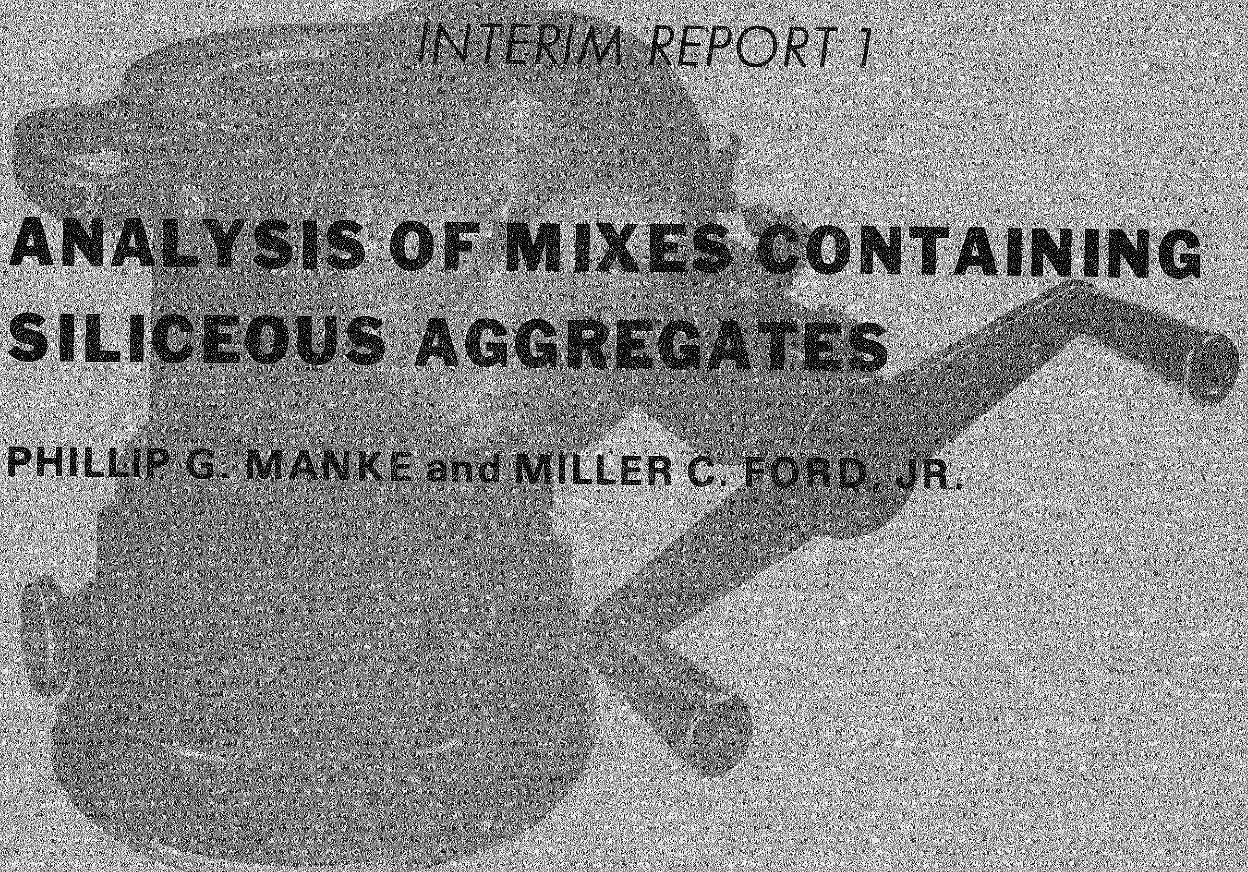
JOINT HIGHWAY RESEARCH PROGRAM
PROJECT 72-03-3

EVALUATION OF BITUMINOUS MIXES IN PAVEMENT STRUCTURES

INTERIM REPORT 1



ANALYSIS OF MIXES CONTAINING SILICEOUS AGGREGATES

PHILLIP G. MANKE and MILLER C. FORD, JR.

SCHOOL OF CIVIL ENGINEERING
OKLAHOMA STATE UNIVERSITY
STILLWATER, OKLAHOMA

LIBRARY LOCATION

11 TRAN ADMINISTRATION . . . .

12 PERSONNEL MANAGEMENT. . . . .

LAND ACQUISITION. . . . .

TRANSPORTATION FINANCE. . . .

TRANSPORTATION ECONOMICS. . .

PHOTOGRAMMETRY.

22 HIGHWAY DESIGN. . . . .

HIGHWAY DRAINAGE. . . . .

DEVELOPMENT. .

PAVEMENT DESIGN . . . .

PERFORMANCE. . . . . . .

BRIDGE DESIGN . . . .

MATERIALS AND

CEMENT AND CONCRETE . . .

GENERAL

MINERAL AGGREGATES. . . . .

GENERAL. . . . . .

AND MAINTENANCE EQUIPMENT. .

SAFETY. . . . . .

USER

TRAFFIC CONTROL AND OPERATIONS. . . .

TRAFFIC FLOW. . . . . . .

MECHANICS . . . .

. . . .

URBAN TRAN ADMINISTRATION .

82 COMMUNITY VALUES. . . . .

URBAN LAND . . . .

URBAN TRAN SYSTEMS. . . .

GENERAL . . . .

EVALUATION OF BITUMINOUS MIXES IN PAVEMENT STRUCTURES

INTERIM REPORT I

ANALYSIS OF MIXES CONTAINING SILICEOUS AGGREGATES

by

Phillip G. Manke
Project Director

and

Miller C. Ford, Jr.
Research Assistant

Research Project 72-03-3
Joint Highway Research Program

conducted for the

State of Oklahoma, Department of Highways

by the

School of Civil Engineering
Office of Engineering Research
Oklahoma State University
Stillwater, Oklahoma

June, 1973

The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the Oklahoma Department of Highways.

ne

t can

e

m

es on

es

s s

P. G. M.

M. C. F

I

VI

ons

mens

B -

X on us t

i t

l

ion

i

c

c es

I

I

t

l

t ses t

ume s

a consi e ies

as e

e es t i

li ilure

es a

res ons to

resi t

can i c.

resis , a

ile as

re s d

resis s a

on s

I ish

or d

i es in on.

limes lt

s a

d res s a

to

resis t

i si

i si

ma s) s

ly y

concerns

s

n

i

l

t i was

as t n

on

lm oven va ues. al was

as t

ve

or

sources were in

li , 3 one 4

es were

c es

ness, as i

neral its

c on

t

e as lt (on was

s

was ica s si us vacuum

process of refining. Physical properties of the asphalt cement are shown in Table I.

TABLE I

PHYSICAL PROPERTIES OF ASPHALT CEMENT

Characteristics	ASTM[1] Method	Test Value
Penetration, 77 F, 100 g, 5 sec	D5	93
Ductility, 77 F, cm	D113	150+
Viscosity at 275 F, Kinematic, cST	D2170	400
Thin Film Oven Test	D1754	
Penetration After Test, 77 F, 100 g, 5 sec	D5	60
Percent of Original		64
Ductility After Test 77 F, cm	D113	150+
Average Weight Loss	D1754	
Percent of Original		+0.018
Specific Gravity, 77/77 F	D70	1.003
Softening Point, F	D2398	118
Flash Point, F	D92	580+

[1] 1972 Book of ASTM Standards, Part 11, (1).

Aggregates

The aggregates sampled are from a wide range of sedimentary rock types. The oldest rock is an Ordivician limestone; the chert is Mississippian in age; one limestone and two sandstones are Pennsylvanian in age; one sandstone and the conglomerate are Permian and the youngest three aggregates are Quaternary gravels. The aggregates are identified as to location, geologic unit, geologic age (period) and general aggregate classification and are shown in Table II.

The Oklahoma Highway Department (OHD) has performed standard physical tests on similar aggregates prior to using them in highway construction work. Records of the OHD were examined and representative test values for the physical properties of the aggregates obtained. These values are shown in Table III. Chemical analysis of some rock types sampled were also available from the OHD and are reported in Table IV.

The aggregates that were chosen for study represent major sources of material available for highway construction in Oklahoma. The aggregate samples were secured by visiting each source and procuring a sample from stockpiles at the site. Approximately 150 lb of material of each aggregate type were obtained, representing a size gradation from the 3/4 in. to the No. 10 sieve.

The petrographic description of the limestones and sandstones are based on the study of thin sections and acid etched slabs by Willard McCasland. General quarry or pit descriptions of interest are given. The aggregates are identified by the town adjacent to their location. For purposes of description, the aggregates are divided into four groups: limestone, sandstone, chert, and gravel.

Limestone

A limestone is a bedded sedimentary deposit consisting chiefly of calcium carbonate. Limestone is generally of marine origin and the remains of sea-living organisms may be common.

TABLE II

AGGREGATE IDENTIFICATION AND SOURCE

ODH NO.	Sample	County	Location[1]			Geologic Unit Period	General Classification
			Sec.	Twp.	Rg.		
38-01	Cooperton	Kiowa	32	6N	15W	Kindblade limestone Ordovician	Limestone
03-01	Stringtown	Atoka	16	1S	12E	Wapanucka limestone Pennsylvanian	Siliceous Limestone
08-01	Cyril	Caddo	36	6N	10W	Rush Springs Permian	Calcareous Sandstone
31-01	Keota	Haskell	23	10N	23E	Bluejacket Pennsylvanian	Siliceous Sandstone
46-01	Onapa	McIntosh	31	11N	17E	Bluejacket Pennsylvanian	Siliceous Sandstone
63-01	Asher	Pottawatomie	4	6N	4E	Wellington-Admire Permian	Chert Gravel[2]
45-01	Broken Bow	McCurtain	4	7S	26E	Alluvial Deposit Quaternary	Siliceous Gravel
68-01	Gore	Sequoyah	19	12N	21E	Alluvial Deposit Quaternary	Siliceous Gravel
12-01	Hugo	Choctaw	36	5S	17E	Terrace Deposit Quaternary	Chert Gravel
58-01	Miami	Ottawa	31	29N	23E	Boone Mississippian	Chert

[1] Based on USPLS Indian Meridian.

[2] From a loosely consolidated conglomerate.

TABLE III

AGGREGATE PHYSICAL PROPERTIES

Sample	Bulk Specific[1] Gravity	Absorption	L. A.[2] Abrasion	Soundness[3]		Insoluble[4] (%) (+ No. 200 sieve)
				$NaSO_4$	$MgSO_4$	
Cooperton	2.67	0.8	24	0.8	4.4	1.2
Stringtown	2.57	0.5	22	4.4	6.3	72.8
Cyril	2.64	0.9	37	4.1	---	59.2
Keota	2.48	2.4	40	---	---	96.3
Onapa	2.47	4.1	35	8.9	---	92.1
Asher	2.46	3.2	25	6.5	---	99.8
Broken Bow	2.69	1.3	25	---	---	98.3
Gore	2.68	0.6	29	---	2.7	97.9
Hugo	2.52	1.8	20	---	2.8	99.0
Miami	2.56	1.2	23	2.9	---	95.4

[1] Reference ASTM Designation: C 127.

[2] Reference ASTM Designation: C 131.

[3] Reference ASTM Designation: C 88.

[4] Reference Oklahoma Test Method OHD-L-25.

TABLE IV

AGGREGATE CHEMICAL ANALYSIS

Sample	Per cent by Weight								
	$CaCO_3$	$MgCO_3$	SiO_2	Al_2O_3	Total Fe (as FeO)	P_2O_5	K_2O	Na_2O	S
Cooperton	90.98	2.36	4.48	0.69	0.21	0.01	0.07	0.07	0.11
Stringtown	30.47	0.58	62.42	2.36	1.95	0.13	0.12	0.03	0.23
Cyril	22.10	15.17	59.00	2.62	0.68	0.08	0.16	0.08	0.11
Keota	0.97	0.76	90.57	6.16	1.21	0.05	0.20	0.06	0.02
Onapa	2.18	1.65	83.00	10.56	1.63	0.08	0.33	0.06	0.02

Oklahoma has abundant limestone sources in some sections of the state. Since limestone was considered to be highly resistant to stripping, one limestone (Cooperton) of high purity was chosen for use as a basis for comparing relative stripping tendencies with other aggregates. Two other limestones of decreasing calcium carbonate content, i.e., decreasing purity, were also studied.

Cooperton. This material came from the folded limestones and dolomites of the Arbuckle Group that form hogbacks in northeastern Kiowa County. These hogbacks are generally known as the Limestone Hills Area of the Wichita Mountain Uplift. The quarry working face is about 210 feet high, out of a massively bedded limestone ridge which stands about 300 feet above the surrounding terrain. The structural dip of the formation was about 20 degrees north.

The rock is a gray to mottled gray and buff colored, very fine-grained, fossiliferous, hard limestone. The Cooperton specimen contains 32 per cent pellets and oolites, ranging in size from 0.05 to 1.0 mm, with a median size of 0.35 mm. Grains are 100 per cent rounded, medium to tightly packed, well sorted and are cemented with sparry calcite. Micrite composes 30 per cent and dolomite 3 per cent. There are a few veins which are filled with calcite. The rock fractures along the calcite vein. There is no evidence of shale within the quarry site. The rock is classified as biopelsparite using the Folk (2) system and packstone using the Durham (3) system.

Stringtown. This sample consists of about 60% tannish-gray to chocolate brown, fine-grained limestone, 30% dark grayish-brown chert, and 10% sandy shale. These were present in alternating layers about 1 in. in thickness.

The quarry is located southeast of the Choctaw Fault and northwest of the Tri Valley Fault in the frontal belt of the Ouachita Mountains. Beds are very steeply dipping to vertical in attitude. The working face of the quarry is about 240 feet high and is worked in 60 feet lifts.

Numerous veins, filled with calcite, are present at 0.2 inch to 1.0 inch intervals. The thin section contains two distinct units compositionally: one composing 80% of the slide is chert and the other is dolomite. The crystals of dolomite are 0.04 to 0.2 mm in size and are loosely packed. The chert is very dense and is composed of cryptocrystalline quartz (chalcedony). The rock tends to fracture along the calcite veins. Minor amounts of pyrite are present in the specimen. The rock is classified by McCasland (4) as siliceous limestone.

