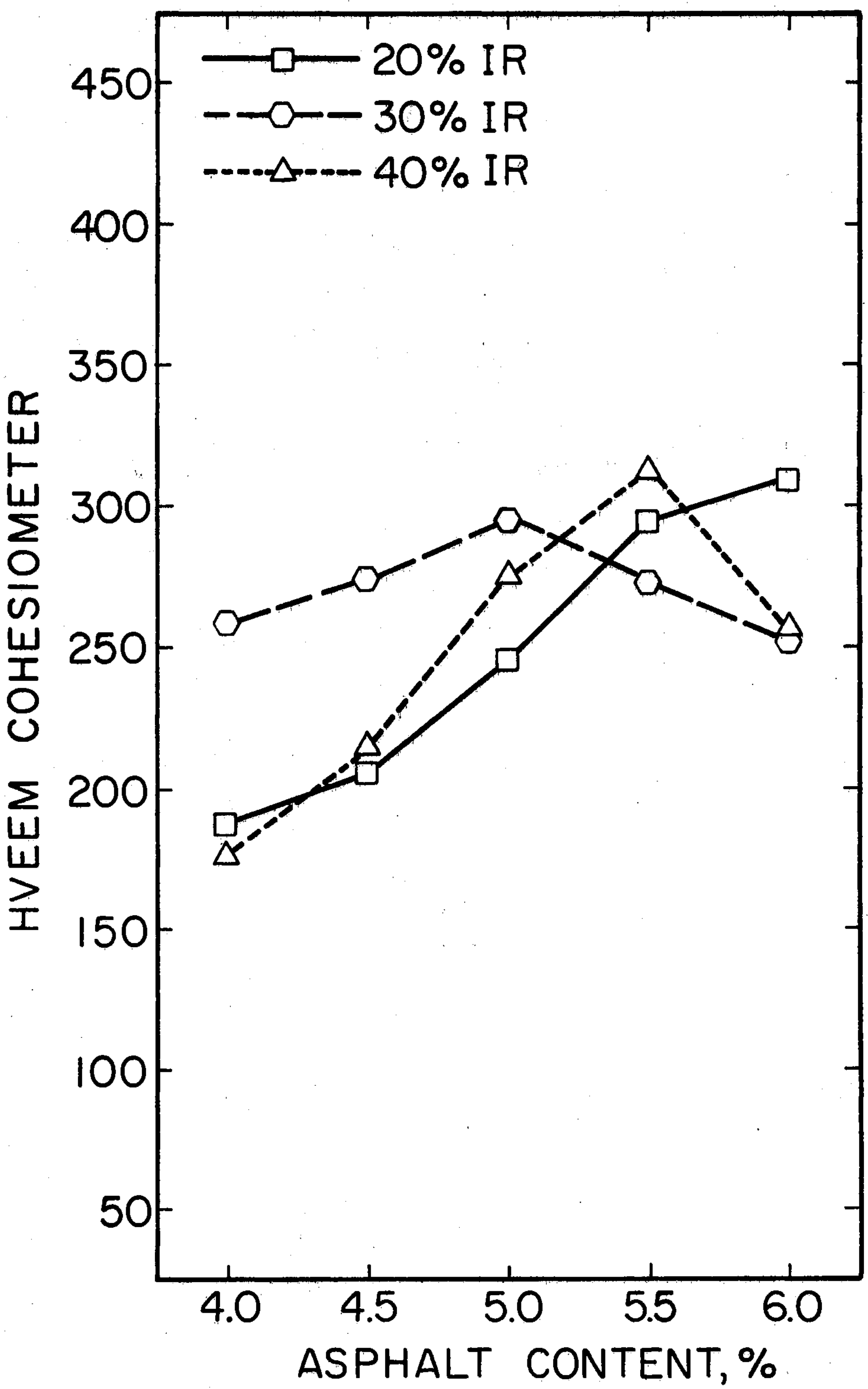

Figure 20. Hveem Cohesion Versus Asphalt Content
Cyril Sandstone

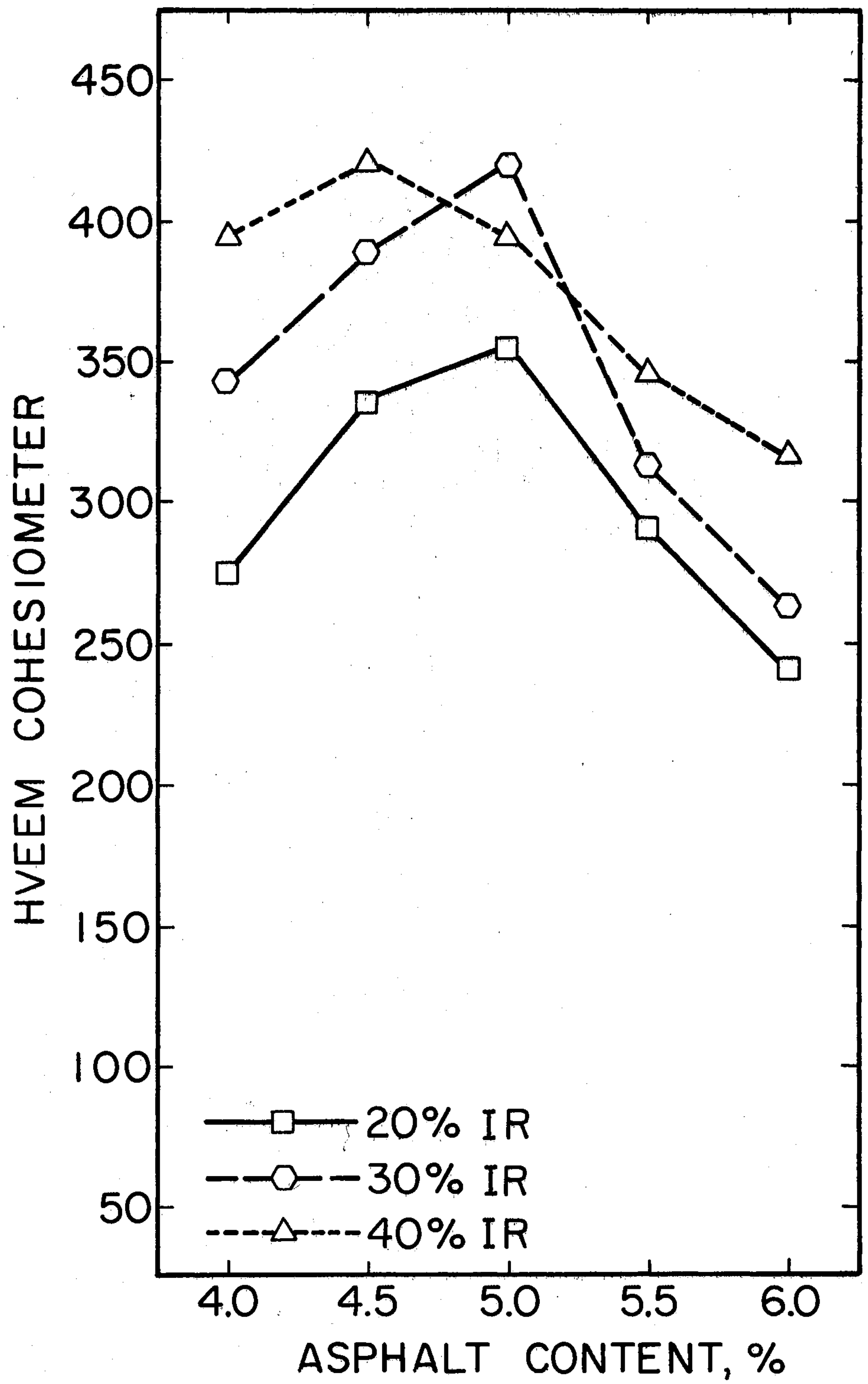

Figure 21. Hveem Cohesion Versus Asphalt Content
Keota Sandstone

that all four aggregate samples had sufficient fractured faces to pass the minimum requirement specified by the Oklahoma Highway Department (16).

The Asher chert gravel mixes exhibited stability results that were slightly different from the three other gravels. As seen in Figure 22, the stabilometer values remained above minimum (35) at an asphalt content of 4 1/2%. It was noted that the Asher particles from the quarry sample were coated with a red hematite dust. Only washing of the particles during the specific gravity tests removed this dust. Possibly, this hematite dust hardened the asphalt which enhanced the stability of the mix.

Figures 23, 24, and 25 show the results of the stabilometer tests conducted on the mixes that incorporated individually Broken Bow gravel, Gore gravel, and Hugo gravel into the standard limestone mix. Each of these nine aggregate blends showed stability results that are typical of critical aggregate mixtures. With the addition of only 1/2% asphalt, the stabilometer values dropped by as much as 10 in some cases. However, at the optimum asphalt contents, stabilometer values were above the minimum value of 35. Possibly, at optimum asphalt content, compaction effort placed the particles in good contact with each other creating adequate stability. When the asphalt content was increased slightly, the particle contact was lost due to the presence of excess asphalt and the stability decreased drastically. Low voids in the mineral aggregate (VMA) possibly caused these critical mix tendencies.

Like the stability test results, the Asher chert gravel mixes showed different trends in the cohesiometer test results than did the