Sandstone

Sandstone is a sedimentary rock composed of noncarbonate grains 0.06 to 2.0 mm in diameter, which are cemented together in some fashion. The cementing material may be quartz, opal, calcite, dolomite, clay, or oxides of iron, either reddish (hematite) or yellowish (limonite). The colors are variable, white to gray, buff to dark yellow, and red to reddish brown are common (5). These colors depend largely on the nature of the cement.

According to McBride (6) many different classification systems for sandstone have been proposed, but none has been devised which adequately treats all of the important sandstone attributes. The sedimentary structure, texture, and composition of the sandstone are the three main characteristics used in their study. Composition is generally the most important feature for evaluation as a highway material.

Cyril. This quarry in the Anadarko Basin area has a working face of about 100 feet into the massively bedded sandstone, which contains some cross-bedding. This rock is mostly gray, with some red and yellow. It is a calcareous to dolomitic sandstone.

The quartz sand grains were well sorted, subangular to subrounded, loosely packed and cemented together with recrystallized microspar. The grains range in size from 0.02 to 0.3 mm, with a median size of 0.15 mm. Some of the grains were partially coated with hydrocarbon material. No veins were evident in the section. The quartz grains comprised 42% of the section. The specimen is classified as a quartzose sandstone (quartzarenite) (6), with calcareous cement.

Keota. The quarry face consists of about 20 feet of gray, massive to thin-bedded sandstone, overlain by a brown thin-bedded sandstone, about 6 feet thick. The sample consists of 80% quartz grains and 1% feldspar bound together by a chert cement.

Grains are well sorted, subangular, closely packed, and range in size from 0.05 to 0.4 mm, with a median size of 0.2 mm. Grains appear corroded and sutured together. About 30% of the sample has a distinct yellowish color or stain. Some thin laminae of organic material were noted. The specimen is classified as a quartzose sandstone (6).

Onapa. The quarry has a 20 foot working face, with 7 feet of grayish-tan, thin to thick-bedded sandstone underlain by interbedded shale and sandstone. The overburden is buff sandstone and gray shale, about 3 feet thick.

The specimen consists of 68% quartz grains, and 31% siliceous matrix (chert and chalcedony). Carbonaceous material, present as striations, comprises 1% of the specimen. Grains are subangular, well sorted, and range in size from 0.04 to 0.20 mm, with a median size of 0.15 mm. The quartz grains are tightly packed and appeared corroded or etched by the matrix. The specimen is classified as quartzose sandstone (6).

Chert

Chert is composed of cryptocrystalline quartz of various varieties. It is characterized by its hardness, conchoidal fracture, or splintery fracture if porous, and a variety of colors. The more dense rocks are very tough and are usually gray to black, or white to brown and have a waxy luster. The porous varieties have a chalky-like surface and are generally lighter in color; white or yellow, brown and red stain are common (7).

Miami. The sample of whitish gray chert was obtained from a stockpile of crushed waste material from the Eagle-Picher Zinc Mine. The material was excavated from massive chert beds, with 50 to 70 feet of working face in the mine. The aggregate contains 92% chert, 6% limestone and 2% dolomite, zinc, iron and other trace metals. The crushed rock edges tend to be very sharp.

Gravel

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

The particles which compose gravel are rocks fragments and individual minerals.

The form and appearance of these pebbles depend on the conditions of erosion, transportation, and deposition. Those which have experienced considerable transportation are likely to have a very smooth surface with a characteristics faintly dimpled, slightly dented appearance caused by their repeated collisions during movement. If the pebble is composite in nature, it commonly is pitted by weathering and removal of softer or more easily altered minerals (5).

Asher. This material was obtained from a pit which has a working face of about 15 feet in the Maud conglomerate. Overburden is 10 to 15 feet of sandstone. Large aggregate (1.5 in. and above) is separated at the crusher for use as decorative stone.

The aggregate is a whitish pink to brown, fairly homogeneous gravel, with some hematite stain. An analysis indicates that 94% of the sample is banded chert and 6% is cherty limestone, with some hydrocarbons present as a surface coating. The material is classified as a chert gravel (4).

Gore. This material is from a deposit in the Arkansas River, where it was obtained by dragline operations. The excavating procedure secured material which is fine sand to cobble size.

The aggregate pieces are multicolored, heterogeneous and subangular to rounded before crushing. Mineral and rock analysis of the sample indicates that it is 59% quartz, 22% chert, 10% granite, 8% feldspar, and 1% sandstone. The material is classified as a siliceous gravel (4).

Broken Bow. This sample was obtained from the flood plain adjacent to the Mountain Fork River. The pit was 25 to 30 feet deep and was worked by dragline. Soil overburden averaged 5 feet in depth. Boulders, cobbles, and sand represent the sizes of material obtained from the pit.

The gravel is 50% quartz, 24% chert, 21% quartzitic sandstone, and 4% metamorphic rocks. The aggregates are subrounded prior to crushing operations. The material is classified as a siliceous gravel (4).

Hugo. This material is a terrace deposit of the Kiamichi River. The pit excavation was operating on a 10 feet deposit of sand and gravel, which had been covered with 2 to 4 feet of soil overburden. The aggregate is 93 to 95% chert, 5 to 7% sandstone, with traces of quartz, ironstone, and feldspar. The aggregate is a homogeneous mixture of brown chert rocks, which are well rounded before crushing. The aggregate is classified as a chert gravel (4).

CHAPTER III

DESIGN PROCEDURES

In general, commonly used mix design procedures and specimen preparation techniques were employed in this study. Details of these procedures have been omitted for the sake of brevity. Where appropriate, references to specific procedures are listed and deviations from these standard methods are discussed. For the immersion-compression and stripping evaluation phase of the work, special methods of specimen preparation and modified standard test procedures were used. These are described in detail under the appropriate heading in Chapter IV.

Mix Design

The aggregate gradation used for the mixes was based on the Oklahoma Highway Department specifications (8) 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 V. Figure 1 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 (9, 10).

TABLE V

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, (8).

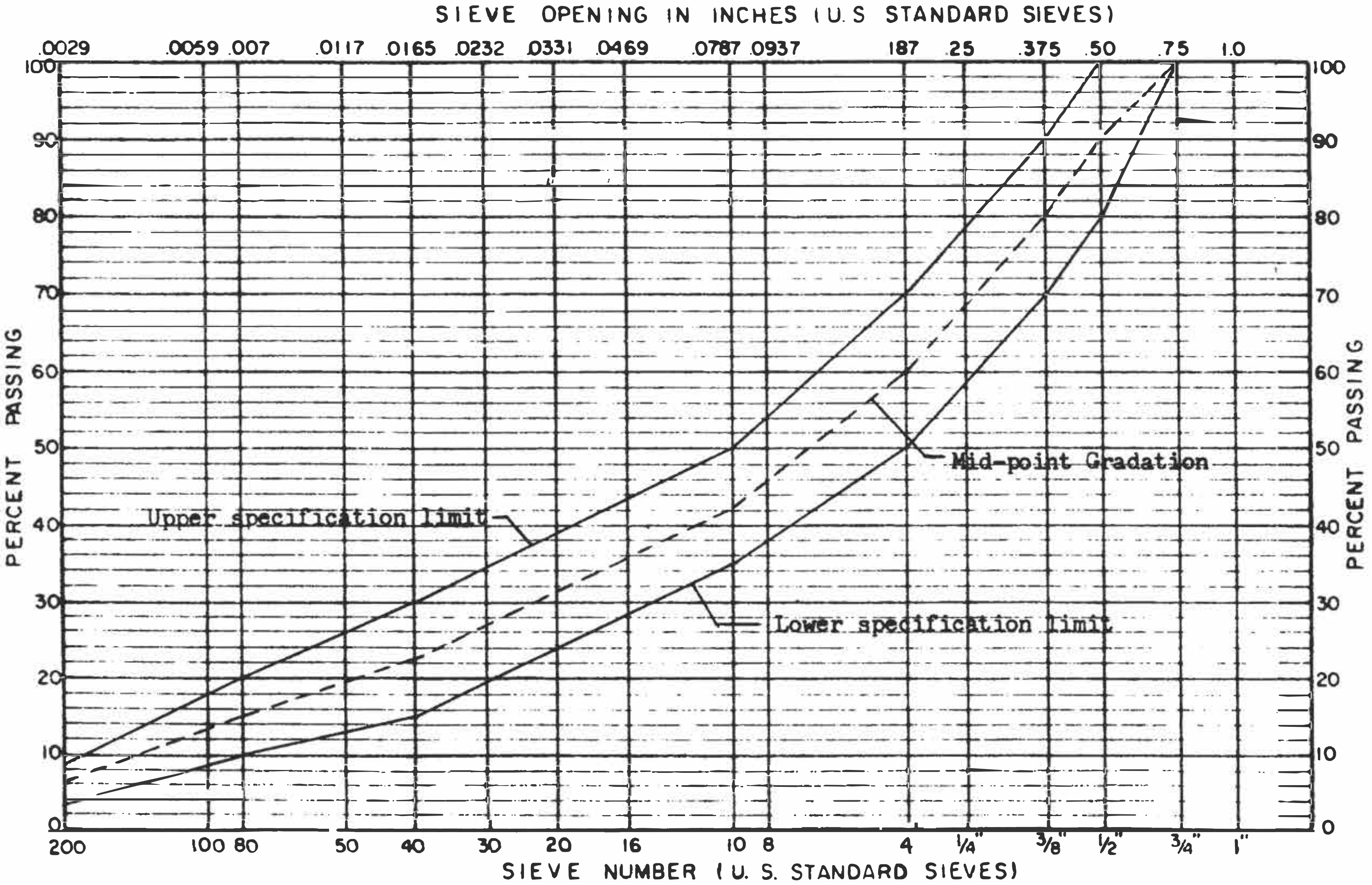


Figure 1. Type "B" Specifications and Mid-Point Gradation

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 combination 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.

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 VI. 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 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 (11). The acid-insoluble residue values for the various aggregates were obtained from the OHD and are shown in Table III.

TABLE VI

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

Asphalt-aggregate mixtures containing 20, 30, and 40% (by weight of aggregate) acid-insoluble material were prepared and tested. 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.

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 VII.

Preparation of Specimens

Pans containing 1000 grams of the blended aggregates were placed in a large gas-fired oven and brought to 250 F ± 10 F. The asphalt cement was placed in a large forced-air oven and heated to 250 F ± 10 F. Using a Mettler P-3 balance, the proper amount of 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 2). 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

TABLE VII

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

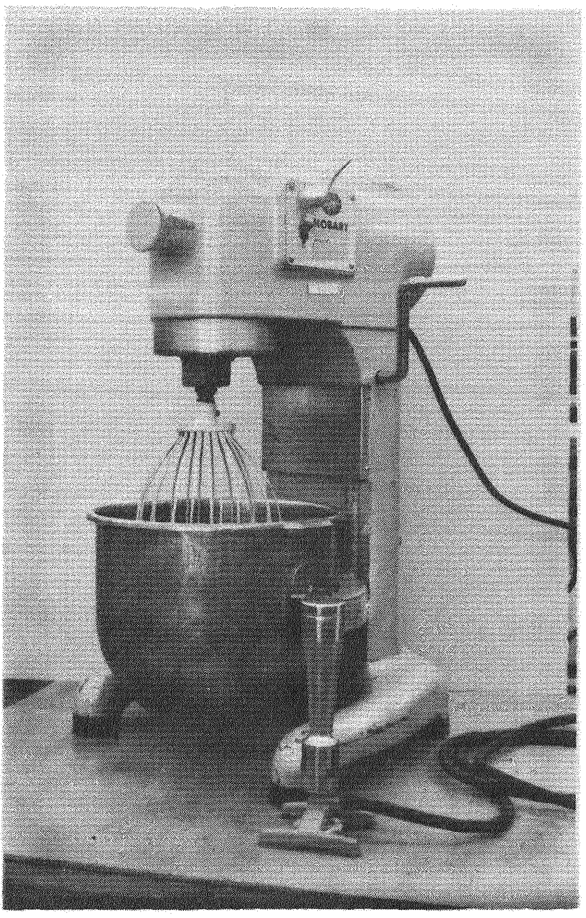


Figure 2. Hobart C-100 Mixer With Wire Whip Attachment

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 2 to 4 minutes of mixing was 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 (12). 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 3).

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 cooling 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 100 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.

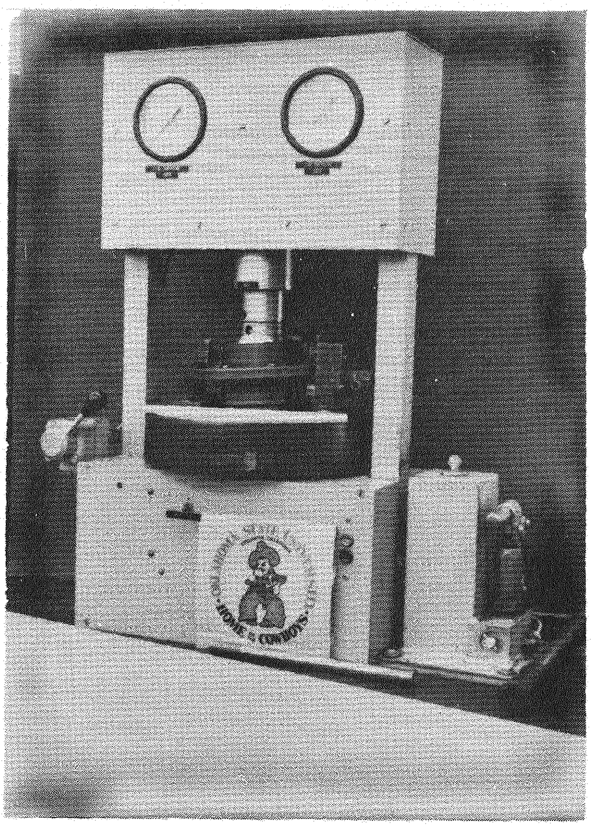


Figure 3. Motorized Gyratory Shear Compaction Device

This procedure for molding the specimens conforms closely to that described in the test method, OHD-L-8 (11), except that the Oklahoma Highway Department used 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. 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.