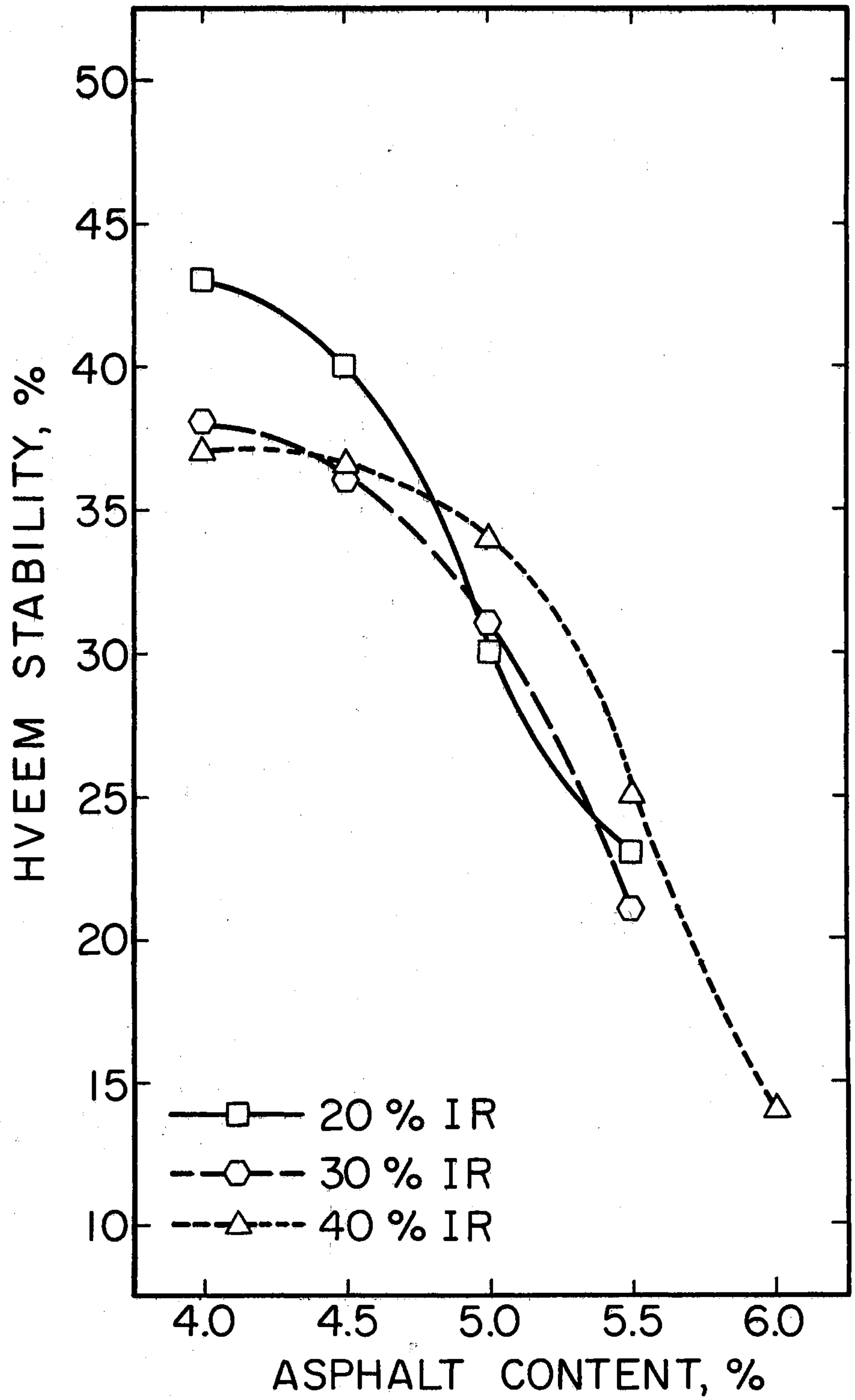

Figure 22. Hveem Stability Versus Asphalt Content
Asher Chert Gravel

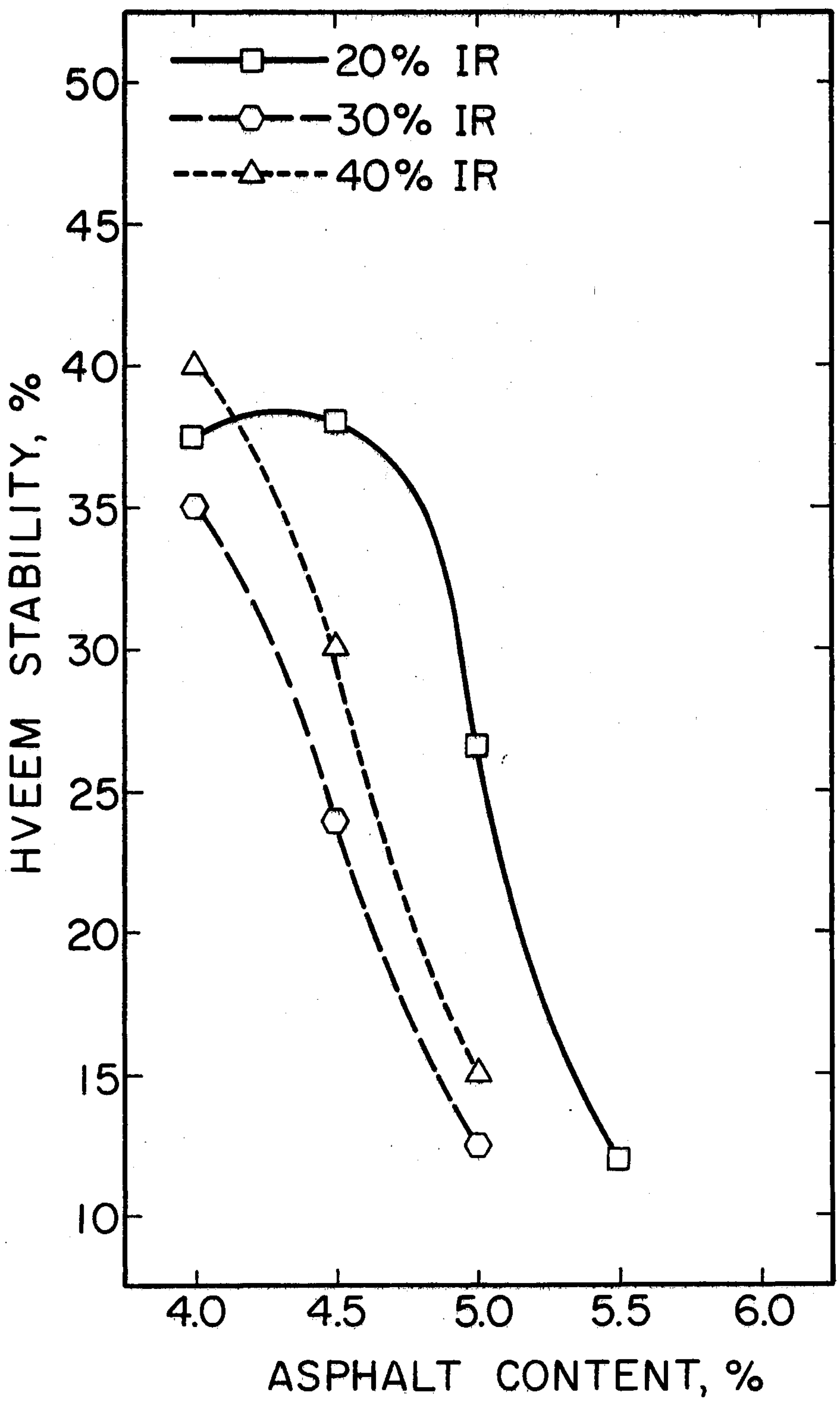

Figure 23. Hveem Stability Versus Asphalt Content
Broken Bow Chert Gravel

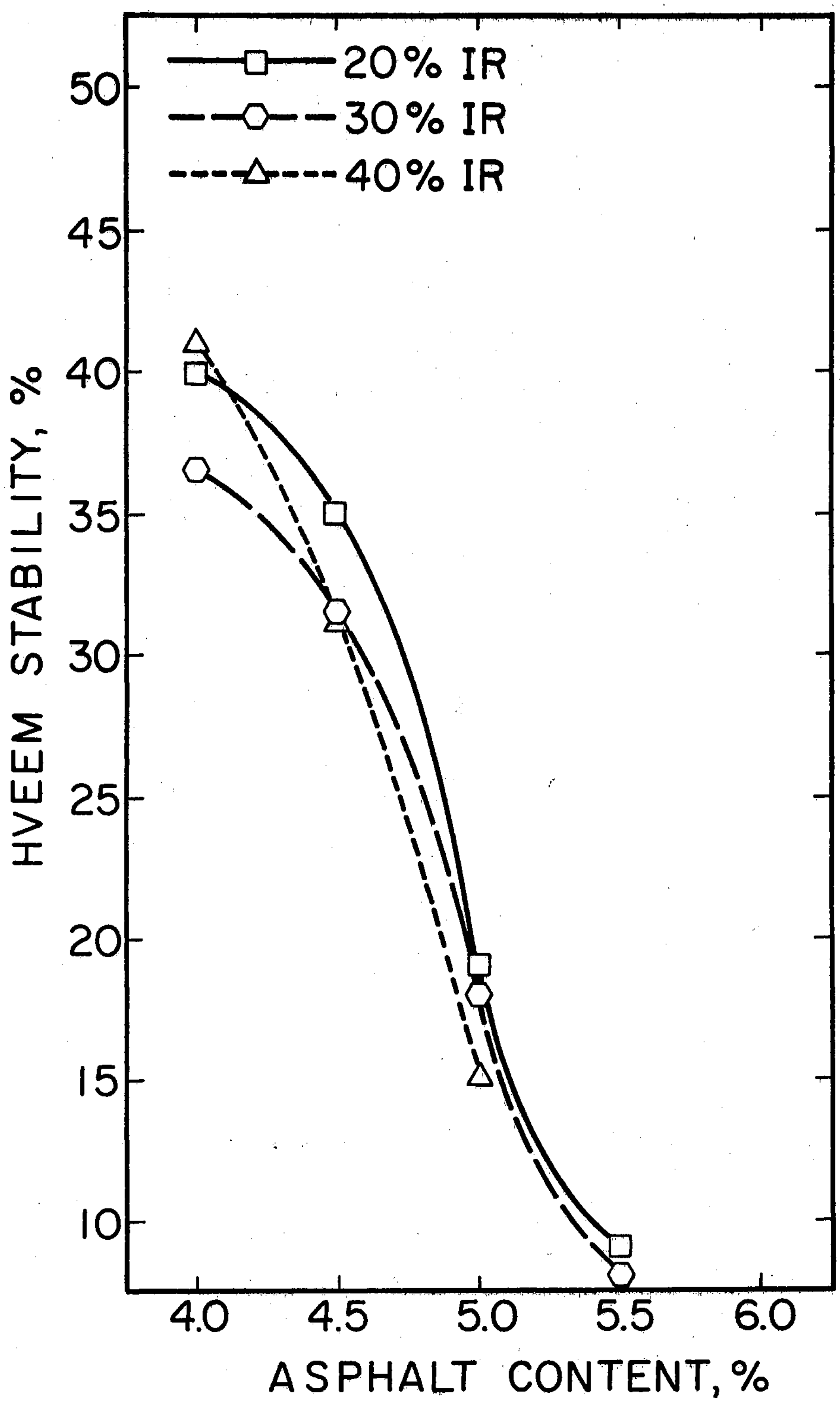

Figure 24. Hveem Stability Versus Asphalt Content
Gore Gravel

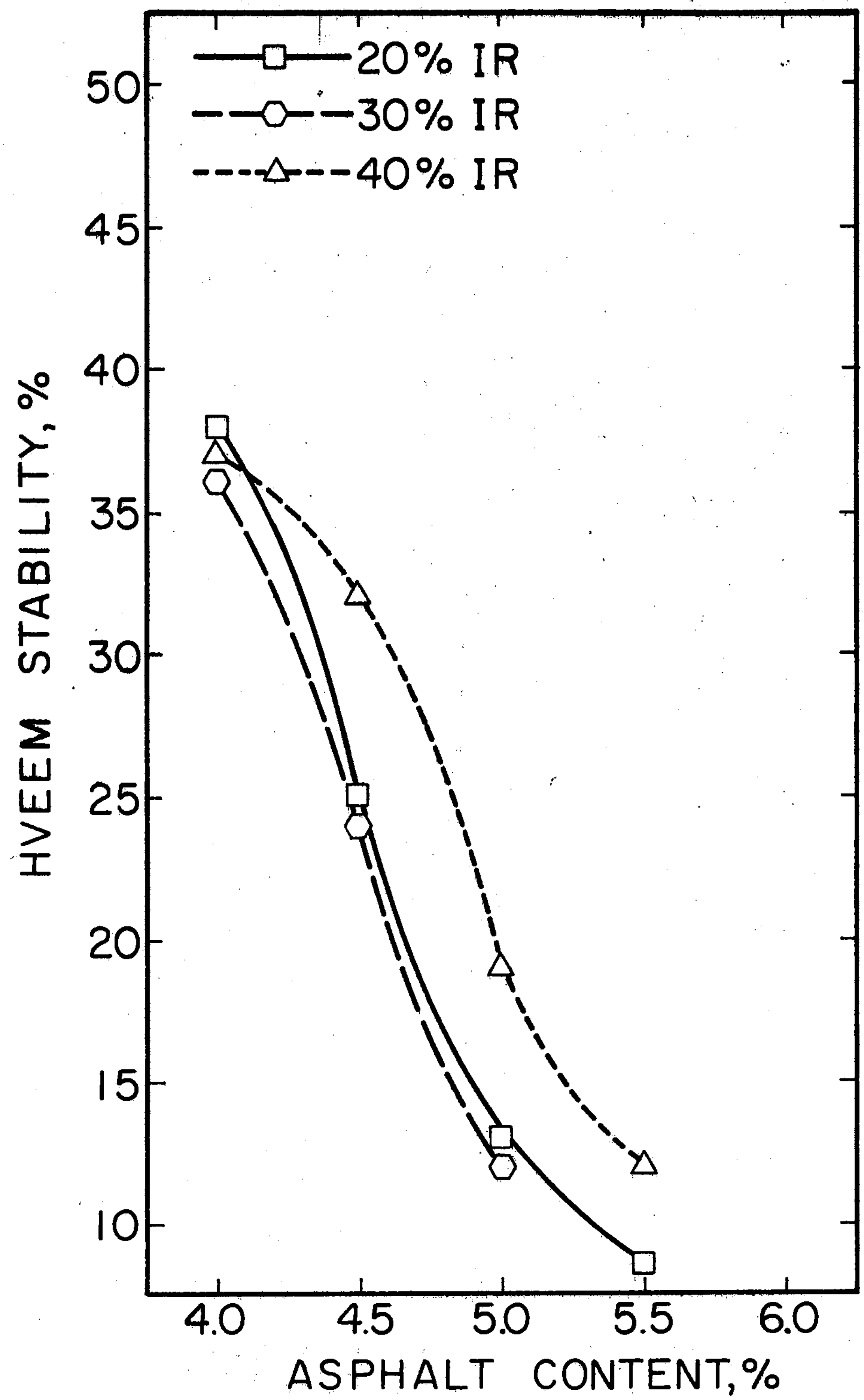

Figure 25. Hveem Stability Versus Asphalt Content
Hugo Chert Gravel

other three gravel aggregates. Figure 26 shows that the maximum cohesiometer values occurred at the higher asphalt contents for the Asher gravel. The hematite dust on the gravel, acting in the same manner as the limestone dust, required the use of more asphalt to adequately coat the particles for obtaining maximum cohesion.

Results of the cohesiometer test on the nine mixes involving Broken Bow gravel, Gore gravel, and Hugo gravel are shown in Figures 27, 28, and 29. In general, the maximum cohesion of these blends occurred at the lower asphalt contents. With addition of asphalt, the cohesiometer values rapidly decreased in magnitude. These three aggregates were tested towards the end of the study; therefore, the higher cohesiometer values indicated operator variation more so than an actual increase in the cohesion of the mixes.

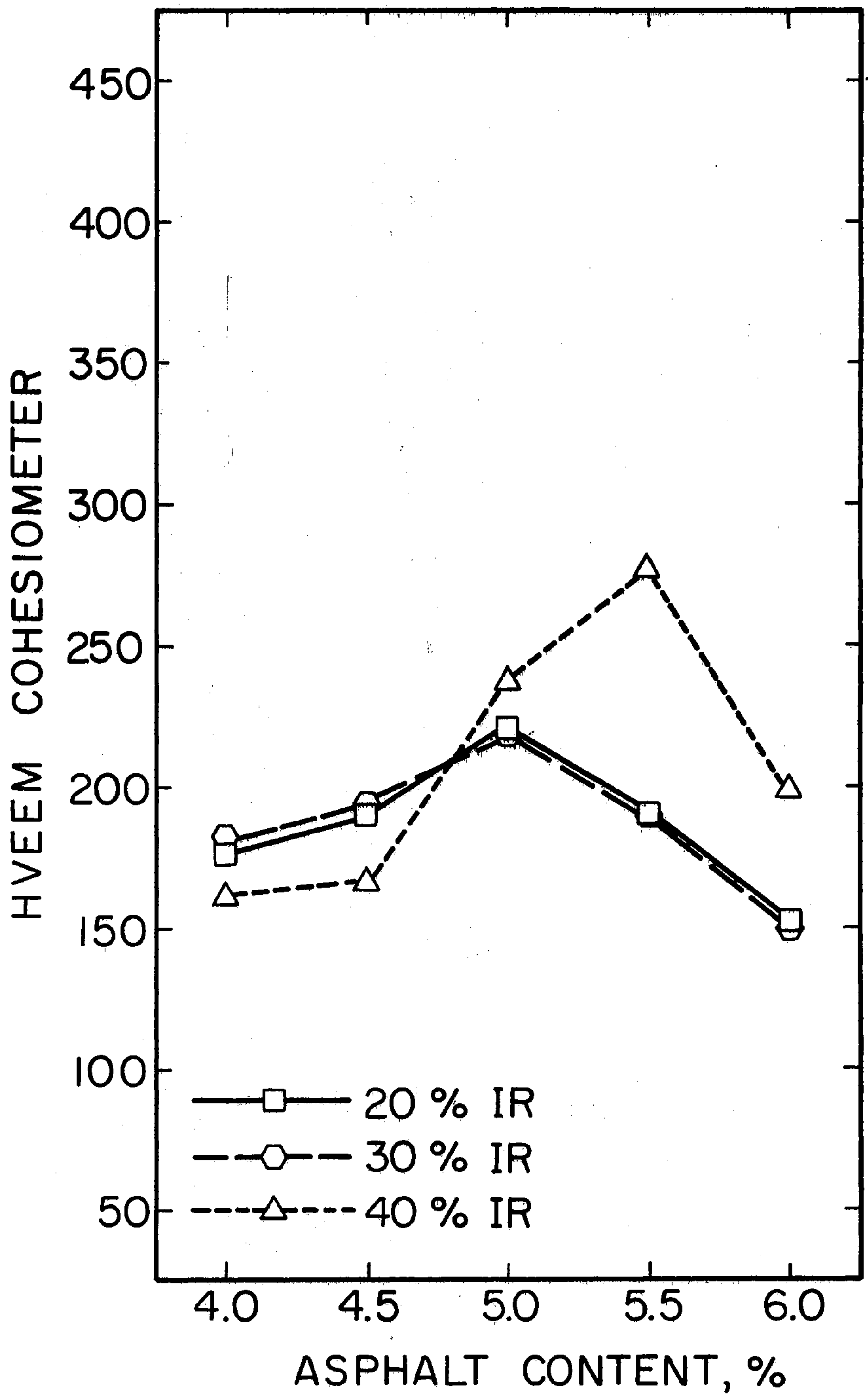

Figure 26. Hveem Cohesion Versus Asphalt Content
Asher Chert Gravel

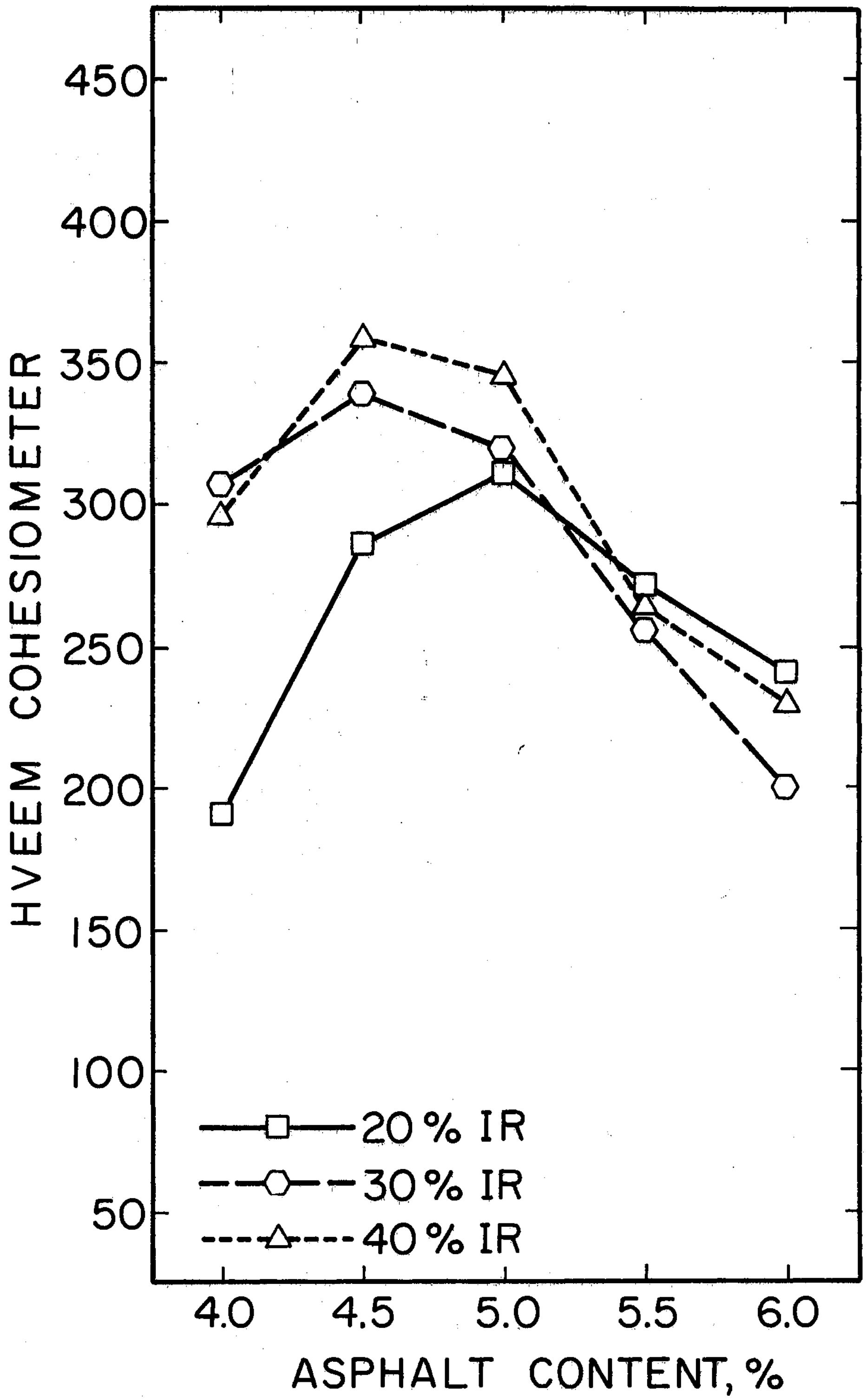

Figure 27. Hveem Cohesion Versus Asphalt Content
Broken Bow Chert Gravel

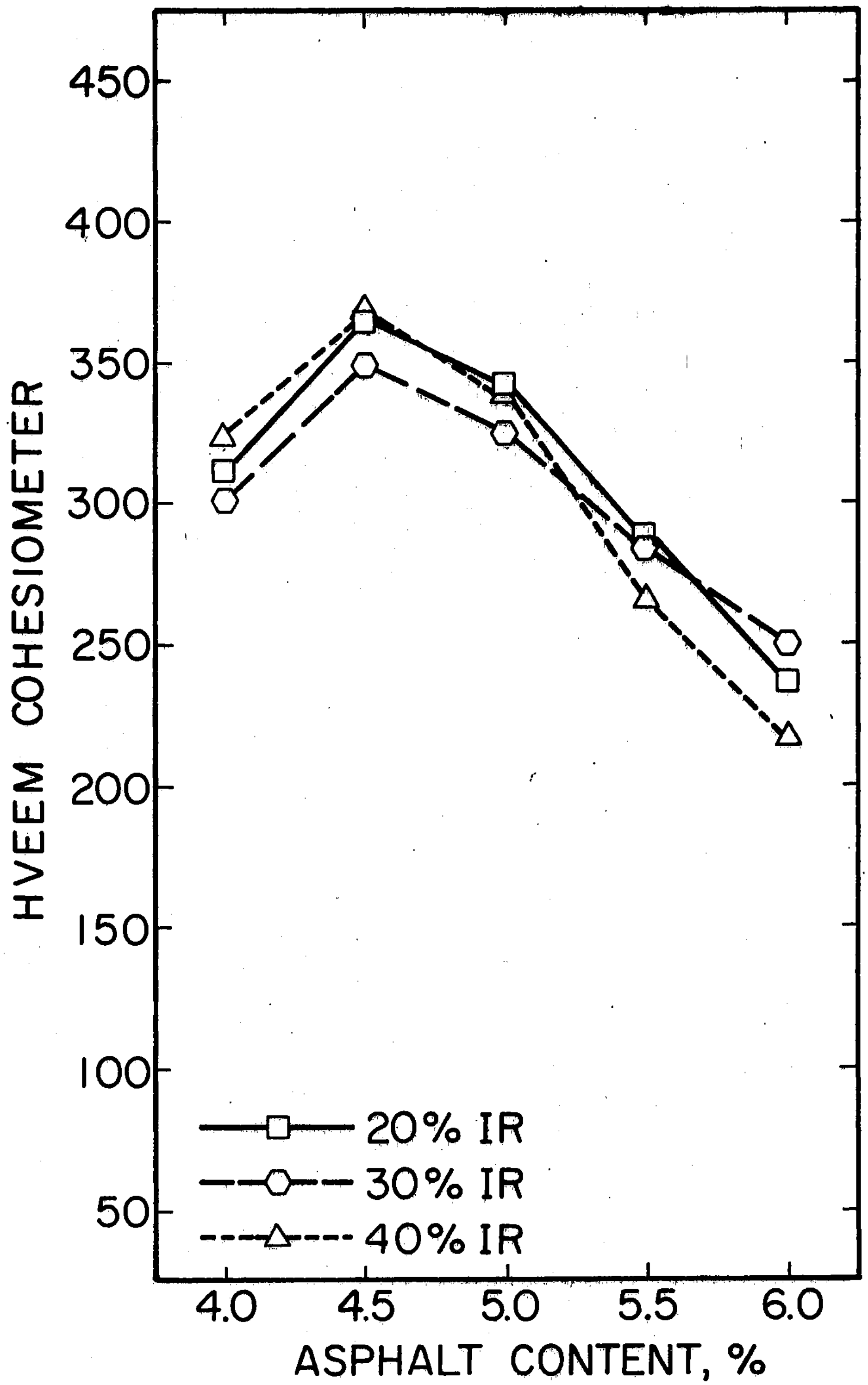

Figure 28. Hveem Cohesion Versus Asphalt Content
Gore Gravel

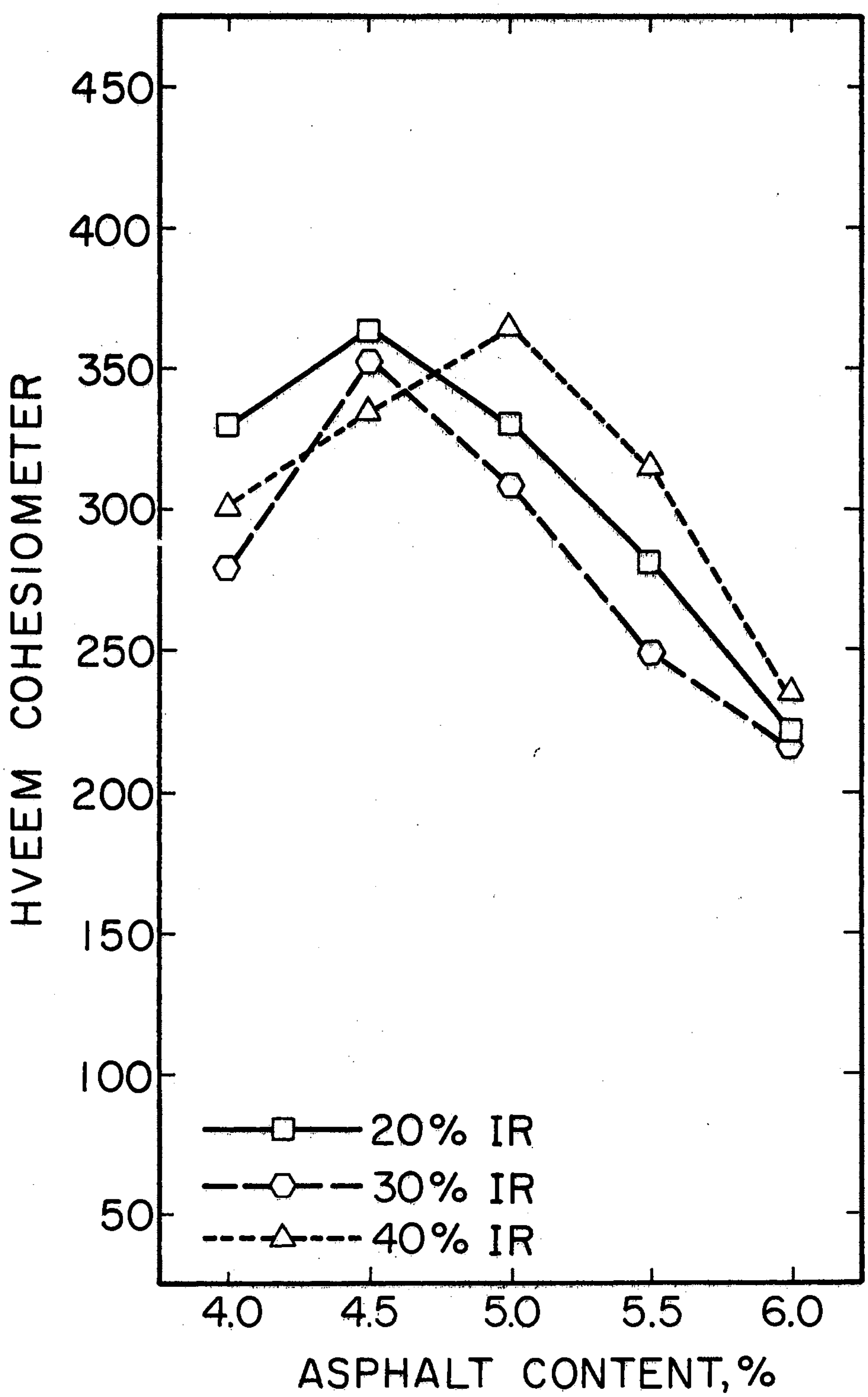

Figure 29. Hveem Cohesion Versus Asphalt Content
Hugo Chert Gravel

CHAPTER VI

ASSOCIATED LABORATORY TESTS AND RESULTS

Fractured Faces

Internal friction in an aggregate is that property which resists the movement of the particles past one another under the action of an imposed load. This resistance is formed by the interlocking of the particles and the surface friction between adjacent particles. A round, smooth, uncrushed gravel is comparatively low in internal friction because particle interlocking is impossible and the surface friction is low. The stability value obtained in testing asphalt mixes is largely a reflection of the internal friction of the mineral aggregates, more so than the cementing action of the asphalt cement. If such a mix consists of smooth rounded particles with a given asphalt binder, its stability is considerably reduced. However, if a rounded gravel aggregate is crushed, particles with one or more crushed or fractured faces are produced. The aggregate then becomes more desirable for use in an asphalt mix primarily because of its higher internal friction.

Several recent research studies (8, 9) have concluded that by incorporating crushed siliceous aggregate in the coarse fraction of a surface course paving mixture, resistance to polishing and thus skid resistance of the pavement is increased substantially. The Oklahoma Highway Department specifies that for hot-mix hot-laid asphalt concrete surface mixtures, at least 50%, by weight of the aggregate retained on the #4 sieve shall be composed of particles having one or more fractured faces.

Since the four gravel aggregates used in this study had not been produced specifically for use in surface course mixtures, the Oklahoma Highway Department's method for determining the percentage of crushed particles, test method OHD-L-18 (1), was slightly modified. The aggregate was sieved into three sieves, 3/4" - 1/2", 1/2" - 3/8", and 3/8" - #4. Each size sample was reduced to approximately 500 gram quantities using a mechanical splitter. Duplicate 500 gram amounts in each of the three sizes were prepared so that two operators could conduct the test. Each particle was examined by hand for a crushed or fractured face, separated into pans, and the percentage of crushed particles was determined by weight for each sieve size. The three size percentages were then averaged. A weighted average percent fractured faces was calculated for each aggregate based on the combination of each sieve size according to the mix design batch weights. Also, a total mix weighted average was calculated for each of the three percents insoluble residue combinations of the four gravel aggregates. These average values are shown in Table VII. The Cooperton limestone aggregate in these blends was assumed to have 100% fractured faces. These tabulated values indicate that the coarse fractions of the gravel aggregate blends had fractured face percentages greater than 90%.

Bulk Impregnated Specific Gravity

The use of a proper specific gravity of aggregate is of paramount importance in the design of bituminous mixtures. In order to obtain a

TABLE VII

PERCENT FRACTURED FACES

Aggregate	Aggregate Average	Weighted Average[1]	Total Mix Weighted Average[1] 20%	30%	40%
Asher Chert Gravel	50.3	53.9	91.3	86.7	82.1
Broken Bow Chert Gravel	64.7	62.0	92.6	88.7	84.8
Hugo Chert Gravel	51.1	55.1	91.4	86.8	82.2
Gore Gravel	68.5	66.8	93.6	90.1	86.7

[1]According to project Mix Design.

true comparison between the theoretical density and the actual density of a bituminous pavement, the specific gravity of the aggregate blend must be accurately determined. Two conventional specific gravities have been used by various agencies and highway departments. These are the bulk specific gravity and the apparent specific gravity. Depending upon the water absorption of the aggregate, the proper specific gravity ranges between the bulk and the apparent specific gravity. Because aggregates absorb bitumen to a variable extent, the two conventional specific gravities have proven unsatisfactory for general use with porous aggregates.