CHAPTER IV

TEST PROCEDURES

Following the molding sequence of operations the hot test specimens were extracted from the mold, marked with an identification number, and allowed to cool. After cooling to room temperature, the height of each specimen was determined by averaging a series of measurements made at the center and the ends of two orthogonal diameters of the specimen. The bulk specific gravity of the compacted specimens was then determined using the method outlined in test procedure OHD-L-14, Method B (11).

Stability Test

The stabilometer test, ASTM Designation: D 1560 (1), 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 compacted asphalt-aggregate specimen subjected to a given vertical pressure. The test values indicate the relative ability 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 4 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 test loads of 500 lb, 1000 lb and each 1000 lb thereafter.

The stability value, S, was then determined from a conversion chart, or graphical solution of Hveem's stability equation. Because of the influence of the height on the relative stability value, the measured values (for specimens of various heights) were converted to equivalent stability values for a standard height specimen using a correction chart.

Cohesion Test

The cohesiometer test, ASTM Designation: D 1560 (1), 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 5 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

Figure 4. Stabilometer and Compression Testing Machine

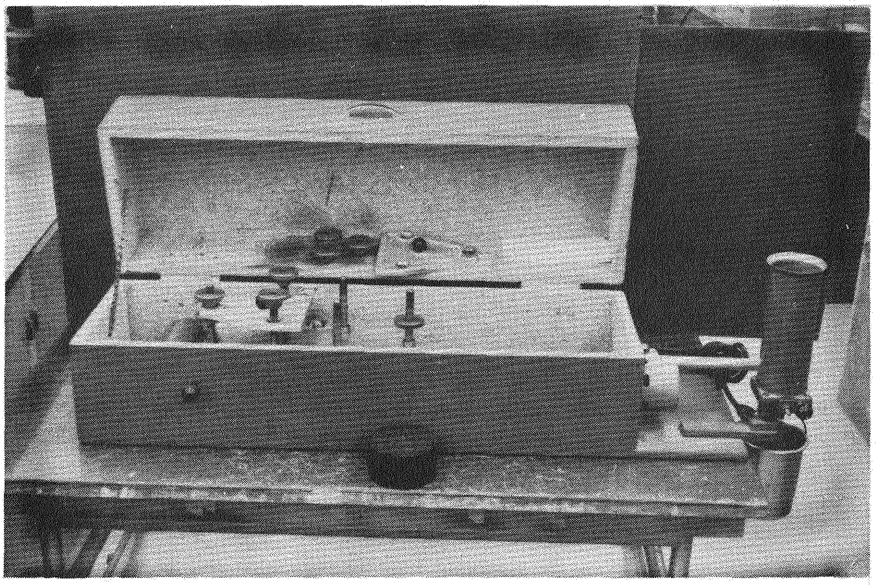


Figure 5. Cohesiometer

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:

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

where: C = cohesiometer val (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).

Immersion-Compression Test

The immersion-compression (I-C) test procedure adopted was patterned after the standard method, ASTM Designation: D 1075 (1). However, several variations from this standard method were used to take advantage of available molding and testing equipment. The specimens were molded with a motorized gyratory-shear compactor, rather than using the static double plunger compression method of ASTM. Instead of the ASTM method of testing in block compression, the specimens were tested in axial compression, across the diameter, using a Marshall Stability Testing Head. The specified ASTM specimen height to thickness ratio is about 1.0, while the specimens tested in the Marshall head had a height to thickness ratio of about 2.0.

Two series of specimens were used in the I-C tests. The first series consisted of specimens which had been mixed, molded, and tested for stability and cohesion. This series included the 4, 4 1/2, and 5% asphalt content specimens from the 20, 30, and 40% insoluble residue mixtures of each of the siliceous aggregates. These compacted specimens were heated to approximately 250 F, broken down with a spatula, and then remolded and tested according to the following described procedures.

Test results from the first series of specimens were considered inconclusive regarding the relative stripping tendencies of the siliceous aggregates. Consequently a second series of I-C specimens were prepared from each of these materials. In this series the coarse aggregate fraction (3/4 in. to No. 10 size) of the basic mixture, described in Chapter II, consisted of 100% of the respective siliceous aggregate. In order to obtain comparable immersion-compression data, 4 specimens were molded at 4% asphalt content and 4 specimens were molded at 5% asphalt content.

Mixing and Molding

The blended aggregate mix and asphalt cement were heated to a temperature of 325 F. The aggregate was placed in the mixing bowl and the proper amount of asphalt cement added. The mixing was accomplished for each individual specimen using a Hobart Mixer. About three minutes of mixing time was required to fully coat the aggregate with asphalt.

The mixture was then placed in a holding oven to obtain a molding temperature of 260 ± 5 F.

The specimens were molded using the motorized gyratory-shear compactor. The standard molding procedure was modified for the immersion-compression specimens by gyrating the mixture one cycle (3 revolutions) under a load of 50 psig, rather than gyrating the specimen at 50 psig pressure until the end point of 100 psig pressure was obtained with one full stroke of the hydraulic jack. A leveling pressure of 800 psig was then used to obtain final compaction. The standard procedure requires a leveling pressure of 2500 psig.

This modified molding procedure was used in preparing all of the immersion-compression test specimens. After compaction, the specimens had a 4 in. diameter and were approximately 2.1 in. high. Each group of specimens was divided into wet and dry sub-groups such that the bulk specific gravity of the specimens in each sub-group were approximately equal. After grouping, the specimens were cured 24 hours in a 140 F oven.

Curing and Vacuum Saturation

Preliminary investigations revealed that the mid-point gradation of the OHD Type B mixture was a relatively dense mixture and that the specimens were not fully saturated when soaked in a 140 F water bath for 24 hours. Previously published research by Pauls and Goode (13) showed that vacuum saturation of open graded mixtures resulted in no detrimental effects to the specimen. Therefore, it was deemed desirable to vacuum saturate the wet specimens to insure there was water available to provide an opportunity for stripping.

The vacuum saturation process used was based upon work done by Manke (14). The wet specimens were placed in a vacuum dessicator, and the air evacuated for 10 minutes, the specimens were flooded with deaired distilled water and the vacuum process continued an additional 10 minutes, using a vacuum of 29 inches of mercury. Application of the vacuum was then discontinued and the system opened to atmospheric pressure forcing water into the void spaces.

The saturated surface dry weight of the wet specimens was then obtained to determine the amount of water absorbed. These specimens were then immersed for 24 hours in a 140 F distilled water bath.

The dry specimens were placed in plastic, water tight, bags and cured at room temperature for 24 hours. The wet specimens and dry specimens (in their water proof bags) were placed in a 77 F water bath 2 hours prior to their compressive test.

Testing

The ultimate strength of each specimen was determined using a Marshall Stability testing head. The arrangement of the specimen and Marshall testing head in the testing machine is shown in the photograph of Figure 6. Loading rate of the testing machine was adjusted for 0.2 inch per minute. The compressive strength of each specimen was determined by dividing the maximum load obtained by the specimen cross-sectional area. The percent of retained strength of each group of specimens was determined by dividing the average strength of the wet specimens by the average strength of the dry specimens and multiplying by 100.

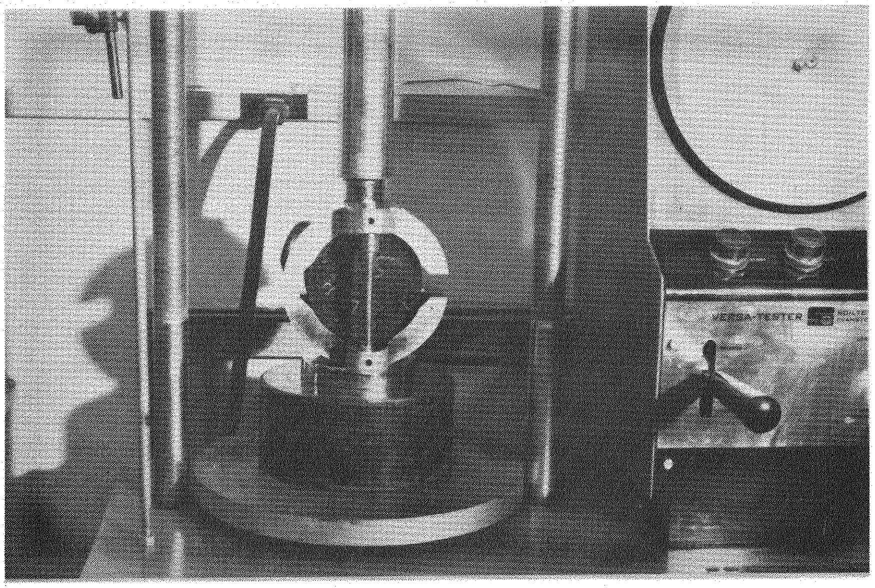


Figure 6. Immersion-Compression Test Using the Marshall Stability Testing Head

Static and Dynamic Stripping Tests

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

Sample Preparation

Each of the ten coarse aggregates being evaluated in this work had been previously sieved into different sizes for the I-C mixtures. The 3/8 inch to No. 4 size material was quartered and resieved to obtain approximately 2000 g of material passing the 3/8 inch sieve and retained on the 1/4 inch sieve.

The aggregate sample was then washed, oven dried, and quartered to obtain representative samples of approximately 100 g each. Exactly 100.0 ± 0.2 g of the dry aggregate was weighed and placed into large aluminum moisture boxes for storage until required in the testing work. The number of individual rock particles in each 100 g lot was counted. Ten samples of each aggregate under study was prepared in this manner. The 10 samples were used, in a random manner, in performing these tests: 2 specific gravity and absorption tests, 2 static immersion stripping tests, 2 dynamic immersion stripping tests, and 2 surface reaction stripping tests. The remaining 2 samples were held in reserve.

Coating

The aggregate and asphalt cement were heated to 250 F prior to the coating operation. To each of the 100 g samples of aggregate, 6 g of asphalt was added. The mixture was stirred and manipulated with a spatula until each rock was coated with asphalt. A hot plate was used to heat the mixture, as required to achieve 100% coating. About 3 minutes of hand mixing time was ordinarily required. The gravel aggregate samples (Asher, Broken Bow, Gore and Hugo) were much easier to coat with asphalt than the crushed aggregate samples.

The individual particles of asphalt coated rock were placed in a pan of cold distilled water after mixing. Cold water was necessary to prevent the coated rocks from sticking together.

Static Immersion Stripping Test

The sample preparation and coating procedure as given above follows the standard method of test for coating and stripping of bitumen-aggregate mixtures, ASTM Designation: D 1664 (1). After cooling in the chilled water the coated sample was placed in a glass jar and covered with 600 ml of distilled water. The jar was capped and placed, partially submerged, in a 77 F water bath, and left undisturbed for 18 hours.

The amount of stripping was then visually estimated, using the ASTM standard procedure. To facilitate this evaluation, a comparison graph or chart was prepared. This chart was prepared by tracing the outline of typical aggregate particles inside a circle the same diameter as the glass jar in which the samples were immersed. The cross sectional areas of the aggregate particles were darkened to represent different amounts of coated surface. This comparison chart is shown in Figure 7.

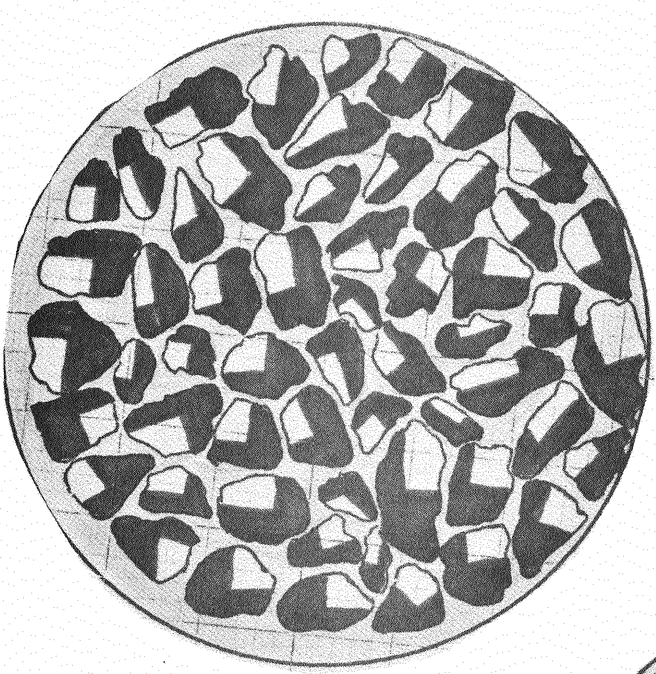
No stripping of any of the various aggregates was observed when coated with asphalt cement and subjected to the static immersion stripping test at 77 F. With a longer period of immersion or higher immersion temperatures, it was anticipated that some stripping of the aggregates would occur. Therefore, the samples were then placed in a 140 F water bath and left undisturbed for 18 hours. The amount of stripping (which was considerable) was then visually estimated, using the comparison chart of Figure 7. This test method was designated the static immersion stripping test at 140 F.