Ricketts et al. (22) conducted an evaluation of the two conventional specific gravities for non-porous to very porous aggregates. From this study evolved the concept of bulk impregnated specific gravity, which is a function of the ratio of bitumen absorption to water absorption of an aggregate. Bulk impregnated specific gravity, SG_{bi}, is defined as "the ratio of the weight, A, in air of a given volume of a permeable aggregate (including solids, impermeable pores, and pores normally permeable to water but which are variable permeable to bitumen) at a stated temperature to: the weight in air of an equal volume, V_t, of distilled water at a stated temperature minus the weight of the volume, V_b, of bitumen absorbed by pores which are permeable to it," or:

$$SG_{bi} = \frac{A}{V_t - V_b} \quad .$$

Theoretically, when an aggregate absorbs no bitumen, its bulk impregnated specific gravity equals conventional bulk specific gravity. Conversely, if absorbed bitumen equals water absorption, its bulk impregnated specific gravity equals conventional apparent specific gravity. If the permeable voids are partially filled with asphalt, the bulk impregnated specific gravity will be somewhere between bulk and apparent. For example, an aggregate that is unable to absorb any asphalt will have a bitumen to water absorption ratio equal to zero regardless of the aggregate's water absorption. However, if an aggregate has a bitumen absorption equal to its water absorption, the ratio of the two absorptions will equal one. Results of tests conducted by Ricketts et al., as illustrated in Figure 30, showed that the value of the bulk impregnated specific gravity varied linearly between the bulk and the apparent specific gravity as the ratio of the bitumen to water absorption increased.

In general, the test procedure outlined by Ricketts et al. is the same as that used by the Corps of Engineers (23) and the procedure outlined by the Oklahoma Highway Department, test method OHD-L-7 (1). The latter procedure was used in this study. A representative 1000 gram sample of aggregate was weighed into a pan. In this case, the sample was a prototype of the aggregate blends based upon mix design batch weights. Approximately 1000 grams of asphalt were poured into a large can equipped with a wire handle. A long sheet metal strip, 1" wide, was inserted into the asphalt to facilitate stirring. The aggregate and asphalt were heated to 260 F ± 5 F for at least four hours. The aggregate was then slowly poured into the asphalt while stirring with the metal strip until all the entrapped air was removed. The cans of asphalt and aggregate were then allowed to cool to room temperature.

The following relation was used to calculate the bulk impregnated specific gravity:

$$SG_{bi} = \frac{A}{(D - E) - (B - C)}$$

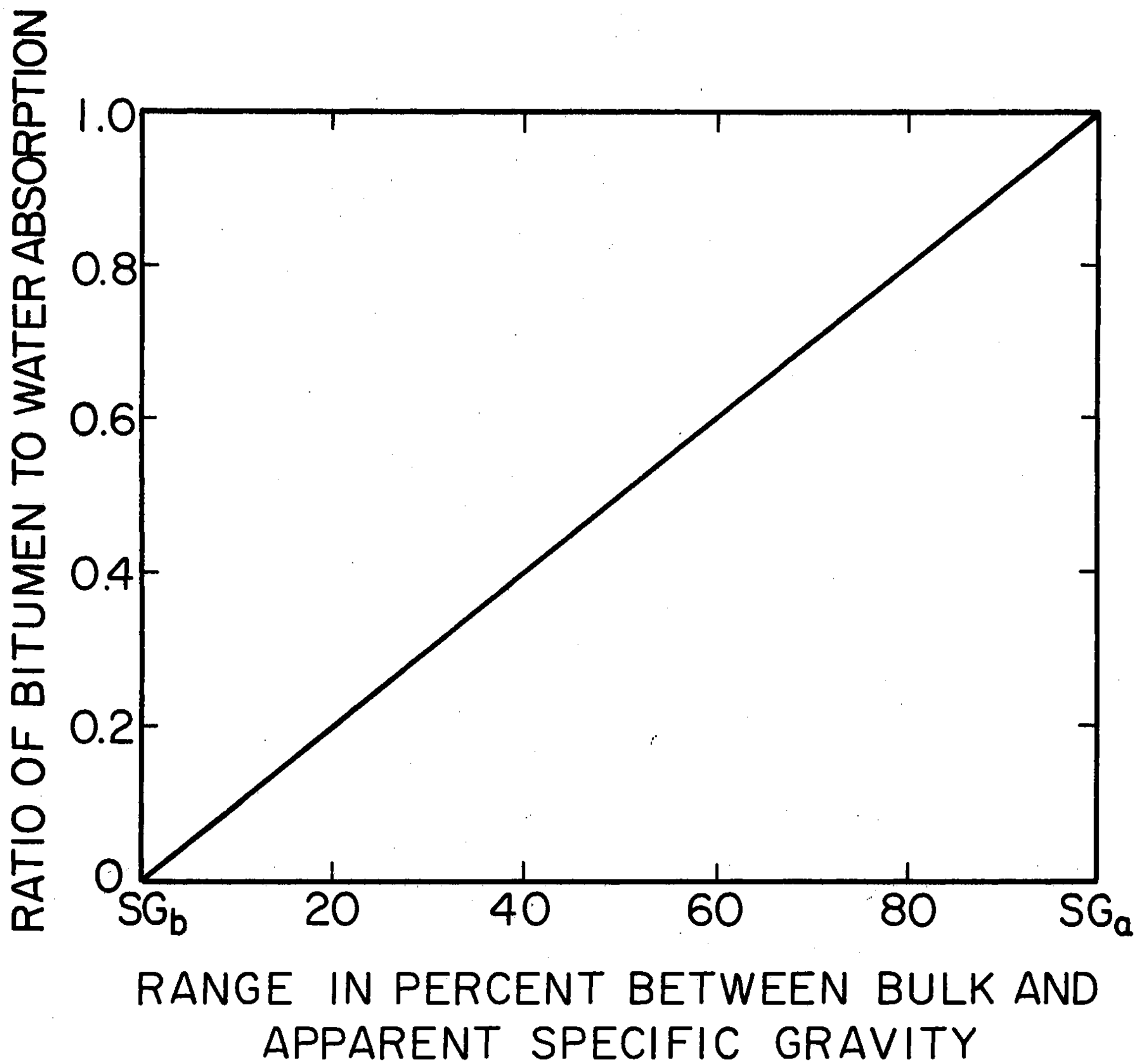

Figure 30. Interrelationship Between Ratio of Bitumen to Water Absorption and Specific Gravity Range (After Ricketts, et al., 22)

where: SG_{bi} = bulk impregnated specific gravity

A = weight of oven-dry aggregate

B = weight of can + stirrer + asphalt in air

C = weight of can + stirrer + asphalt in water

D = weight of can + stirrer + asphalt + aggregate in air

E = weight of can + stirrer + asphalt + aggregate in water.

Table VIII shows the bulk impregnated specific gravities of the blended aggregates used in this study.

Calculated Specific Gravity of Blended Aggregate

The bulk specific gravity (saturated surface dry basis--SSD) of each aggregate was determined as outlined in Chapter V. An average bulk specific gravity (SSD) of the blended aggregate was computed using the percent by weight of the respective aggregates in a given mix. The following relation was used to determine the calculated average bulk specific gravity of the blended aggregate:

$$SG_{ca} = \frac{100}{P_L/G_L + P_S/G_S + P_A/G_A}$$

where: SG_{ca} = calculated bulk specific gravity of the blended aggregate

G_L = average bulk specific gravity (SSD) of the combined sizes of Cooperton Limestone

G_S = average bulk specific gravity (SSD) of the combined sizes of Arkhola sand

G_A = average bulk specific gravity (SSD) of the combined sizes of the siliceous aggregate

P_L = percent by weight of Cooperton limestone

P_S = percent by weight of Arkhola sand

P_A = percent by weight of the siliceous aggregate.

TABLE VIII

BULK IMPREGNATED SPECIFIC GRAVITY FOR BLENDED AGGREGATE MIXTURES

Aggregate	Bulk Impregnated Specific Gravity		
Cooperton Limestone (Standard Mix)	2.69		
Standard Mix Plus Siliceous Aggregate	Acid-Insoluble Residue 20%	30%	40%
Asher Chert Gravel	2.63	2.61	2.59
Miami Chert	2.65	2.61	2.59
Onapa Sandstone	2.58	2.58	2.53
Stringtown Limestone	2.59	2.56	2.54
Cyril Sandstone	2.61	2.57	2.56
Broken Bow Chert Gravel	2.61	2.60	2.58
Gore Gravel	2.56	2.52	2.49
Hugo Chert Gravel	2.58	2.53	2.49
Keota Sandstone	2.54	2.51	2.49

Table IX lists the calculated bulk specific gravity (SSD) of each of the combined aggregates plus the average bulk specific gravity (SSD) of the Cooperton limestone-Arkhola sand-siliceous aggregate blends.