Dynamic Immersion Stripping Test

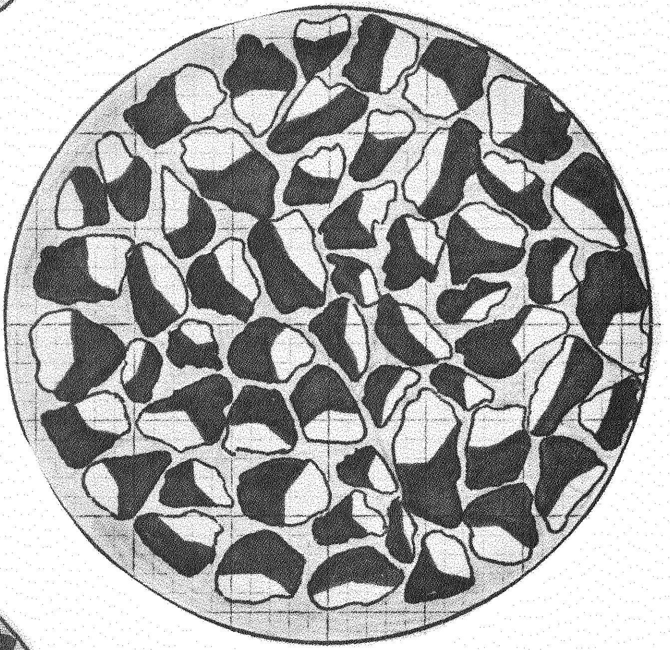
In order to accelerate the stripping action of water on coated aggregate a dynamic stripping device was constructed. The literature review showed many previous investigators had used a dynamic immersion stripping (DIS) test to evaluate the effects of water on asphalt coated aggregate. The method originally used by Nicholson (15) was followed in this study.

An apparatus was designed and built to hold six glass jars of approximately one quart capacity. The device was rotated about a horizontal axis at about 40 rpm. This caused the coated aggregate sample to fall from one end of the jar through the water to the other end during each revolution. A photograph of this dynamic stripping device is shown in Figure 8.

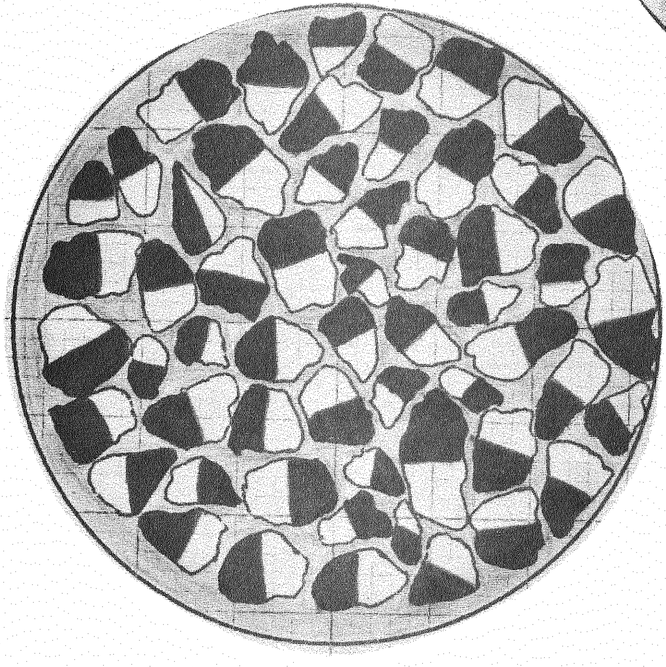
Preliminary tests using the DIS device revealed that the non-stripping aggregate (Cooperton limestone) would partially strip when



(d) 70 Per Cent Coated



(e) 60 Per Cent Coated



(f) 50 Per Cent Coated

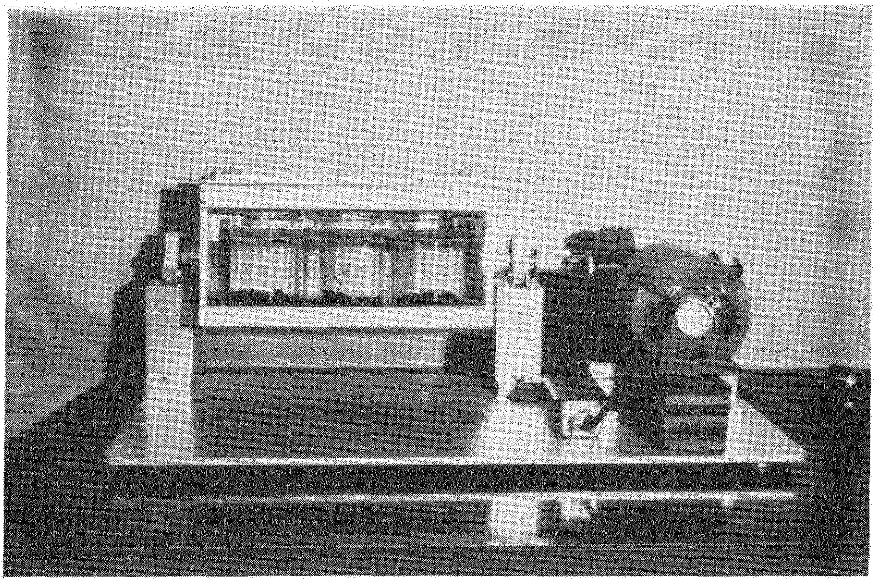


Figure 8. Dynamic Immersion Stripping Machine

the sample was tumbled continuously for 4 hours. The hydrophilic (siliceous) aggregate particles, also retained more than 50 per cent of their coating at the end of 4 hours of tumbling. Therefore, a 4 hour DIS test period was chosen, with the temperature maintained at about 68 F, which was the normal laboratory temperature.

The dynamic immersion stripping test procedure involved coating the aggregate with asphalt cement as was done for the static immersion test, then subjecting the coated aggregate particles to 4 hours of water agitation. The visual estimate of the amount of stripping was made at the end of 1 hour, 2 hours, and 4 hours. This evaluation method was similar to that used in the static immersion test, using the comparison chart of Figure 7.

It is noted that the intended use of the dynamic immersion stripping test was to subject each aggregate sample to the same effect of agitation in water, and then to compare the visually estimated relative stripping tendency of each different type of aggregate. The same stripped aggregate sample was used in the Surface Reaction Test, where a more quantitative evaluation of the amount of stripping that occurred was obtained.

Surface Reaction Test

The objective of this test was to evaluate in a more quantitative manner the amount of surface stripping undergone by an asphalt coated aggregate sample. The technique employed was based on the following hypotheses:

1) The exposed surface area of the aggregate is proportional to the change in gas pressure resulting from the reaction between a suitable reagent and the aggregate surface.

2) An asphalt cement coated aggregate will not react with the reagent and create a significant pressure.

Development of the surface reaction test (SRT) required: a suitable reagent, equipment to measure the reaction gas pressure and temperature, and a suitable test procedure.

Reagents

A suitable reagent for this work is defined as one that, when placed in intimate contact with an aggregate surface, will cause a chemical reaction creating a measurable gas pressure. A reagent strength was desired such that the resulting chemical reaction would not be so violent as to deeply etch the aggregate surface.

Preliminary laboratory work indicated that the calcium carbonate ($CaCO_3$) in the limestone would react in the desired manner using about 1.0 normal hydrochloric acid (HCl). A 100 g sample of aggregate, when reacted with 200 ml of 1.0 normal HCl acid solution would create between 4 and 10 psi of gas pressure. The acid solution would be depleted in about 10 minutes of reaction time at room temperature. Carbon dioxide (CO_2) is the gas generated in this reaction.

For aggregate composed mainly of silicon dioxide (SiO_2) the reagent required to obtain a measurable gas pressure was concentrated hydrofluoric acid (HF). The reaction creates noxious silicatetrafluoride gas (SiF_4). This acid and gas are highly toxic to humans and must be handled very carefully. All work with hydrofluoric acid was carried out in a well ventilated fume hood, using appropriate safety equipment. Preliminary work indicated that although the SiF_4 pressure was small, it was of sufficient magnitude to be measured.

Mixed composition aggregates are those containing appreciable amounts of both $CaCO_3$ and SiO_2, and other constituents. The reagent desired was one that would react with both types of chemical compounds. Preliminary work indicated that a mixture of HF and HCl would create a measurable gas pressure when reacted with aggregate of mixed composition. A 200 ml acid solution was composed of: 27 ml concentrated hydrofluoric acid, 54 ml concentrated hydrochloric acid, and 119 ml of distilled water.

Equipment

This method of test required the measurement of the gas pressure generated when an aggregate sample is inundate with a suitable acid. Since the temperature of the reaction affects the volume of the gas, it was necessary to measure and record simultaneously the pressure and temperature involved in the reaction.

The device developed to accomplish this gas pressure and temperature measurement consisted of modifying a six quart stainless steel pressure cooker, and equipping it with suitable instrumentation to measure and record simultaneously the pressure and temperature.

A Sargent dual-arm recorder (Model DSRG) was used for recording both temperature and pressure in the pressure container (see Figures 9 and 10). A strain-gage type pressure transducer and a thermistor were mounted on the lid of the container and connected to the recording instrument. With this equipment, pressure in the vessel could be determined to the nearest 0.025 psig and temperature to the nearest 0.5 C.

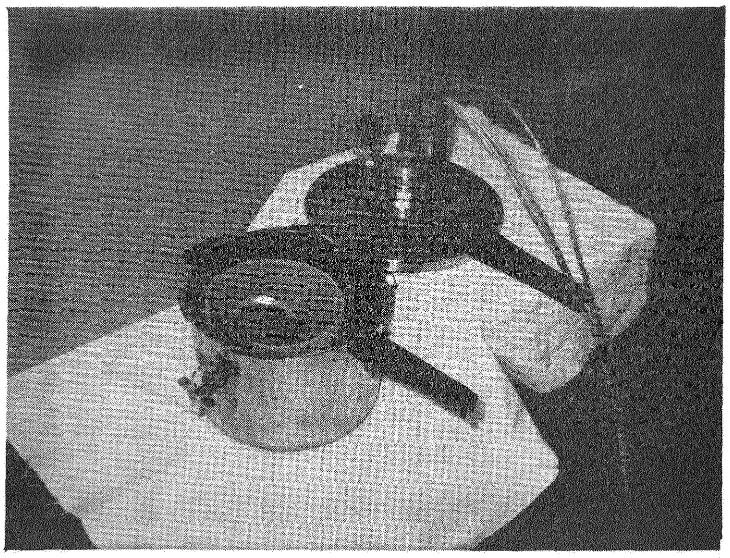


Figure 9. Pressure Container Device

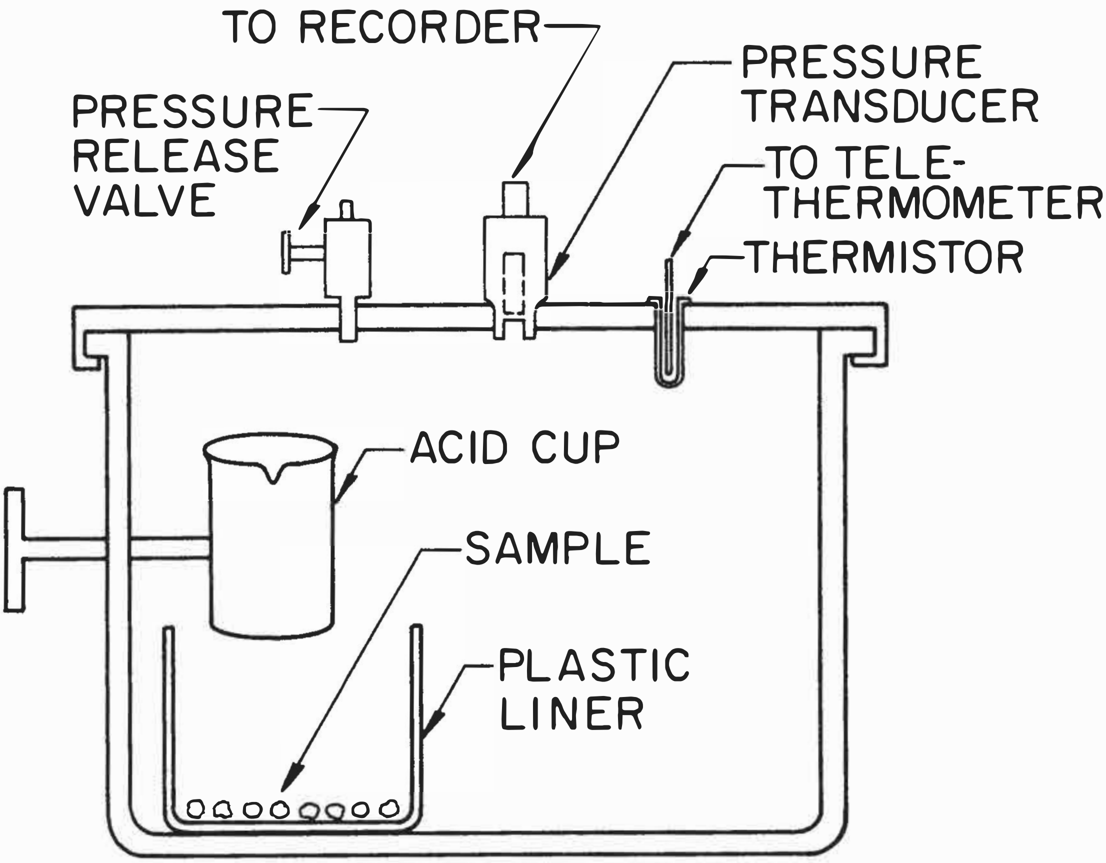


Figure 10. Details of Pressure Container Device

Test Procedure

The aggregate to be evaluated was prepared in duplicate samples. Half of the aggregate samples had been previously coated with asphalt cement and partially stripped in the DIS test. An uncoated duplicate sample of the aggregate was immersed in distilled water at the same time as the DIS specimen was immersed. At the end of the DIS test, the partially stripped and uncoated samples were dried by blotting with paper towels and spread out in pans to air dry. The samples were air dried approximately 24 hours before initiating the surface reaction test.

It was desirable to perform this test on oven dry materials, however, when the partially stripped aggregate samples were oven dried at 212 F, the remaining asphalt cement diffused and completely recoated the stripped aggregate surfaces. Because of this recoating tendency, oven drying was eliminated and the samples were simply air dried before testing.

A liter of acid solution was prepared for use in testing each different type aggregate. Before initiating a test, samples to be tested, acid solutions, and the pressure vessel were placed in a fume hood and brought to a constant temperature of 68 F.