Maximum Specific Gravity (Vacuum Saturation Method)

Rice (24) described a procedure for determining the maximum specific gravity of a voidless sample of bituminous paving mixture. A loose, uncompacted asphalt-aggregate mixture was placed in a calibrated volumetric flask containing enough deaired distilled water with a wetting agent to cover the sample. A vacuum was applied to the flask to reduce the air pressure in the flask and release any entrapped air from between the particles. The flask was then filled with the prepared water and weighed. The maximum specific gravity was determined from the relation:

$$G_R = \frac{A}{A + D - E}$$

where: G_R = maximum specific gravity

A = weight of coated particles in air

D = weight of flask filled with water

E = weight of flask and sample filled with water.

This method was subsequently adopted by the American Society of Testing and Materials as a standard test procedure (ASTM Designation: D 2041) (19).

The procedure followed in this study was identical to the standard ASTM test. Two compacted specimens from a given mixture having approximately the same bulk specific gravity were selected. Each specimen was

TABLE IX

AVERAGE BULK SPECIFIC GRAVITIES (SSD)

	Average Bulk Specific Gravity of Aggregate	Average Bulk Specific Gravity of Blended Aggregates		
Cooperton Limestone	2.67	2.66		
Arkhola Sand	2.65			
		Acid-Insoluble Residue		
		20%	30%	40%
Asher Chert Gravel	2.38	2.63	2.62	2.59
Miami Chert	2.53	2.65	2.64	2.63
Onapa Sandstone	2.33	2.62	2.59	2.57
Stringtown Limestone	2.52	2.64	2.63	2.61
Cyril Sandstone	2.63	2.66	2.65	2.65
Broken Bow Chert Gravel	2.53	2.65	2.64	2.63
Hugo Chert Gravel	2.53	2.65	2.64	2.63
Gore Gravel	2.46	2.64	2.62	2.61
Keota Sandstone	2.37	2.62	2.61	2.59

placed in a 10" x 15" shallow pan and heated in a large forced-air oven at 250 F ± 5 F for approximately two hours. The specimens were then broken down as nearly as possible into individual asphalt coated particles. The higher the asphalt content, the harder it was to break down the specimen into individual coated aggregate particles. The particles from the fragmented specimens were stirred until they had cooled to room temperature and, therefore, did not adhere to one another. Two 1/2 gallon volumetric flasks were calibrated at 77 F ± 0.5 F using deaired, distilled water containing a wetting agent added at a concentration of 0.1%.

The asphalt coated particles were spooned into the clean, dry flasks and the weight of the flasks and samples was determined. The prepared water was then siphoned into the flasks to cover the samples. Each of the flasks containing the water and sample was subjected to approximately 29 inches of Hg vacuum for 15 minutes. The flask was shaken vigorously several times during this period to facilitate the release of entrapped air. Care was taken not to lose the very fine particles carried to the top by the boiling water during the evacuation period. After the evacuation, the contents of the flasks were carefully brought to atmospheric pressure. Figure 31 shows the equipment used in the evacuation process.

After evacuation of the air from the sample, the flasks were filled to the top of the neck with the prepared water. The filled flasks were then placed in a constant temperature water bath set at 77 F ± 0.5 F for a period of ten minutes. The flask, filled with water and sample, was towel dried and weighed. The maximum specific gravity was calculated using the previously stated relation.

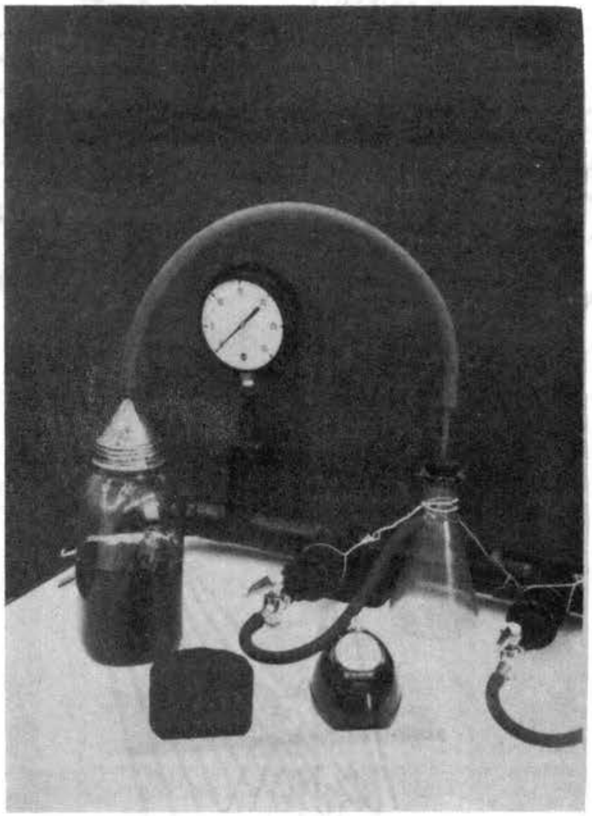

Figure 31. Equipment Used in Rice's Vacuum Saturation Test

Theoretical Maximum Specific Gravity

The theoretical maximum specific gravity of a compacted asphalt-aggregate mixture, i.e., the specific gravity of a voidless mixture, can be determined from the following relation:

$$G_{TM(ca)} = \frac{100}{\%AC/G_{AC} + \%Agg./SG_{ca}}$$

where: $G_{TM(ca)}$ = theoretical maximum specific gravity of a mix

$\%AC$ = percent asphalt content (by total weight)

$\%Agg.$ = percent aggregate content (by total weight)

G_{AC} = specific gravity of asphalt cement

SG_{ca} = average specific gravity of combined aggregate.

Note: $\%AC + \%Agg. = 100\%$

The theoretical maximum specific gravity is used to determine the percent density of a compacted bituminous specimen. Using a given G_{AC} and %AC, the theoretical maximum specific gravity of a mix depends upon the value used for the average specific gravity, SG_{ca}, of the combined aggregate. For comparative purposes, values of SG_{ca} and SG_{bi} for mixes containing 4, 4 1/2, and 5% asphalt contents, were used in the above equation to calculate two different theoretical maximum specific gravities, $G_{TM(ca)}$ and $G_{TM(bi)}$, respectively.

Comparison of Percent Density

The percent theoretical density of a compacted asphalt-aggregate mixture can be determined using the following relationship:

$$\% \text{ Density} = \frac{\text{Bulk Specific Gravity of Compacted Specimen}}{\text{Theoretical Maximum Specific Gravity of Mix}} \times 100$$

The percent density relates directly to the volume of the solids embodied in the compacted specimen and indirectly to the volume of the voids present in the specimen. Using the two previously determined theoretical maximum specific gravities, $G_{TM(ca)}$ and $G_{TM(bi)}$, and the maximum specific gravity from the vacuum saturation method, G_R, in the above relationship, three different percent densities can be determined. These percent density values, based on the "calculated method", $G_{TM(ca)}$, the "bulk impregnated method", $G_{TM(bi)}$, and "Rice's method", G_R, were plotted versus the asphalt content for each aggregate combination used in this study.

Calculated Method

Employing the theoretical maximum specific gravity of the combined aggregate, $G_{TM(ca)}$, in the above expression for percent density assumes that the material proportions in the compacted specimen are exactly the same as those used in calculating the theoretical specific gravity of the mixture. It is reasonable to expect that in preparing a given mixture, some inaccuracies in weighing the asphalt and aggregate occur and that some material is lost during the mixing and molding sequence, e.g., the small amounts of asphalt and fine aggregate that adhere to the mixing pans, implements, and mold. Thus, the molded specimen does not contain exactly the same amounts of material as were used in formulating the mixture. While this discrepancy is usually small and probably has only a minor effect on the results, it does point out an inaccuracy inherent to percent density computations when a "calculated" theoretical specific gravity value is used.

In addition, the dependency of the calculated theoretical specific gravity on the gravities of the asphalt and combined aggregate used in the mixture gives rise to other inaccuracies in the percent density determination. That is, any errors in the specific gravities of the respective materials are carried over and magnified in the percent density of the specimen. In many cases, these percent density values do not reflect the actual density of the specimen and are entirely unsatisfactory. Results of this nature were obtained in this study.

Bulk Impregnated Method

The foregoing discussion points out the necessity of using a truly representative or correct specific gravity value for the aggregate in a mixture. The use of bulk specific gravity (SSD) and bulk impregnated specific gravity values reflects an attempt to take into account variable absorptiveness, which greatly influences the gravity of an aggregate. How well these attempts succeed, i.e., how realistic the subsequent percent densities are, depends primarily on how accurately the respective specific gravity determinations are carried out.

The bulk impregnated test procedure requires that a known blend of aggregate be immersed in a comparatively large volume of asphalt cement. It is not reasonable to expect that all the air entrapped by pouring the aggregate into the asphalt is removed by stirring. This entrapped air induces erroneous values of the bulk impregnated specific gravity which is carried over in the computed percent density of the specimen. For example, increasing the volume of entrapped air decreases the bulk impregnated specific gravity which, in turn, increases the percent density. The bulk impregnated test does not provide a realistic model of a compacted specimen or an actual pavement core sample.

Rice's Method

Percent density values based on the use of Rice's measured maximum specific gravity of the mixture eliminates the inaccuracies ascribed to the use of the "calculated" theoretical maximum specific gravity and the unrealistic immersion of aggregate in asphalt. Rice's vacuum saturation method utilizes a realistic model or test specimen and the procedure is theoretically sound. In other words, the procedure is simple and straightforward with the mixture truly representative of the components in the compacted specimen. In fact, the actual asphalt content and blended aggregate gradation need not be known to determine the percent density of the specimen or pavement core sample by Rice's method.

Figure 32 shows the percent density values obtained from each of the three methods versus the asphalt content for the standard Cooperton limestone-Arkhola sand mixture. The bulk impregnated method gave a density of about 1% less than the calculated method. Since the limestone has a water absorption of only 1.14%, the bulk specific gravity (SSD) should have been larger than the bulk impregnated specific gravity; however, previously tabulated results showed the reverse to be true. Possibly, an error in the bulk impregnated test occurred.

The density obtained from Rice's method was 0.5% less than the calculated method. The low water absorption of the limestone would imply an even lower asphalt absorption. Results showed that Rice's method accounted for this low asphalt absorption since its density values

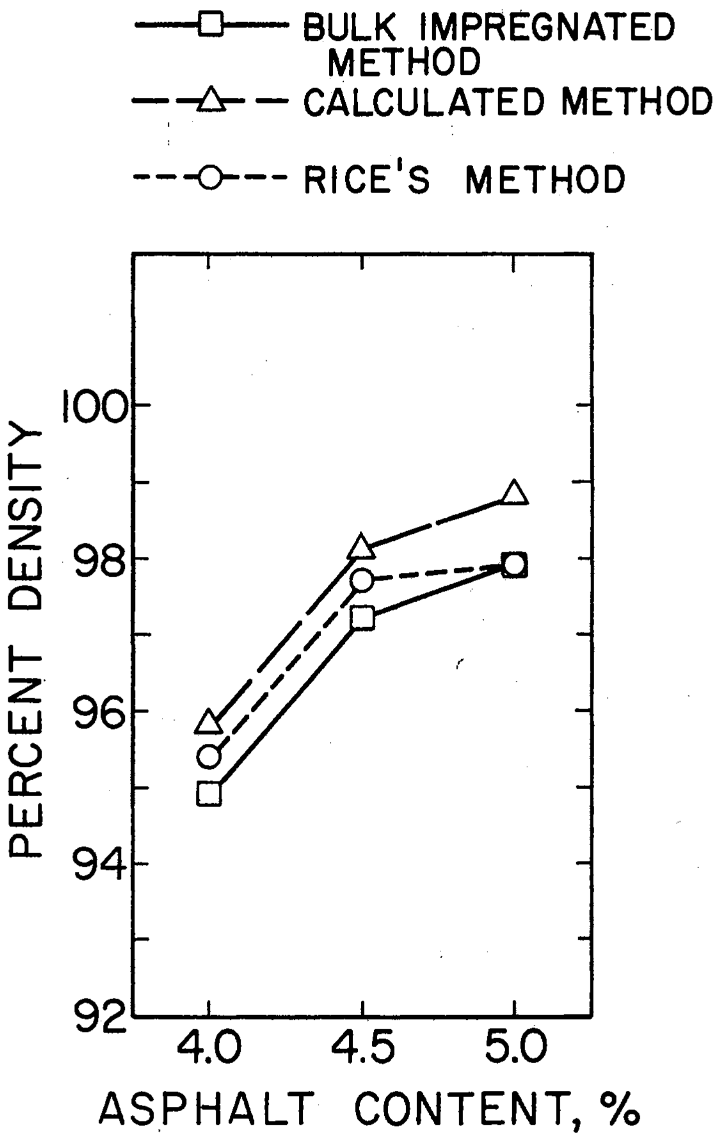

Figure 32. Percent Density Versus Asphalt Content Cooperton Limestone Mixes.

were slightly less than those from the calculated method, which did not account for asphalt absorption.

As has been stated previously, the selected gradation of the aggregate blends resulted in very dense specimens with low VMA values. Even at low asphalt contents of 4 and 4 1/2%, the percent voids were very low.

Figure 33 shows the results for the Stringtown limestone mixes. The incorporation of the entrapped air in the bulk impregnated test tended to decrease its specific gravity values which, in turn, increased the percent density over and above the percent density based on the calculated method. Results showed that density values from the bulk impregnated method were from 1 1/2 to 3% higher than the calculated method. In some cases, bulk impregnated densities were above 100%, which is unrealistic.

The closeness and parallelism of the calculated method and Rice's method percent densities were indicative of the low absorptive capability of the Stringtown limestone. Rice's method density values were lower indicating that the limestone did absorb some asphalt.

Results of the Stringtown limestone mixes were representative of the remainder of the aggregate mixes. The density values from the bulk impregnated method ranged from 1 to 5% higher than the calculated method with many of them going above 100%. The density values from the calculated method were fairly close to those of Rice's method depending upon the absorption of the respective aggregates. Results of the remaining aggregate blends are given in Appendix B.

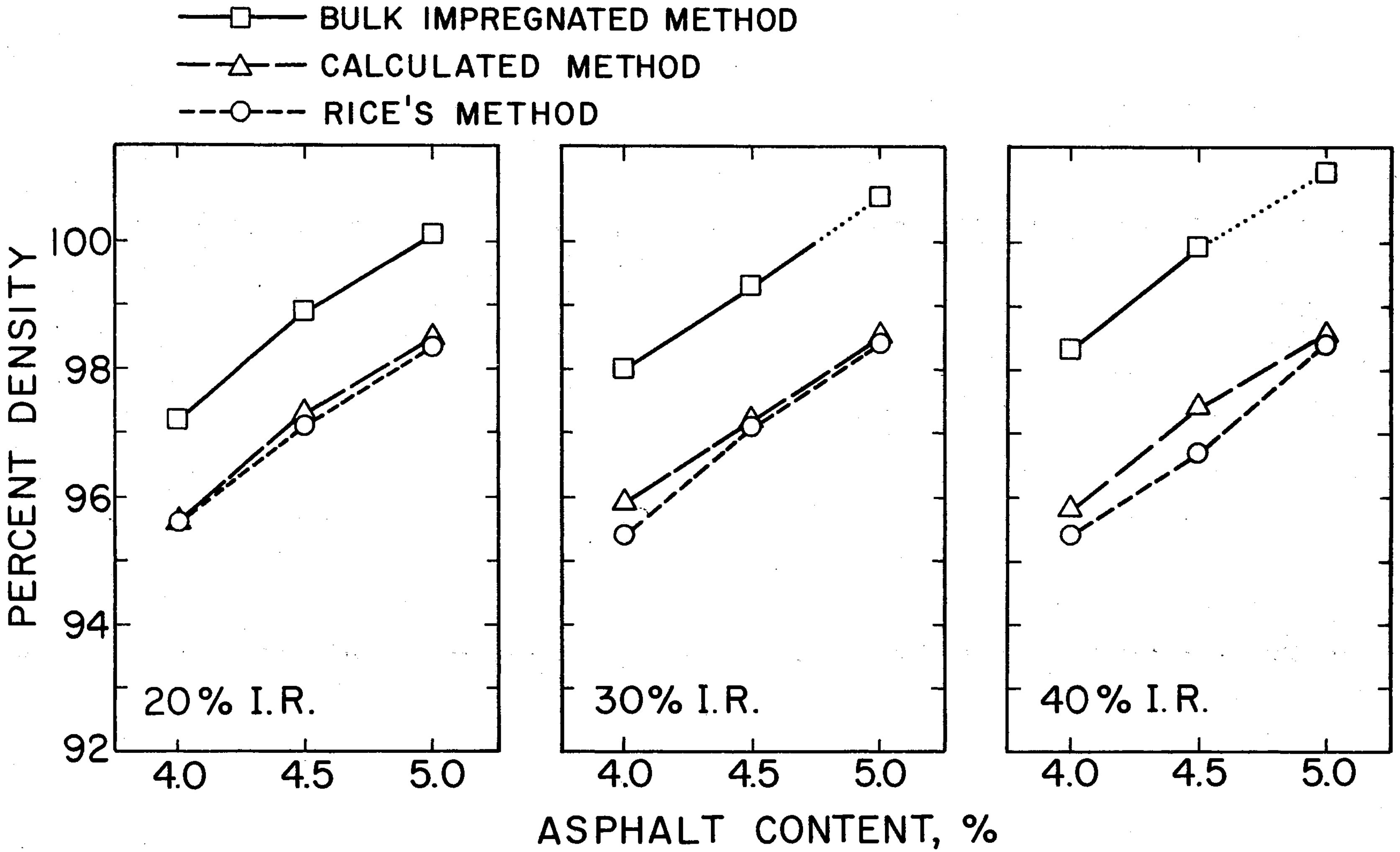

Figure 33. Percent Density Versus Asphalt Content Stringtown Limestone Mixes

CHAPTER VII

CONCLUSIONS

The following conclusions can be made based on the testing procedures and the materials employed in this study.

1. Incorporating siliceous aggregates into the standard mix had little detrimental effect on the stability of the respective mixes.

2. Various percentages of siliceous aggregates had little if any effect on the cohesion or tensile strength of the respective mixes. In all cases, cohesiometer test values were well above the recommended minimum value of 50.

3. Since incorporation of varying percentages (20-40%) of siliceous material does not influence the stability and cohesion of a mix to any great extent, the use of siliceous aggregate in a surface course bituminous mixture should be beneficial in improving the skid resistant quality of the surface. However, it is strongly recommended that a subsequent investigation be made to determine whether these siliceous aggregates have any serious effects on the durability of the mixes.

4. Selection of the mid-point gradation of the Type B specification limits resulted in a very dense-graded blend of aggregate having a very low VMA value. In many instances, this gradation produced "critical" mixtures in which slight variations in asphalt content caused drastic reductions in stability. Also, using this gradation resulted

in optimum asphalt contents ranging from 4 to 4 1/2%, well below the recommended range of 5 to 7 1/2%.

5. Cohesiometer values are highly dependent upon the manner in which the operator conducts the cohesiometer test. Higher values obtained during the latter stages of the study are attributed to improved techniques in performing the test.

6. Of the three methods employed to determine the theoretical maximum specific gravity of the asphalt-aggregate mixture, Rice's method resulted in more realistic or acceptable values of percent density for the compacted specimens.

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APPENDIX A

PROCEDURE FOR MOLDING ASPHALT-AGGREGATE SPECIMENS

Dimensions of Compaction Press and Mold

Inside Diameter of Mold = 4.0 inches

Inside Diameter of Press Cylinder = 3.188 inches

Procedure Using Motorized Gyratory Compaction Device

1. Asphalt-aggregate mixture is compacted at 250 F ± 5 F. The mold and base plate are heated to approximately 250 F. Place base plate inside mold and insert a paper disc in the mold.
2. Using a funnel, spoon the hot asphalt-aggregate mixture into the mold in three equal layers, lightly tamping each layer. After placing all the mixture in the mold, use a spatula to move any large particles away from the sides of the mold. Place a paper disc on top of the mixture.
3. Slide the mold and contents onto the rotating platen and center it beneath the ram of the press. Pump the ram down into the mold on the mixture until a pressure of 50 psi is read off the low pressure gauge.
4. Pull the cam-lever down cocking the mold to the angle of gyration. Flip the reset switch and press the start button. The mold will rotate three revolutions.
5. When the mold stops, raise the cam-lever leveling the mold. Again apply 50 psi pressure with full strokes of the pump handle. Continue this rotating procedure until one full stroke gives a pressure of 150 psi.

6. When 150 psi is obtained with one stroke, apply 2500 psi pressure by pumping the handle at approximately one stroke per second.

7. Reverse the control valve and release the vertical pressure slowly and remove the press ram from the mold by pumping the handle, which raises the ram.

8. Allow the base plate to drop out of the mold and extrude the specimen using an arbor press. Place the specimen on a masonite square, remove the paper discs, and allow it to cool to room temperature. Clean the mold and base plate after each specimen is molded.