The remote sensing thermometer unit, recorder and pressure vessel are shown in Figure 11a. The pressure vessel had been removed from the fume hood for the photograph. The photograph of Figure 11b contains a typical pressure-time curve for an uncoated limestone aggregate sample.

Briefly, the steps in the SRT were as follows:

1) The sample was placed in the pressure vessel, in a plastic container, positioned beneath the acid beaker.

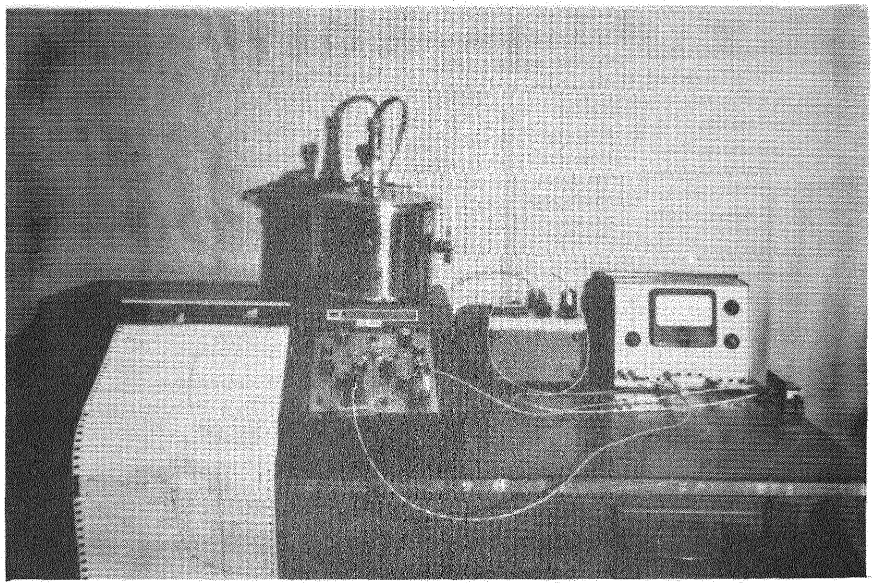


Figure 11a. View of Recorder, Tele-Thermometer and Pressure Container Device

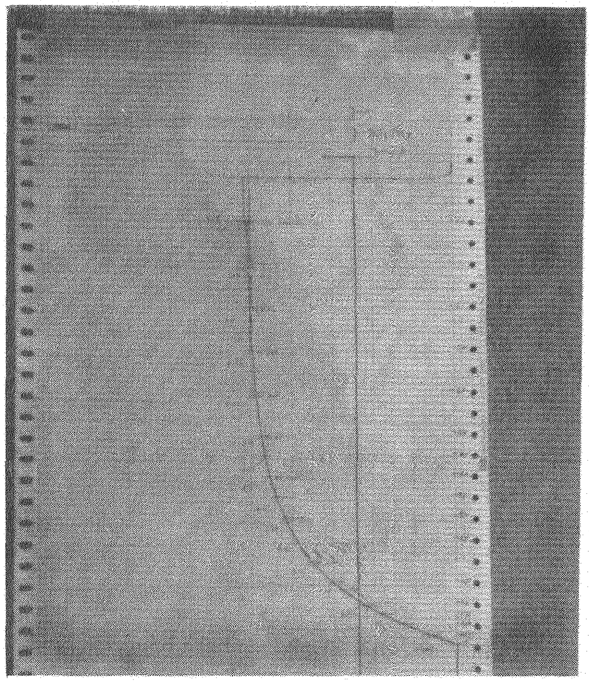


Figure 11b. View of Typical Strip-Chart Recorder Pressure Curve for Uncoated Limestone

2) The 200 ml of acid solution was poured into the beaker.

3) After installing the pressure vessel lid, the pressure recording pen was set to zero, and the recorder chart drive started.

4) Using the external handle, the acid beaker was rotated, pouring the acid solution onto the aggregate sample, and the reaction was monitored by observing the pressure and temperature traces on the recorder.

5) At the completion of the test, which ordinarily lasted 5 minutes, the recorder chart drive was stopped. The pressure was released and the sample was removed from the pressure vessel. The reaction of the acid solution was terminated. The plastic container and acid beaker were then washed and dried prior to initiating another test.

Stripping Evaluation

The pressure-temperature curves plotted on the recorder chart were then analyzed. A horizontal line was drawn on the chart paper for each 15 seconds of elasped reaction time. The pressure and temperature readings were scaled from the chart paper and tabulated. Pressures were adjusted to 68 F for comparative stripping calculations. This adjustment of pressures was necessary due to slightly different operating temperatures and some reactions which created higher temperatures than others.

The change in gas pressure, ΔP, during a stated interval of reaction time was assumed proportional to the exposed aggregate surface area. The ΔP of the uncoated aggregate was compared to the ΔP of the partially coated aggregate and the amount of stripping evaluated. For example, say the uncoated aggregate sample created a ΔP equal to 2.00 psi and the partially coated aggregate sample created a ΔP equal to 0.50 psi, then the amount of surface stripped would be 25%.

Fractured Faces Test

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 (11), was slightly modified. The aggregate was sieved into three sizes, 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 insoluble residue combinations of the four gravel aggregates. These average values are shown in Chapter V, Results and Discussion.

Aggregate Specific Gravity Tests

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

Bulk Impregnated Specific Gravity

Ricketts et al (16) 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:

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

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 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 U.S. Army Corps of Engineers (17) and the Oklahoma Highway Department. The procedure outlined in test method OHD-L-7 (11) was used without modification. The bulk impregnated specific gravities of the blended aggregates used in this study are listed in Chapter V, Results and Discussion.

Bulk Specific Gravity

The bulk specific gravity and water absorption of each individual type of aggregate was determined using a procedure outlined by Manke (18). This method is a modification of the test procedure outlined in ASTM Designation: C127 and C128 (19) to determine the bulk and apparent specific gravity and the water absorption of that aggregate passing the 3/4 in. sieve but retained on the No. 80 sieve, and the apparent specific gravity of the minus No. 80 plus No. 200 sieve size material.

An average bulk specific gravity 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 of the combined sizes of Cooperton Limestone

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

G_A = average bulk specific gravity 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.

Maximum Specific Gravity Test (Uncompacted Mix)

In 1956, Rice (20) described a procedure for determining the maximum specific gravity of a voidless sample of bituminous paving mixture. In this procedure, loose, uncompacted asphalt-aggregate mixture was placed in a calibrated volumetric flask containing enough deaired distilled water containing 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) (1). The procedure followed in this study was identical with that of the standard ASTM test. The measured maximum specific gravity values of the respective mixtures were used to compute the percent densities of the compacted specimens. Percent density values of these specimens were also calculated using the specific gravities of the aggregate blends determined by the bulk specific gravity and bulk impregnated specific gravity test procedures.

CHAPTER V

RESULTS AND DISCUSSION

The purpose of this study was to investigate the effects of incorporating certain percentages of a variety of siliceous aggregates into a "standard" asphalt concrete surface course mixture. The results of the various test procedures employed are presented in tabular form. In some instances, graphical portrayals of the data have been made for comparative purposes and these have been incorporated in Appendices.

Stability and Cohesion Tests

Table VIII shows the stabilometer value, the cohesiometer value, and the percent density of the compacted specimens for each of the aggregates combinations at the selected optimum asphalt content. The aggregate combinations are presented in their order of testing and not grouped as to type of incorporated siliceous aggregate.

As previously mentioned in Chapter III, the selection of the exact mid-point gradation of the Type B specification limits resulted in a very dense aggregate mixture. Results of the voids in the mineral aggregate (VMA) calculations indicated values ranging from 10.8% to 13.6% in these mixtures. To obtain stability values above 35, the specified asphalt content range was lowered. Optimum asphalt contents range from 4 to 4 1/2% asphalt by weight of total mix.

TABLE VIII

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	38	187	96.2
	40	4.25	37	164	96.2
Miami Chert	20	4.50	39	163	96.5
	30	4.50	40	167	97.0
	40	4.50	38	185	96.7
Onapa Sandstone	20	4.50	42	200	96.5
	30	4.50	41	180	95.7
	40	4.50	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	266	96.7
	40	4.25	38	204	96.6
Broken Bow Gravel	20	4.00	38	190	96.5
	30	4.00	35	307	98.2
	40	4.00	40	296	97.5
Gore Gravel	20	4.00	40	311	97.6
	30	4.00	36	302	97.0
	40	4.00	41	323	97.7
Hugo Chert Gravel	20	4.00	38	330	97.7
	30	4.00	36	278	97.9
	40	4.00	37	301	97.8
Keota Sandstone	20	4.00	44	275	97.0
	30	4.00	44	343	96.6
	40	4.00	42	394	97.6

Some of the mixtures, i.e., the mixes containing Keota sandstone and gravels from Hugo, Gore and Broken Bow, exhibited critical tendencies in that slight increases in asphalt content resulted in marked decreases in stability (see graphs in Appendix B). In these mixtures, no definite peak of the stability versus asphalt content curve could be established. Adjustment of the gradation to coarsen it and increase the voids would improve these mixtures and make them less critical.

Realizing that particle surface texture, particle shape, and the mineralogical composition of the particles are important factors contributing to interparticle friction and thus the load supporting capacity of an aggregate mass, it is reasonable to expect a considerable variation in the stability behavior of the respective aggregate mixtures and also different results between mixtures containing various percentages of the same type of siliceous aggregate. Generally, however, the trends of the stability curves for a given type of aggregate were similar and the stability values at the selected optimum asphalt content were quite comparable.

From the stability standpoint, no particular advantage can be assigned to a specific percentage of siliceous material in the mixture. With proper adjustment of the mixture gradation (within the specification limits) it would be possible to achieve adequate stabilities at higher asphalt contents for all mixtures containing up to 40% of the various siliceous aggregates. In fact, the limestone aggregate is not an essential ingredient for good stability. However, considerations other than stability, i.e., durability or stripping resistance, will no doubt govern in the case of mixes containing large percentages of the siliceous aggregates.

The "proper adjustment" of the mixture gradation, mentioned above, would include increasing the percentage of coarse aggregate (plus No. 10 sizes) at the expense of the finer aggregate sizes. By slightly coarsening the mixture, the VMA would be increased and a higher asphalt content could be employed. Also, the substitution of crushed screenings (limestone or siliceous aggregate) for a considerable portion of the rounded river sand used in these mixes could substantially improve their stability values.

The tensile strength or cohesive resistance of a compacted asphalt-aggregate mixture is predominantly influenced by the inherent cohesive properties of the asphalt binder. Adhesive forces developed at the asphalt-aggregate interfaces contribute to a lesser extent to this cohesive strength. The results of the cohesiometer tests failed to indicate any significant effects on this property due to incorporating varying percentages of siliceous aggregates in the mixture. In all cases, the values were well above the recommended minimum cohesiometer value of 50, even at extremely low asphalt contents (see graphs in Appendix C). Generally, the trend of these graphs for a particular aggregate was quite similar. Some divergence was noted in the plots for the Cyril, Keota, and Onapa sandstones and the Asher, Broken Bow and Hugo gravels.

The cohesiometer test proved to be very sensitive to operator technique in that different operators obtained widely different cohesiometer values for like specimens. As shown in Table VIII, the magnitude of the test results increased from top to bottom or toward the latter part of the study. 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.

Immersion-Compression Test

The results of the immersion-compression test on the mixtures remolded by the modified procedure are presented in Table IX. Molded specimens used in the stabilometer and cohesiometer tests were heated, broken down, and recompacted into specimens for this series of tests.

Retained strengths of the various blends ranged from 78 to 124%. The specimens containing 4 and 4.5% asphalt contents, and larger amounts of voids, showed some loss in retained strength. However, there were some exceptions in the blends, notably the 20% Broken Bow mix. As footnoted in Table IX, some of the wet specimens molded at 5% asphalt content indicated appreciable loss in strength. Examination of the amount of water absorbed by these specimens, during vacuum saturation, indicated they had swelled and absorbed excessive amounts of water. Therefore, the loss in strength of these 5% specimens is not attributed to stripping, but resulted from loss in density.

The overall trend indicated less stripping effects (higher retained strength) as the asphalt content was increased. Further, most of the mixtures indicated greater than 100% retained strength at 5% asphalt content. The reason for this increase in strength, after soaking for 24 hours in a 140 F water bath, has not been fully explained. Other researchers have observed similar results (21, 22, 23, 24). The larger asphalt cement contents result in smaller air voids in the compacted specimens and these voids may be connected by extremely small capillaries. The entry of water into the capillaries, without causing stripping of the asphalt cement, creates a tensile force between particles of coated aggregate which results in a higher indicated ultimate strength of the specimen (24).

TABLE IX

PHYSICAL PROPERTIES AND IMMERSION-COMPRESSION TEST RESULTS FOR MIXTURES REMOLDED BY MODIFIED PROCEDURE

Sample	AC (%)	Water Absorbed % (vacuum sat.)	Bulk Specific Gravity	Dry Str. (psi)	Retained Strength (%)
Cooperton	4	3.6	2.300	322	86
	4 1/2	3.0	2.323	312	92
	5	2.1	2.351	319	96
20% Asher	4	3.8	2.270	407	76
	4 1/2	2.8	2.300	380	101
	5	1.7	2.399	356	104
30% Asher	4	3.6	2.270	409	84
	4 1/2	2.9	2.288	402	102
	5	1.9	2.326	395	106
40% Asher	4	4.0	2.244	336	96
	4 1/2	3.3	2.269	375	98
	5	2.5	2.294	294	100
20% Miami	4	4.2	2.268	340	96
	4 1/2	3.0	2.305	320	106
	5	2.3	2.326	329	110
30% Miami	4	3.8	2.282	381	85
	4 1/2	3.2	2.295	370	95
	5	2.2	2.325	304	102
40% Miami	4	4.1	2.268	312	88
	4 1/2	2.9	2.301	332	93
	5	2.3	2.321	273	104
20% Onapa	4	4.15	2.260	370	78
	4 1/2	3.06	2.293	332	92
	5	2.32	2.321	338	97
30% Onapa	4	3.97	2.267	316	86
	4 1/2	3.49	2.280	294	89
	5	2.46	2.305	256	108
40% Onapa	4	4.30	2.233	291	86
	4 1/2	3.60	2.262	292	96
	5	2.40	2.295	274	101

TABLE IX CONTINUED

Sample	AC (%)	Water Absorbed % (vacuum sat.)	Bulk Specific Gravity	Dry Str. (psi)	Retained Strength (%)
20% Stringtown	4	3.78	2.278	310	83
	4 1/2	2.74	2.312	296	97
	5	2.07	2.335	286	106
30% Stringtown	4	3.99	2.272	324	92
	4 1/2	3.10	2.298	320	94
	5	1.92	2.323	318	98
40% Stringtown	4	3.72	2.266	286	90
	4 1/2	2.78	2.293	264	102
	5	2.41	2.308	242	108
20% Cyril	4	3.46	2.311	299	93
	4 1/2	3.14	2.310	280	96
	5	2.39	2.347	264	100
30% Cyril	4	3.71	2.296	342	78
	4 1/2	2.58	2.333	284	95
	5	2.40	2.338	339	98
40% Cyril	4	4.07	2.291	298	86
	4 1/2	2.92	2.326	277	97
	5	2.36	2.336	278	106
20% Broken Bow	4	3.41	2.306	238	117
	4 1/2	2.44	2.333	219	120
	5	1.52	2.362	193	124
30% Broken Bow	4	2.90	2.333	276	96
	4 1/2	2.29	2.358	262	105
	5	2.82	2.376	253	75[1]
40% Broken Bow	4	3.04	2.326	299	90
	4 1/2	2.12	2.348	266	104
	5	2.76	2.369	251	84[1]

TABLE IX CONTINUED

SAMPLE	AC (%)	Water Absorbed % (vacuum sat.)	Bulk Specific Gravity	Dry Str. (psi)	Retained Strength (%)
20% Gore	4	2.72	2.334	297	103
	4 1/2	2.09	2.352	279	103
	5	2.76	2.361	256	84[1]
30% Gore	4	2.95	2.318	343	90
	4 1/2	2.15	2.348	298	103
	5	2.85	2.363	277	80[1]
40% Gore	4	2.78	2.316	333	95
	4 1/2	2.20	2.338	330	100
	5	2.68	2.360	376	77[1]
20% Hugo	4	2.97	2.342	310	91
	4 1/2	2.04	2.363	266	98
	5	1.68	2.389	257	102
30% Hugo	4	2.72	2.335	330	92
	4 1/2	2.04	2.362	295	94
	5	3.16	2.384	292	73[1]
40% Hugo	4	2.82	2.326	317	96
	4 1/2	2.24	2.345	328	98
	5	1.14	2.371	268	105
20% Keota	4	3.04	2.317	331	88
	4 1/2	2.27	2.346	324	95
	5	2.28	2.358	275	100[1]
30% Keota	4	3.09	2.294	330	84
	4 1/2	2.85	2.318	293	95
	5	2.33	2.342	278	89[1]
40% Keota	4	2.27	2.301	272	94
	4 1/2	2.58	2.320	287	96
	5	2.23	2.335	259	92[1]

[1] Wet Specimen Swelled During Vacuum Saturation.

A similar phenomenon has been observed in tests of saturated soil specimens (25). The cause of an increase in the indicated strength of the specimen is attributed to negative porewater pressure. The increase in strength occurs in a dense, saturated specimen having small capillaries. Under a compressive load, if the specimen tends to swell or expand its volume, the water contained by the small capillaries resist this increase in volume. This negative pore-water pressure has the same effect as an increase in confining pressure on the specimen and results in a higher indicated ultimate strength.

No definite trends with regard to the relative stripping resistance of these blends could be established. Apparently, even when the amounts of siliceous aggregate approached 40% by weight of the coarse aggregate in the specimen any detrimental effects on retained strength were too minor to influence the test results. Therefore, mixtures for each of the ten aggregate types were prepared separately, and compacted by the modified gyratory-shear procedure.

The coarse aggregate portion (plus No. 10 material) of the mixtures was composed entirely of the respective aggregate to be evaluated. The fine aggregate fraction (minus No. 10 material) was the same as that used in the standard mixture. Thereby, effects of the fine aggregate on the immersion-compression retained strengths should be the same for each specimen. For each of the ten aggregates, four specimens were molded at 4 and 5% asphalt content, respectively.

Table X shows the physical properties of these specimens and the results of the I-C tests. The Cooperton limestone specimens (5% asphalt content) had the highest retained strength at 116%, while the lowest retained strength of 66% was evidenced by the Onapa sandstone mixture

TABLE X

PHYSICAL PROPERTIES AND IMMERSION-COMPRESSION TEST RESULTS FOR 100% COARSE SILICEOUS AGGREGATE MIXTURES MOLDED BY MODIFIED PROCEDURE

Sample	AC (%)	Water Absorbed % (vacuum sat.)	Bulk Specific Gravity	Dry Str. (psi)	Retained Strength (%)
Cooperton	4	3.1	2.326	203	114
	5	1.5	2.375	214	116
Stringtown	4	2.3	2.249	246	95
	5	2.2	2.289	248	102
Cyril	4	3.5	2.298	251	88
	5	1.7	2.346	248	106
Keota	4	2.4	2.192	226	80
	5	2.3	2.248	236	97
Onapa	4	5.6	2.144	250	66
	5	2.7	2.194	290	76
Asher	4	3.6	2.189	249	95
	5	1.7	2.237	252	104
Broken Bow	4	3.1	2.286	233	94
	5	1.4	2.337	238	106
Gore	4	2.3	2.238	250	80
	5	1.5	2.282	254	96
Hugo	4	2.2	2.266	182	99
	5	2.2	2.324	186	105
Miami	4	3.0	2.244	235	89
	5	2.2	2.290	241	101

(4% asphalt content). In grouping the aggregates as to type at 4% asphalt content, the "limestones" (Cooperton and Stringtown) averaged 104% retained strength, the "gravels" (Asher, Broken Bow, Gore and Hugo) averaged 92%, and the "sandstones" (Cyril, Keota and Onapa) averaged 78%. The Miami chert value fell between the latter two groupings.

Based on the I-C retained strength data presented in the table, the relative stripping resistance of the aggregates (from excellent to poor) is: 1) Cooperton limestone, 2) Hugo chert gravel, 3) Asher chert gravel, 4) Stringtown siliceous limestone, 5) Broken Bow siliceous gravel, 6) Miami chert, 7) Cyril calcareous sandstone, 8) Keota siliceous sandstone, 9) Gore siliceous gravel, and 10) Onapa siliceous sandstone.

Film Stripping Tests

The film stripping resistance of the various siliceous aggregates were evaluated by the static immersion stripping (SIS) test and the dynamic immersion stripping (DIS) test. The average results of the SIS tests, at both 77 F and 140 F, are reported in Table XI. At 140 F, a visually estimated retained coating of 95% on the Hugo chert gravel was the maximum value obtained. More stripping was evident on the other aggregate samples, with the asphalt cement tending to recede from the sharper edges of the aggregate particles where the film was thinnest. The Gore gravel was the least coated sample with an estimated 60% of its surface area exposed after 18 hours immersion.

Percent retained asphalt coating for each aggregate sample after 1, 2, and 4 hours of DIS testing are also presented in Table XI. Estimated values of retained coating ranged from 85% for the Cooperton limestone, Keota sandstone and Onapa sandstone to 65% for the Gore gravel.

TABLE XI

RESULTS OF STATIC IMMERSION AND DYNAMIC IMMERSION STRIPPING TESTS

Aggregate	Static Immersion Ret. Coating (%)		Dynamic Immersion Ret. Coating (%)		
	77 F	140 F	1 hr.	2 hr.	4 hr.
Cooperton	100	85	95	90	85
Stringtown	100	65	95	90	85
Cyril	100	60	90	80	75
Keota	100	50	95	90	80
Onapa	100	50	95	90	85
Asher	100	90	95	90	80
Broken Bow	100	90	95	90	70
Gore	100	40	90	85	65
Hugo	100	95	95	90	80
Miami	100	70	95	85	75

Results of the surface reaction tests are shown in Table XII. These are quantitative values of percent retained coating determined on the same samples subjected to the DIS test. The measured aggregate retained coatings ranged from 93% for the Stringtown siliceous limestone to 54% for the Broken Bow gravel. The aggregates classified by type as "limestone" (Cooperton and Stringtown) had the highest group average of 92% retained coating. The "sandstone" aggregates (Cyril, Keota, and Onapa) averaged 63%, while the "gravels" (Asher, Broken Bow, Gore and Hugo) averaged 68%. These results show the same trend in stripping resistance of the aggregates as previously presented in the immersion-compression test results, i.e., the "limestone" were better than "gravels", which in turn were better than "sandstones".

Table XII is also a summary of the immersion-compression, film stripping and surface reaction test results, which were used to develop a relative ranking of the individual aggregates. The 4% immersion-compression retained strengths were adjusted to obtain a relative maximum retained strength of 100%. That is, the 4% asphalt content values shown in Table X were divided by 1.14.

It is realized that these three tests are independent measures of stripping and, while their results are not comparable with regard to units, they do provide some insight as to the relative stripping tendencies of the respective aggregates. The indicated ranking is based on an average of the three test values for each aggregate. In order of their resistance to stripping, the ranking is as follows: 1) Cooperton limestone, 2) Hugo chert gravel, 3) Asher chert gravel, 4) Stringtown siliceous limestone, 5) Broken Bow siliceous gravel, 6) Miami chert, 7) Cyril calcareous sandstone, 8) Keota siliceous sandstone, 9) Onapa

TABLE XII

SUMMARY OF STRIPPING TESTS RESULTS AND AGGREGATE RANKING

Aggregate	Relative[1] Immersion-Compression 4% AC	Static Immersion 18 hr. @ 140 F	Surface Reaction	Average	Rank
	% Ret. Str.	% Ret. Ct.	% Ret. Ct.	%	
Cooperton	100	85	90	91.7	1
Stringtown	83	65	93	80.3	4
Cyril	77	60	64	67.0	7
Keota	70	50	56	58.7	8
Onapa	58	50	68	58.7	9
Asher	83	90	74	82.3	3
Broken Bow	82	90	54	75.3	5
Gore	70	40	65	58.3	10
Hugo	87	95	78	86.7	2
Miami	78	70	60	69.3	6

[1] Relative immersion-compression values based on Cooperton limestone having a retained strength of 100 per cent (Table X).

siliceous sandstone, 10) Gore siliceous gravel. This ranking corresponds to that made solely from immersion-compression test results with the exception that the positions of the Onapa sandstone and Gore gravel have been interchanged.

Fractured Faces Test

Results of the modified fractured faces test on the gravel aggregates used in this study are presented in Table XIII. The average percent crushed particles or fractured faces in each of these gravels was greater than 50%. A weighted average value based on the percentages of these aggregates utilized in the standard mixture was also computed. The Cooperton limestone aggregate in these blends was considered to be 100% crushed material. In all cases the coarse fractions of these gravel aggregate blends had fractured face percentages greater than 80.

Specific Gravity Tests to Determine Specimen Percent Density

While not an essential part of the primary objective of this study it was deemed desirable to compare several standard methods of determining the percent density of compacted asphalt-aggregate specimens. Two of these methods utilized a calculative procedure in which the "theoretical" maximum specific gravity of the mixture was based on an average specific gravity of the combined aggregate. These combined aggregate specific gravities were obtained from the bulk specific gravity and the bulk impregnated specific gravity tests. The third method employed a "measured" maximum specific gravity of the mixture obtained from Rice's test procedure.

TABLE XIII

PERCENT FRACTURED FACES IN GRAVEL AGGREGATES

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 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.

Results of the bulk impregnated and bulk specific gravity tests on the various aggregates and blends are presented on Table XIV and Table XV. Bulk impregnated specific gravity is a function of the ratio of asphalt absorption to water absorption of an aggregate. Because this test procedure attempts to take into account the variable absorptiveness of the aggregate, bulk impregnated specific gravity is a type of "effective" specific gravity. Thus, the values obtained from this test should always be greater than those from the bulk specific gravity test.

However, with the exception of the Cooperton limestone-Arkhola sand mixture and the 20 and 40% Asher gravel blends, the tabulated results show the bulk impregnated gravities to be smaller than the bulk specific gravities. This indicates consistent but erroneous results from one or both of these specific gravity tests. Although not of major significance to the overall project, this discrepancy points out an inaccuracy inherent to percent density determinations when a calculated "theoretical" maximum specific gravity of the mix is used.

The percent density of a compacted asphalt-aggregate mixture varies inversely with the maximum specific gravity of the mixture according to the following relationship:

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

Employing a calculated "theoretical" maximum specific gravity of the mixture, in the above expression 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

TABLE XIV

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 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 XV

AVERAGE BULK SPECIFIC GRAVITIES

	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 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

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. Also, any errors in the percentages and specific gravities of the asphalt and the combined aggregate are carried over and reflected in the percent densities.

On the other hand, basing percent density values on the "measured" maximum specific gravity of the mixture eliminates the inaccuracies ascribed to the use of a calculated "theoretical" maximum specific gravity. The procedure is straightforward and theoretically sound since the mixture tested is truly representative of the actual components in the compacted specimen. In fact, the percent density of a compacted specimen or pavement core sample can be determined without any additional information or testing relative to the constituents of the mix.

The graphs in Appendix D show percent density values obtained by each of the three methods plotted against asphalt content (4 to 5% range) for each of the aggregate blends. To avoid confusion, the plots of percent density values derived using bulk impregnated specific gravities have been designated "Bulk Impregnated Method."

In a number of cases, the "bulk impregnated" densities and the "calculated" densities were above 100%, indicating a negative air void content. Since this is patently impossible, such values are entirely unsatisfactory for mix design purposes. However, they do point out that for realistic values reflecting the actual density of the specimen the respective specific gravity determinations must be made carefully and accurately with all sources of error reduced to a minimum.