APPENDIX B

PLOTS OF PERCENT DENSITY VERSUS ASPHALT

CONTENT FIGURES 34 - 41

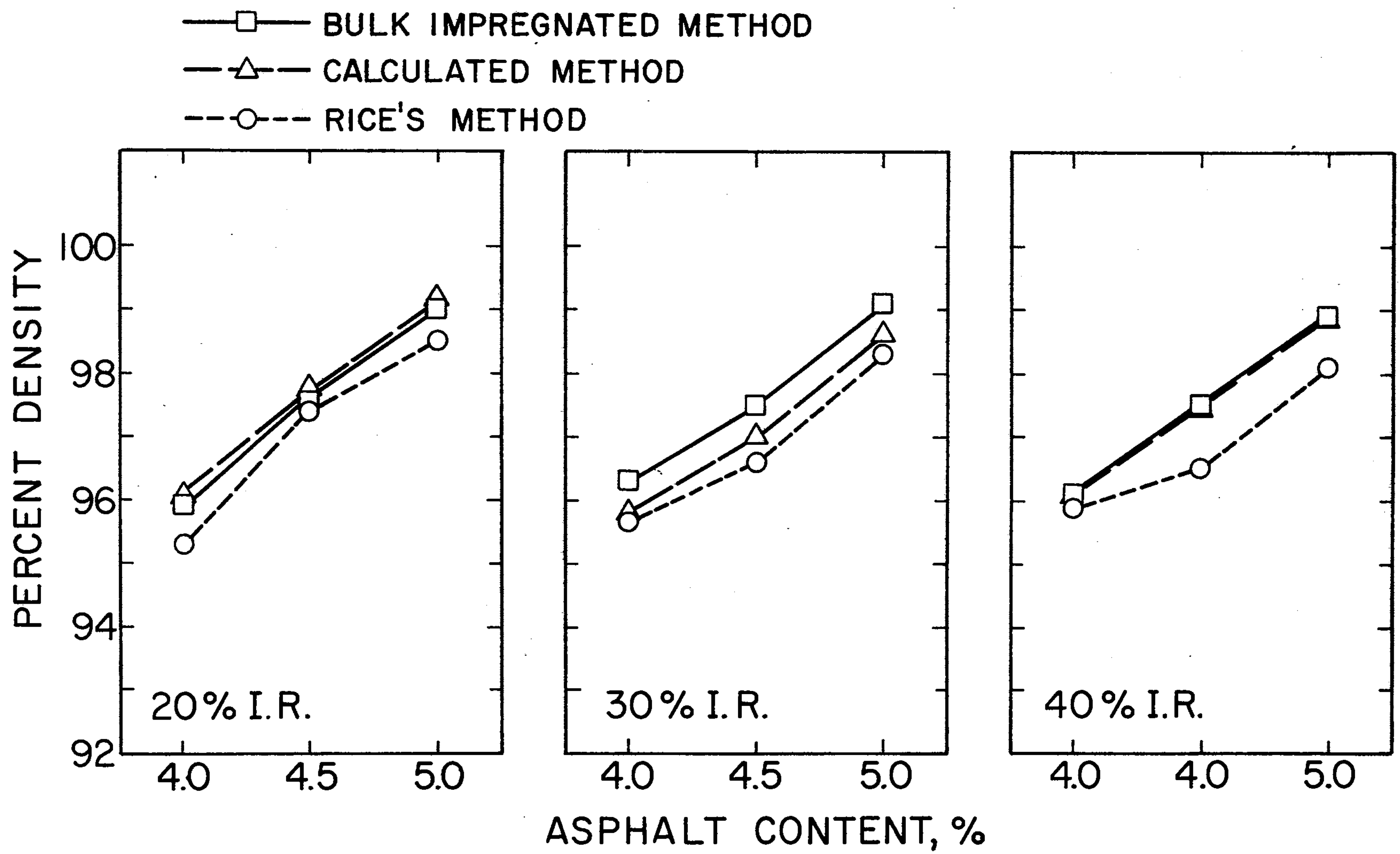

Figure 34. Percent Density Versus Asphalt Content Asher Chert Gravel Mixes

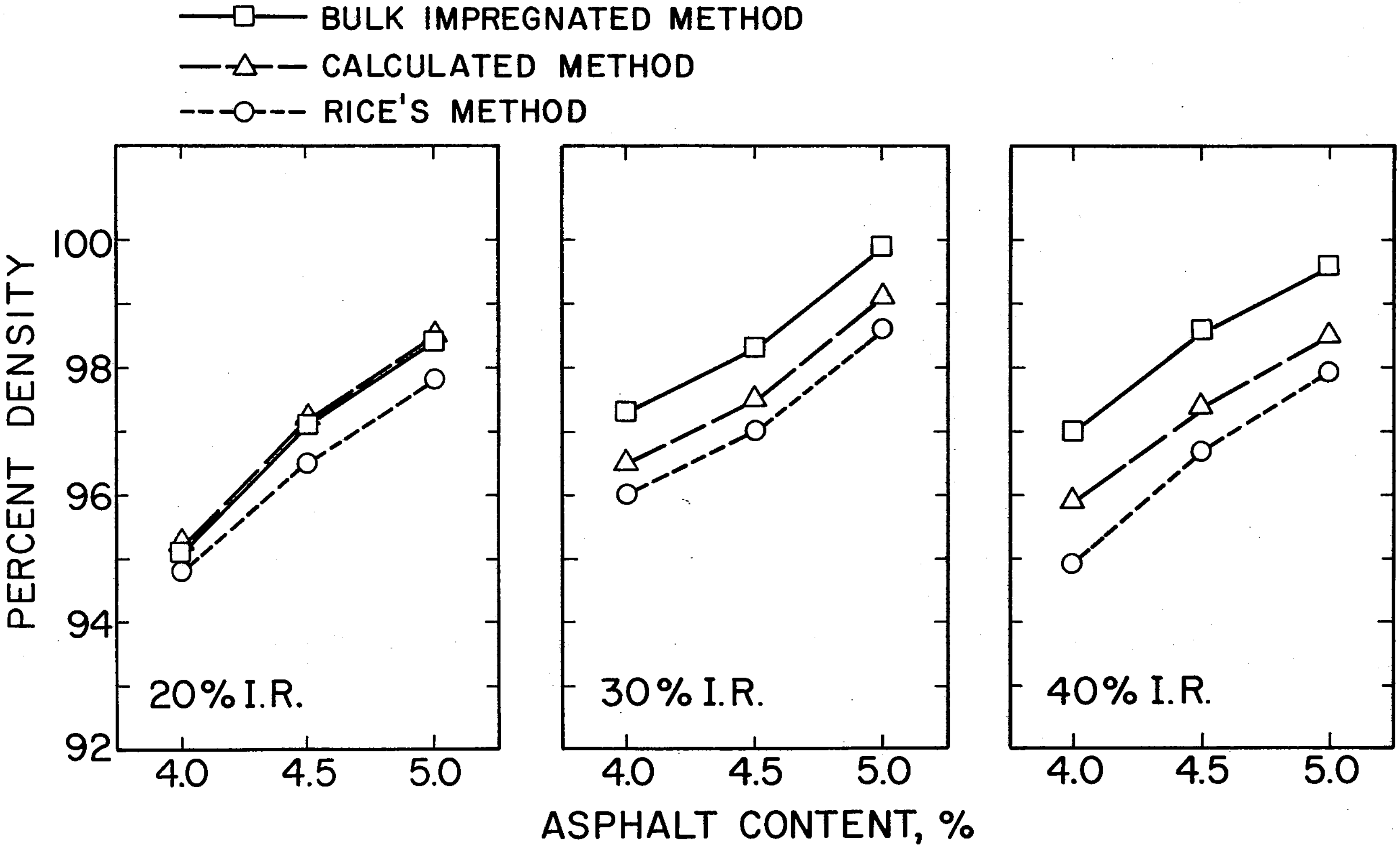

Figure 35. Percent Density Versus Asphalt Content Miami Chert Mixes

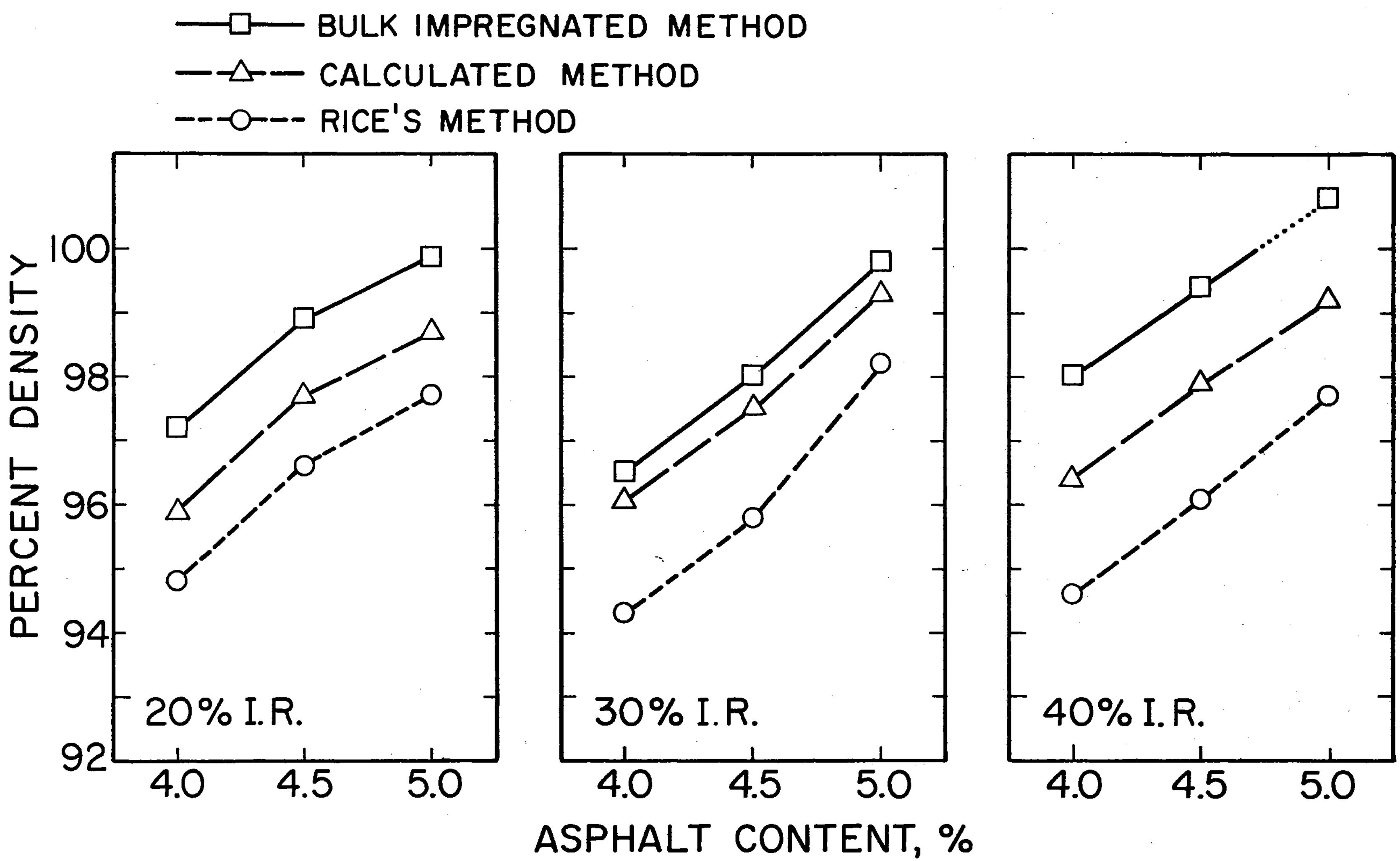

Figure 36. Percent Density Versus Asphalt Content Onapa Sandstone Mixes

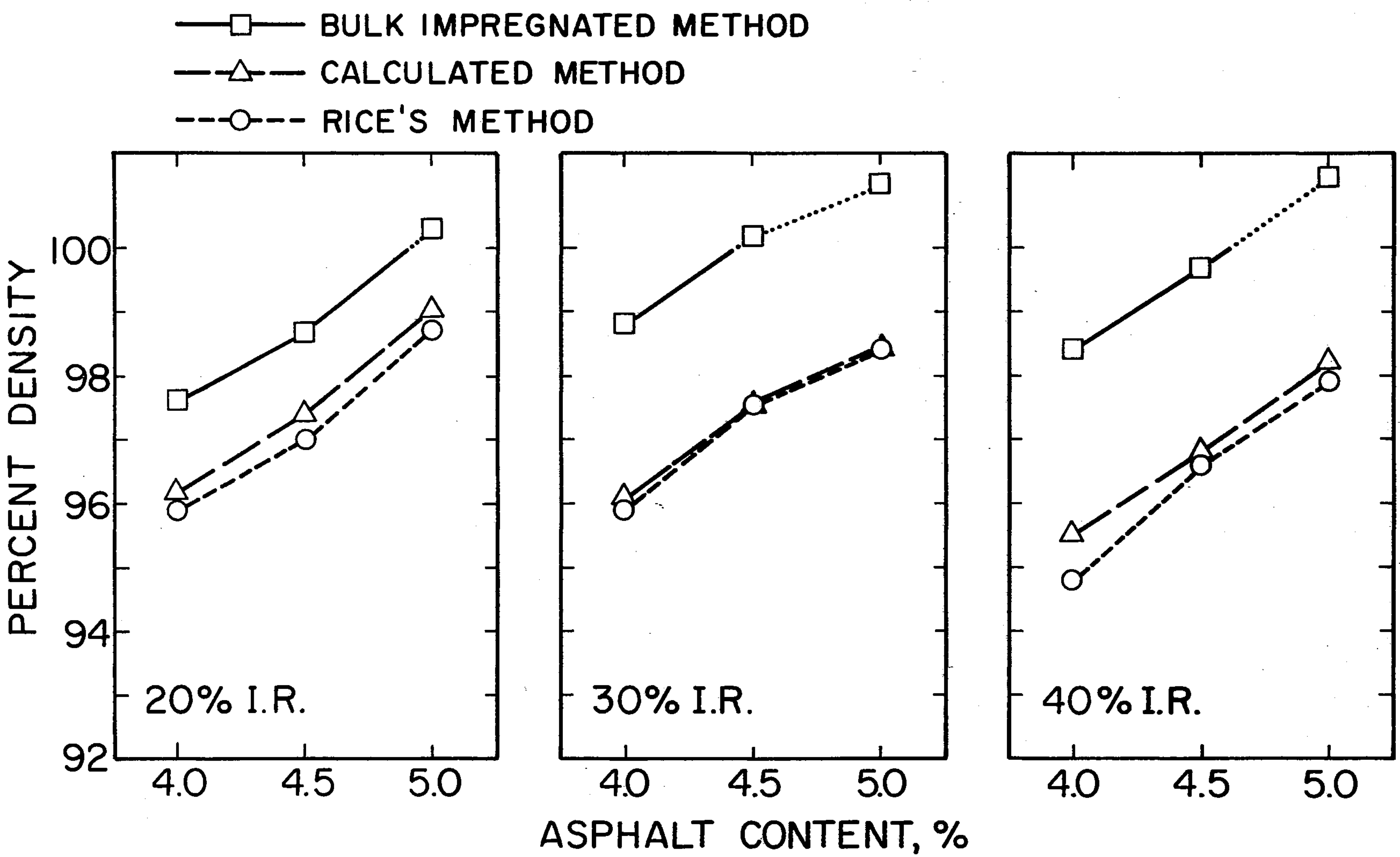

Figure 37. Percent Density Versus Asphalt Content Cyril Sandstone Mixes