The relatively poor density results obtained using bulk impregnated specific gravity values is attributed to operator error and inexperience in performing this particular test. Apparently, all entrapped air was not removed in the stirring operation. However, if this test procedure is to be used with aggregate mixtures containing large percentages of siliceous materials it might be desirable to carefully check the results.

CHAPTER VI

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

Based on the test procedures employed and the materials used in this study, the following conclusions can be made:

1. Incorporating various percentages of acid-insoluble material, i.e., siliceous aggregates, in the standard surface course mixture had little or no detrimental effect on the stability of the respective mixes.
2. The siliceous aggregates had no apparent 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. Selection of the exact mid-point gradation of the Type B specification limits resulted in a very dense-graded blend of aggregate having a low VMA value in the compacted state and relatively low optimum asphalt content. In several instances, particularly in the gravel aggregates, these conditions produced "critical" mixtures. A slight coarsening of the gradation would remedy this situation and permit the use of higher asphalt contents.
4. The modified immersion-compression test can be used to evaluate the stripping tendencies of aggregate mixtures conforming to OHD

surface course specifications. Air void content of compacted asphalt-aggregate mixes greatly influences the results of this test. Obtaining meaningful data requires a reduction of compactive effort in molding the test specimens and some type of vacuum-saturation process to thoroughly soak the molded specimens.

5. The surface reaction test provides a quantitative measure of exposed surface area in a stripped aggregate sample. This test used in conjunction with a dynamic stripping procedure and the static immersion stripping test at 140 F can also be used to determine relative stripping tendencies of surface course aggregates.
6. Of the three methods employed to determine the maximum specific gravity of an asphalt aggregate mixture, Rice's method resulted in more realistic or acceptable values of percent density for the compacted specimens.

Recommendations

In view of the results of this research, the following recommendations are presented:

1. Perform a field evaluation of these same siliceous aggregates to determine their actual performance in an asphalt pavement test section. The validity of the laboratory test results and the relative rankings assigned to the respective aggregates could then be ascertained. Knowledge of how well these aggregates perform under service conditions would be of great assistance in any future work in this area.
2. Investigate the effects of various additives to increase the stripping resistance of these Oklahoma siliceous aggregates. Results of the

immersion-compression test and/or other film stripping procedures should be correlated with the effect of the additives on stability. (Note: A "preliminary" study of this nature is presently under way but due to the large number of additives available and the many sources and types of siliceous aggregates that could be employed a more comprehensive study might be in order.)

3. Further study should be given to the method of determining maximum specific gravity of a paving mixture for mix design purposes. Even when performed by experienced personnel, the bulk impregnated specific gravity test has the proclivity (due to entrapped air) to yield lower than actual aggregate specific gravity values. Such values carry over into the percent density calculations and result in higher densities not representative of those existing in fact. The consequences of this are obvious in relation to specifying and checking the minimum density to be obtained by field compaction.

REFERENCES

(1) American Society of Testing and Materials, 1972 Book of ASTM Standards, Part 11.

(2) Folk, R. L. "Spectral Subdivision of Limestone Types." Classification of Carbonate Rocks, The American Association of Petroleum Geologists, Tulsa, Oklahoma, 1962, pp 62-84.

(3) Dunham, R. J. "Classification of Carbonate Rocks According to Depositional Texture." Classification of Carbonate Rocks, The American Association of Petroleum Geologist, Tulsa, Oklahoma, 1962, pp 108-121.

(4) McCasland, W. "Study of the Wearability and Polish Resistance of Oklahoma's Coarse Aggregates." Research and Development Division, Oklahoma Department of Highways, December, 1972.

(5) Pirsson, L. V. Rocks and Rock Minerals. John Wiley and Sons, Inc., New York, 1908.

(6) McBride, E. F. "A Classification of Common Sandstones." Journal of Sedimentary Petrology, Vol. 33, No. 3, 1963, pp 664-669.

(7) "Standard Descriptive Nomenclature of Constitutents of Natural Mineral Aggregates." Designation: C294, 1972 Book of ASTM Standards, Part 10.

(8) "Standard Specifications for Highway Construction." Oklahoma State Highway Commission, 708.1, 1967, pp 454-458.

(9) Burnett, W. C., Gibson, J. L., and Kearney, F. J., "Skid Resistance of Bituminous Surfaces," Highway Research Board, Research Records No. 236, 1968.

(10) Nichols, F. P., Jr., Dillard, J. H., and Alwood, R. L., "Skid Resistance in Virginia," Highway Research Board, Bulletin No. 139, 1956.

(11) "Laboratory Testing Procedures," Materials Division, State of Oklahoma, Department of Highways, July, 1970.

(12) Texas Highway Department Standard Procedures, "Laboratory Method of Compacting Test Specimens of Bituminous Mixes," Tex-206-F, Tex-207-F, January, 1972.

(13) Pauls, J. T. and Goode, J. F. "Application and Present Status of the Immersion-Compression Test." Proceedings, The Association of Asphalt Paving Technologists, Vol. 16, 1947, pp 373-393.

(14) Manke, P. G. "Evaluation of Various Methods of Soaking Soil-Bitumen Specimens." Unpub. Masters Thesis, Oklahoma State University, 1957.

(15) Nicholson, V. "Adhesion Tension in Asphalt Pavements, Its Significance and Methods Applicable in its Determination." Proceedings, The Association of Asphalt Paving Technologists, Vol. 3, 1932, pp 28-49.

(16) Ricketts, W. C., Sprague, J. C., Tabb, D. D., and McRae, J. L., "An Evaluation of the Specific Gravity of Aggregates for Use in Bituminous Mixtures," ASTM Proceedings, Vol. 54, 1954.

(17) U.S. Army Corps of Engineers, "Engineering and Design of Flexible Airfield Pavements," Engr. Manual 1110-45-302, August, 1958.

(18) Manke, P. G., "Asphalt Mix Design Procedures," Technical Publication #17, School of Civil Engineering, College of Engineering, Oklahoma State University, 1970.

(19) American Society of Testing and Materials, 1972 Book of ASTM Standards, Part 10.

(20) Rice, J. M., "Maximum Specific Gravity of Bituminous Mixtures by Vacuum Saturation Procedure." Presented at Symposium on Specific Gravity of Bituminous Coated Aggregates, ASTM, Special Technical Publication #191, 1956.

(21) Pauls, J. T. and Rex, H. M. "A Test for Determining the Effect of Water on Bituminous Mixtures." Public Roads, Vol. 24, No. 5, July-August-September 1945, pp 115-129.

(22) Swanberg, J. H. and Hindermann, W. L. "The Use of an Abrasion Test as a Measure of Durability of Bituminous Mixtures." American Society for Testing and Materials, STP 94, 1949, pp 67-83.

(23) Goldbeck, A. T. "Immersion-Compression Tests Compared with Laboratory Traffic Tests on Bituminous Concrete." American Society for Testing and Materials, STP 94, 1949, pp 53-66.

(24) Goode, J. F. "Use of the Immersion-Compression Test in Evaluating and Designing Bituminous Paving Mixtures." American Society for Testing and Materials, STP 252, 1959, pp 113-129.

(25) Sowers, G. B. and Sowers, G. F. Introductory Soil Mechanics and Foundations. Second Edition, The Macmillan Company, New York, 1961.

APPENDIX A

PROCEDURE FOR MOLDING ASPHALT-AGGREGATE SPECIMENS

MOLDING PROCEDURE USING MOTORIZED GYRATORY COMPACTION DEVICE

1. The asphalt-aggregate mixture should be compacted at 250 F ± 5 F. The mold and base plate are heated to approximately 250 F prior to molding. Place base plate inside mold and insert a paper disc in the bottom of the mold.
2. Using a funnel and bent spoon, place 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 until a pressure of 50 psi is read on the low pressure gage.
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 gyrate through three complete revolutions.
5. When the mold stops, raise the cam-lever and level the mold. Immediately apply 50 psi pressure with full strokes of the pump handle and continue this procedure until one full stroke of the pump causes the low pressure gage to indicate a pressure of 100 psi or more.
6. When 100 psi is obtained with one full stroke off the pump handle, apply a 2500 psi leveling pressure by continuing to pump the handle at approximately one stroke per second.
7. As soon as the high pressure gage registers 2500 psi, reverse the control valve and release the vertical pressure slowly. Remove the press ram from the mold by pumping the handle to raise 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 HVEEM STABILITY VERSUS ASPHALT CONTENT