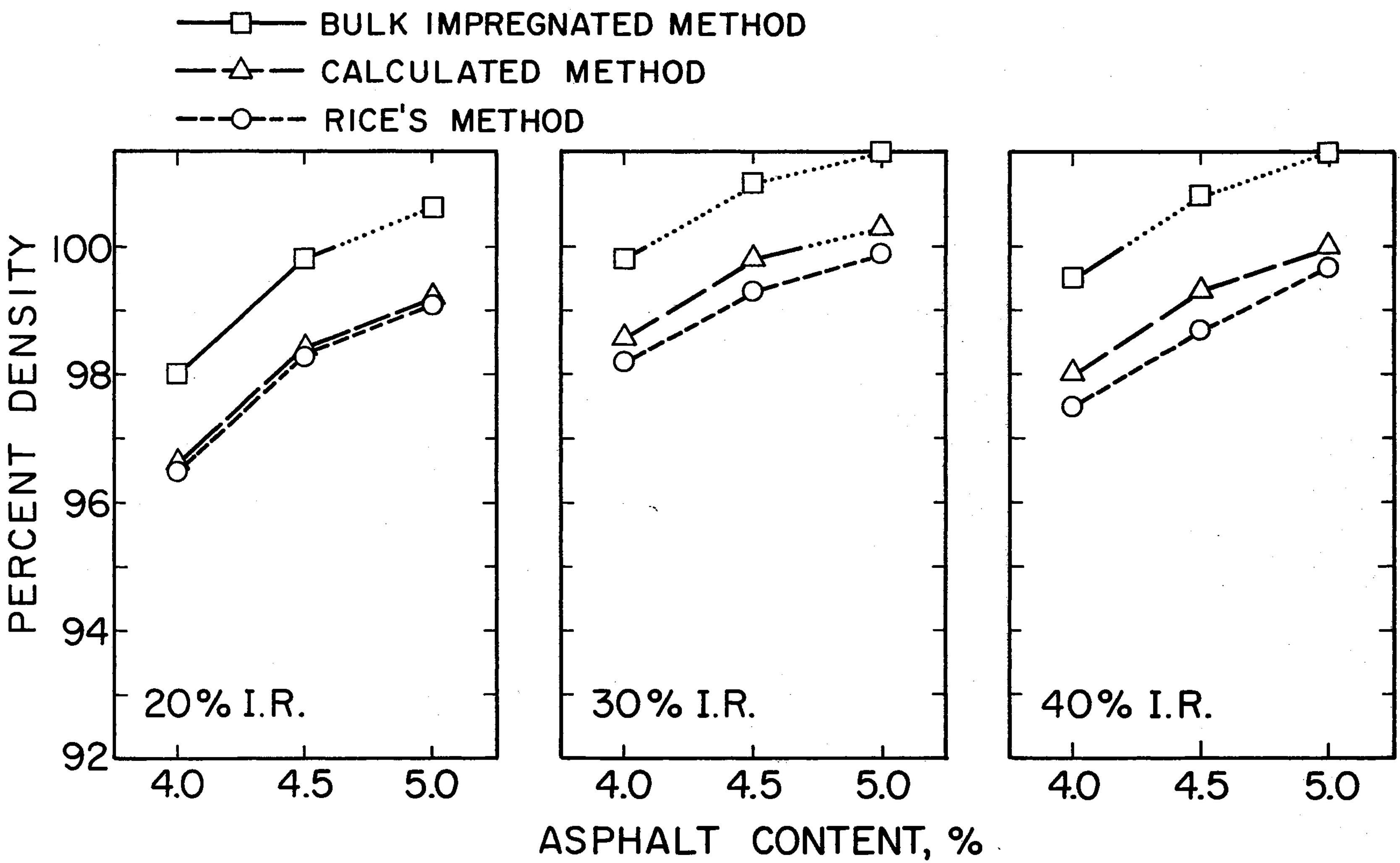

Figure 38. Percent Density Versus Asphalt Content Broken Bow Chert Gravel

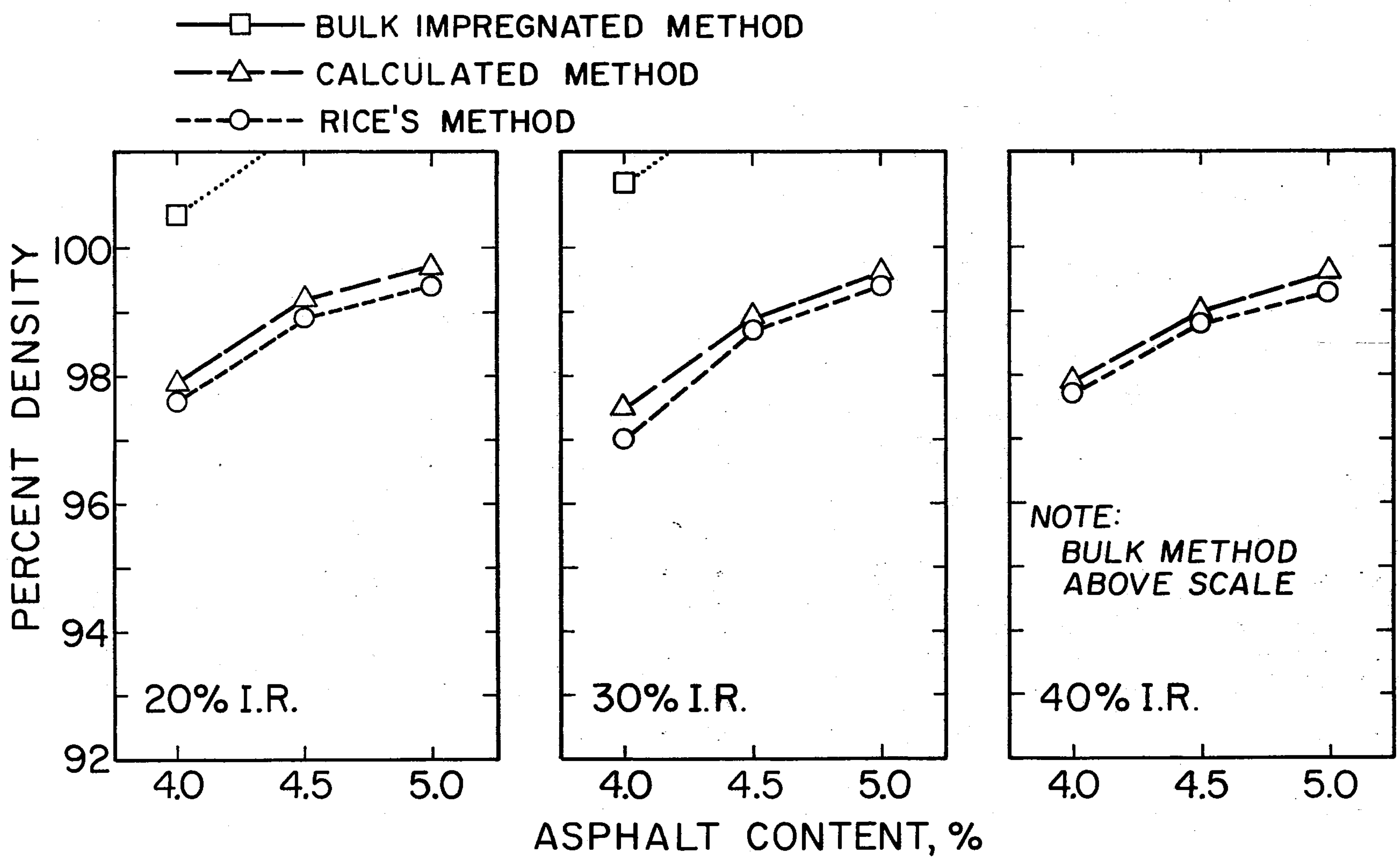

Figure 39. Percent Density Versus Asphalt Content Gore Gravel Mixes

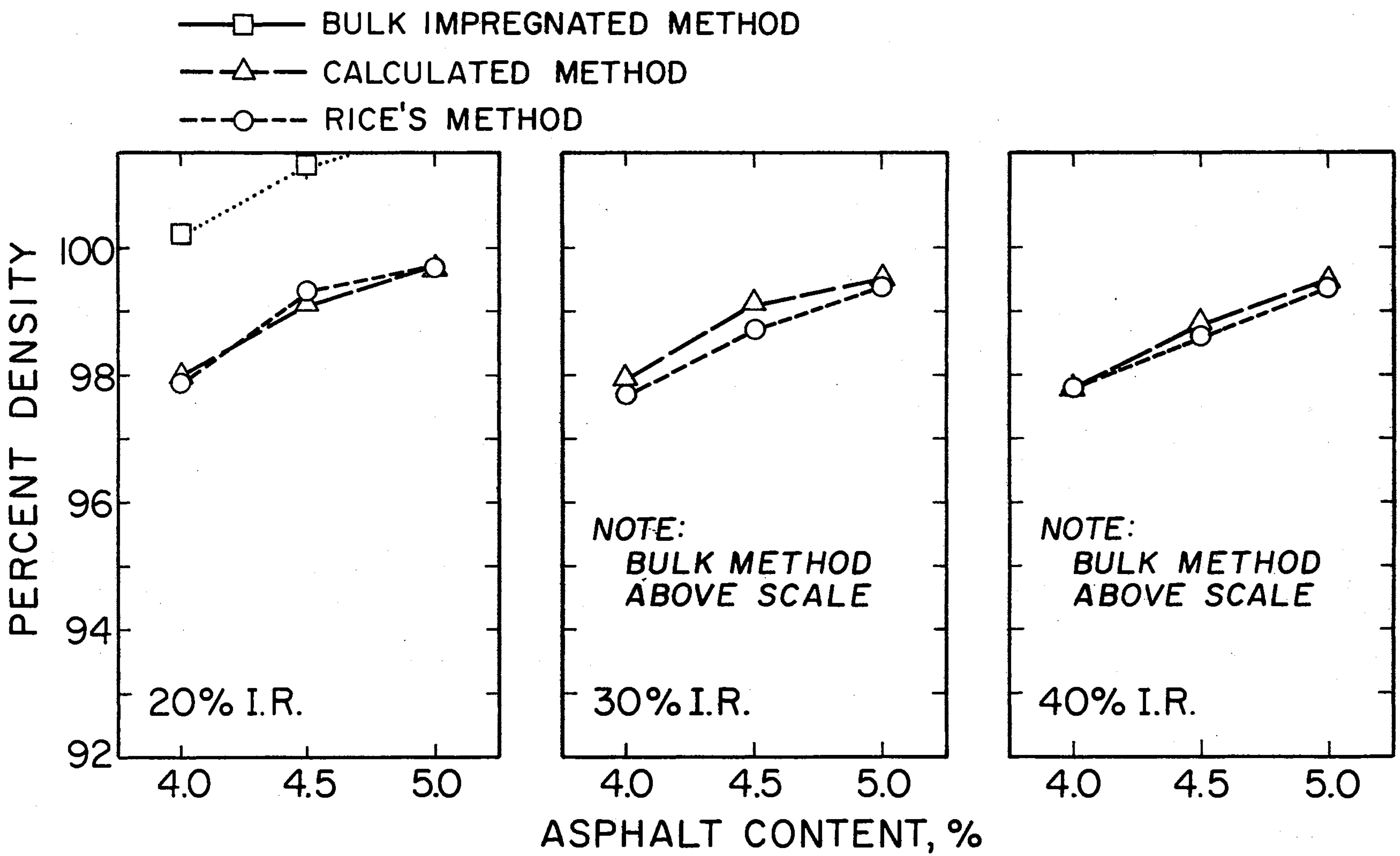

Figure 40. Percent Density Versus Asphalt Content Hugo Chert Gravel Mixes

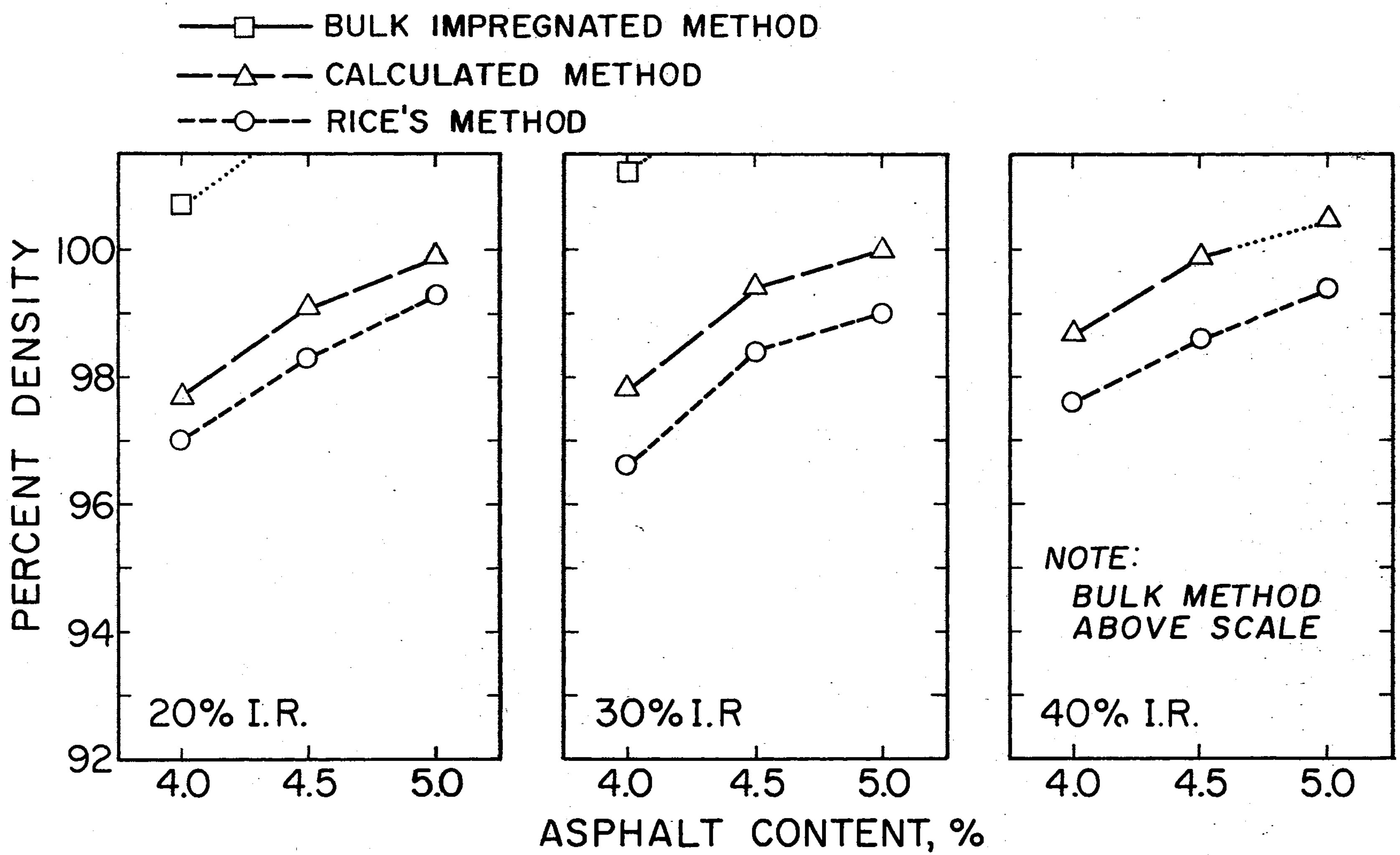

Figure 41. Percent Density Versus Asphalt Content Keota Sandstone Mixes

VITA

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