Figures 12-21

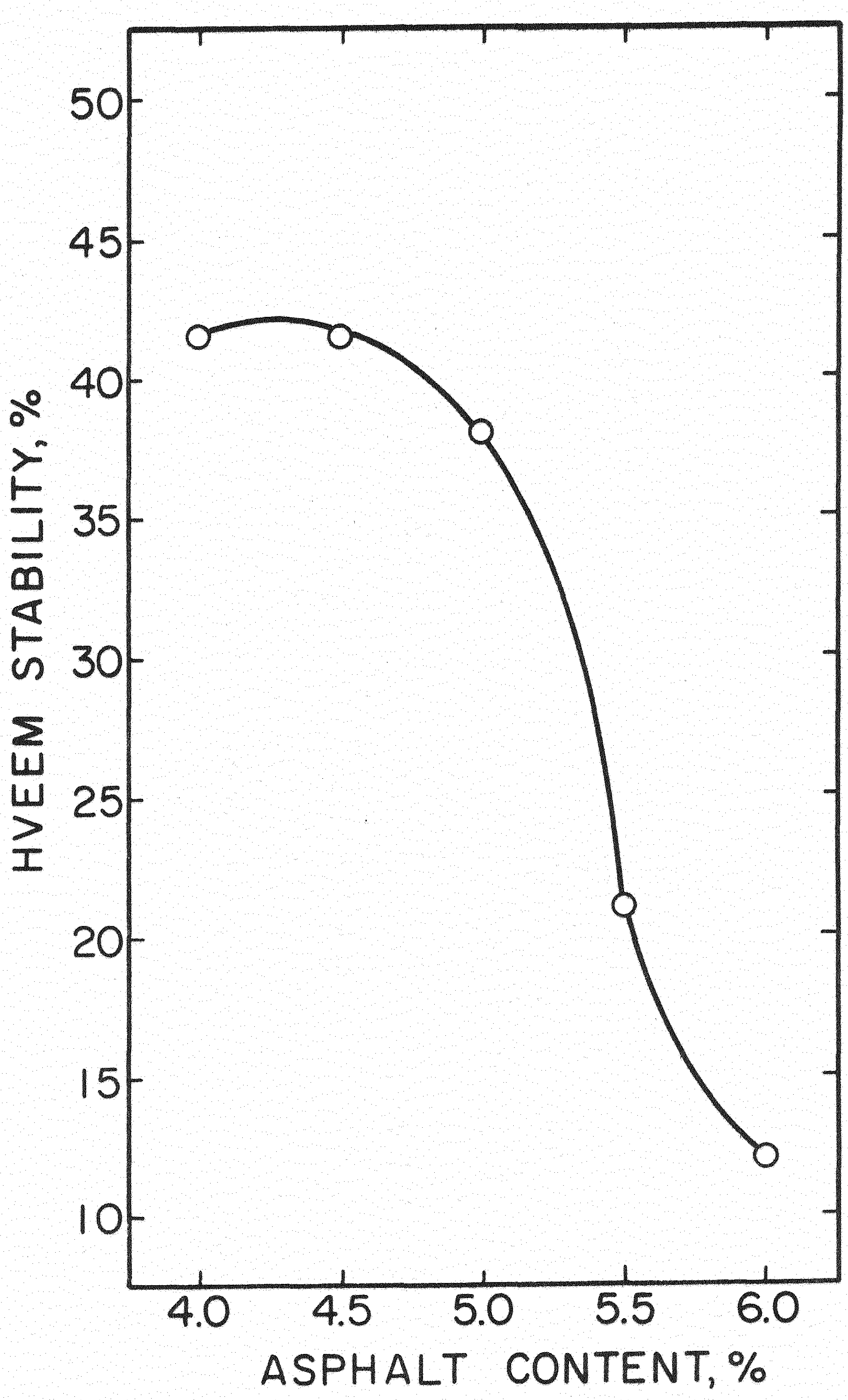


Figure 12. Hveem Stability Versus Asphalt Content
Cooperton Limestone

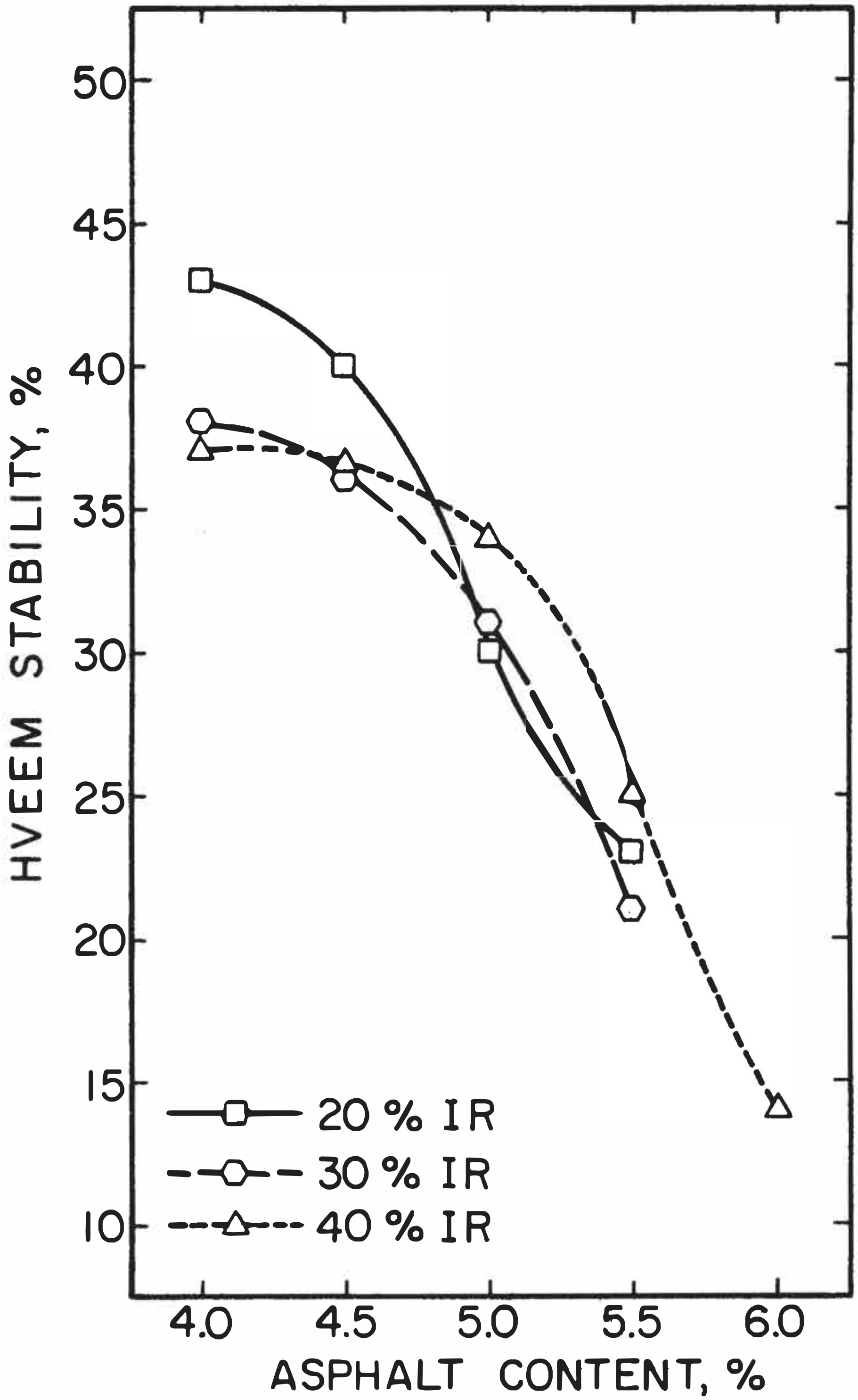


Figure 13. Hveem Stability Versus Asphalt Content
Asher Chert Gravel

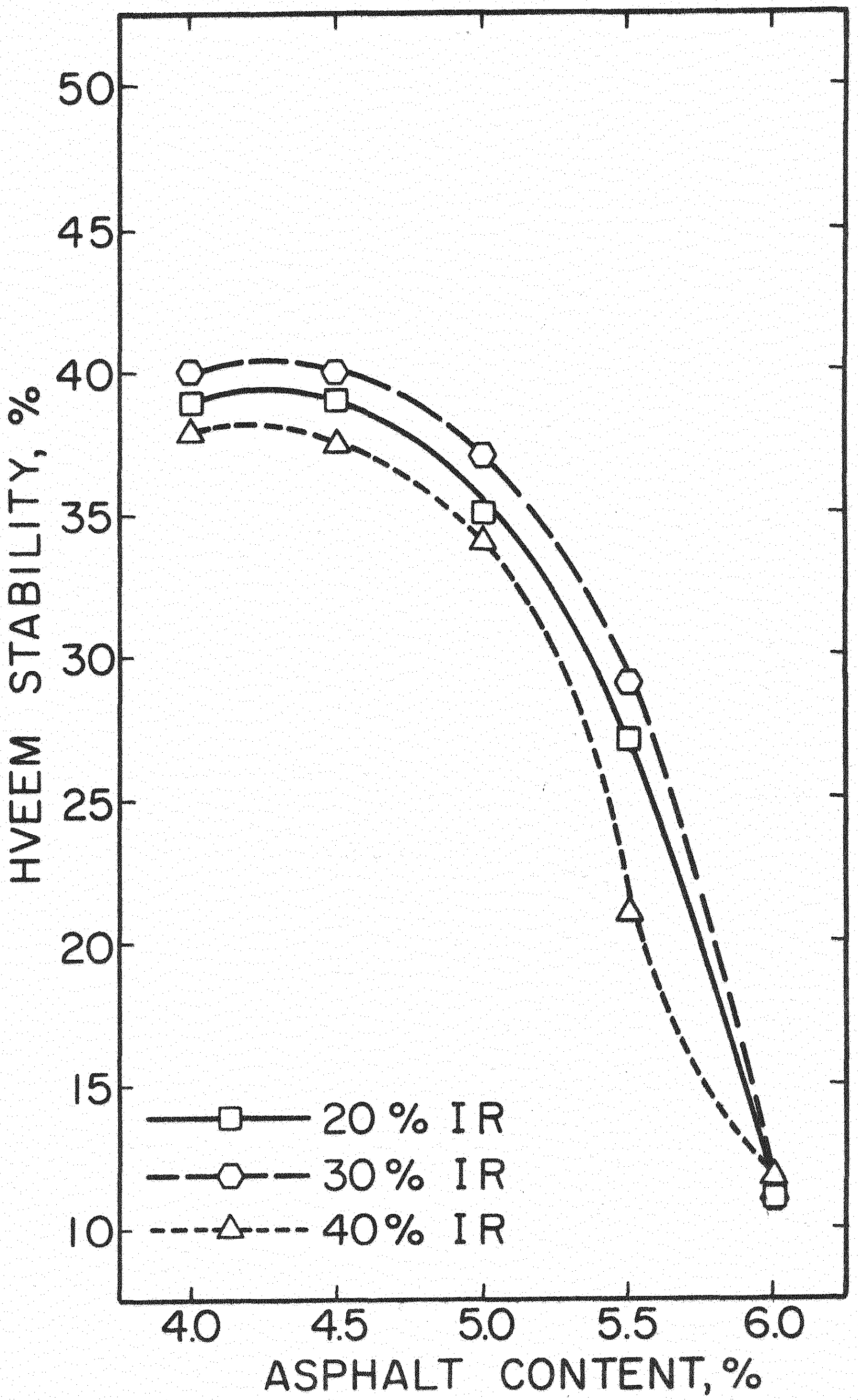


Figure 14. Hveem Stability Versus Asphalt Content
Miami Chert

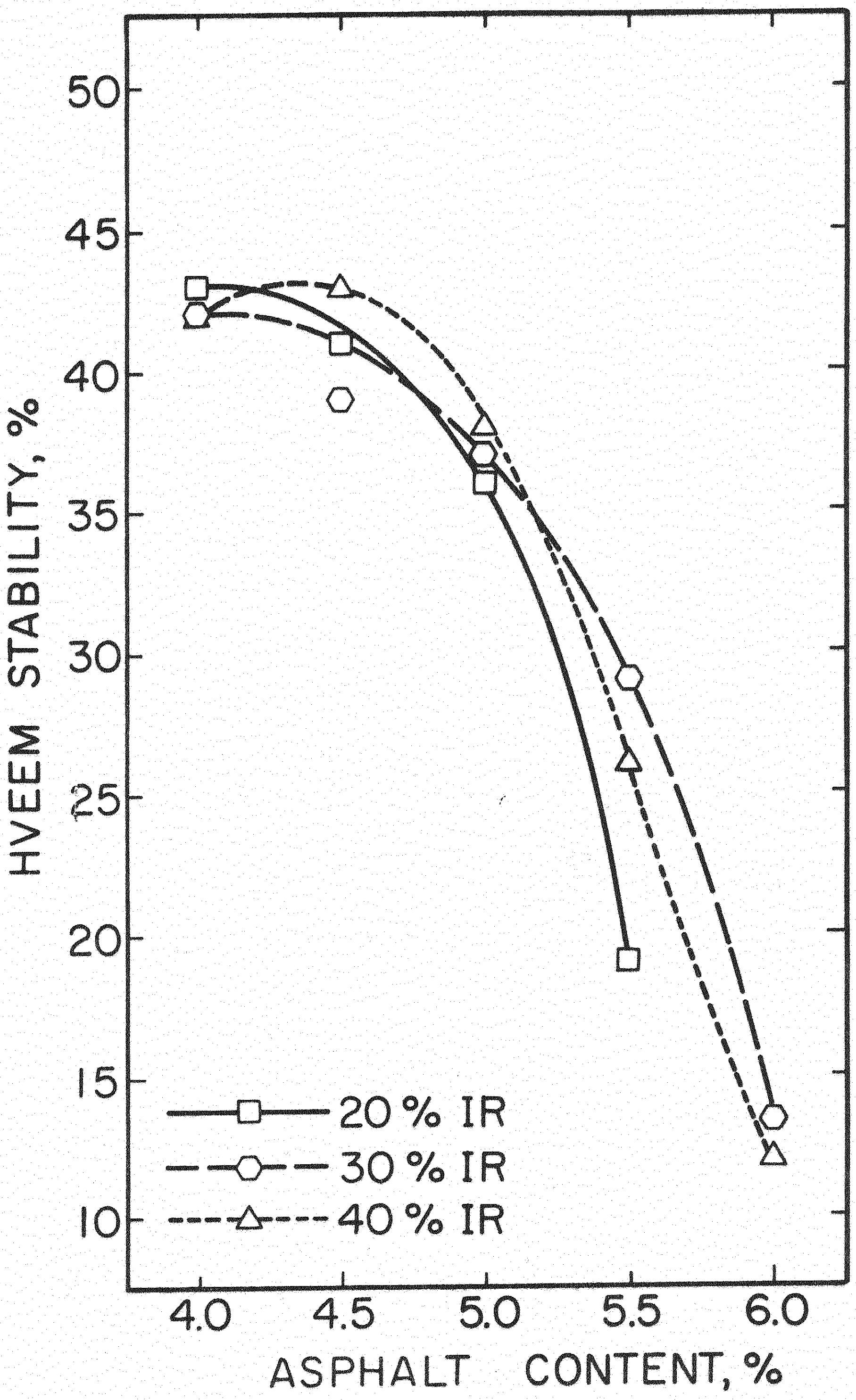


Figure 15. Hveem Stability Versus Asphalt Content
Onapa Sandstone

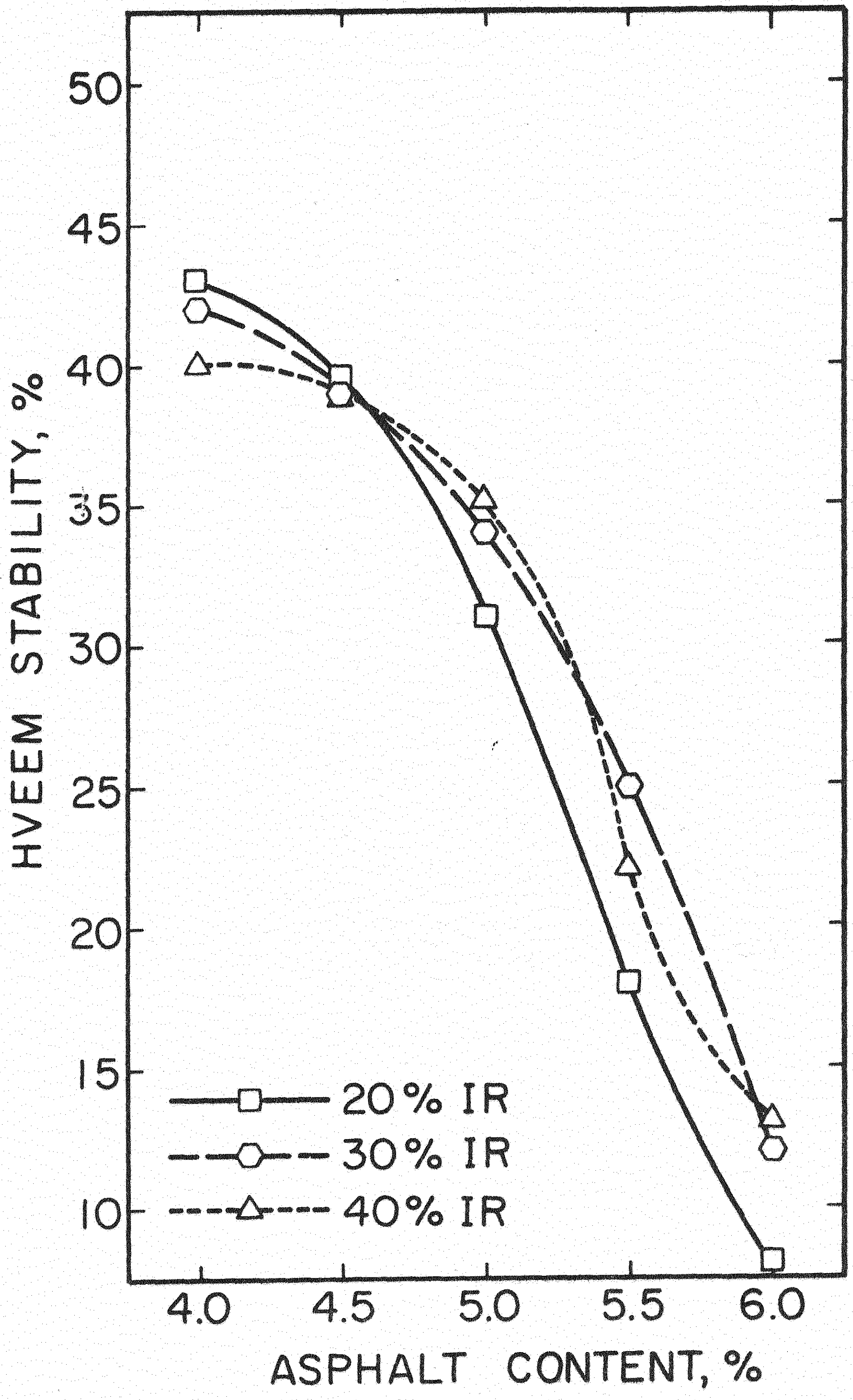


Figure 16. Hveem Stability Versus Asphalt Content
Stringtown Limestone

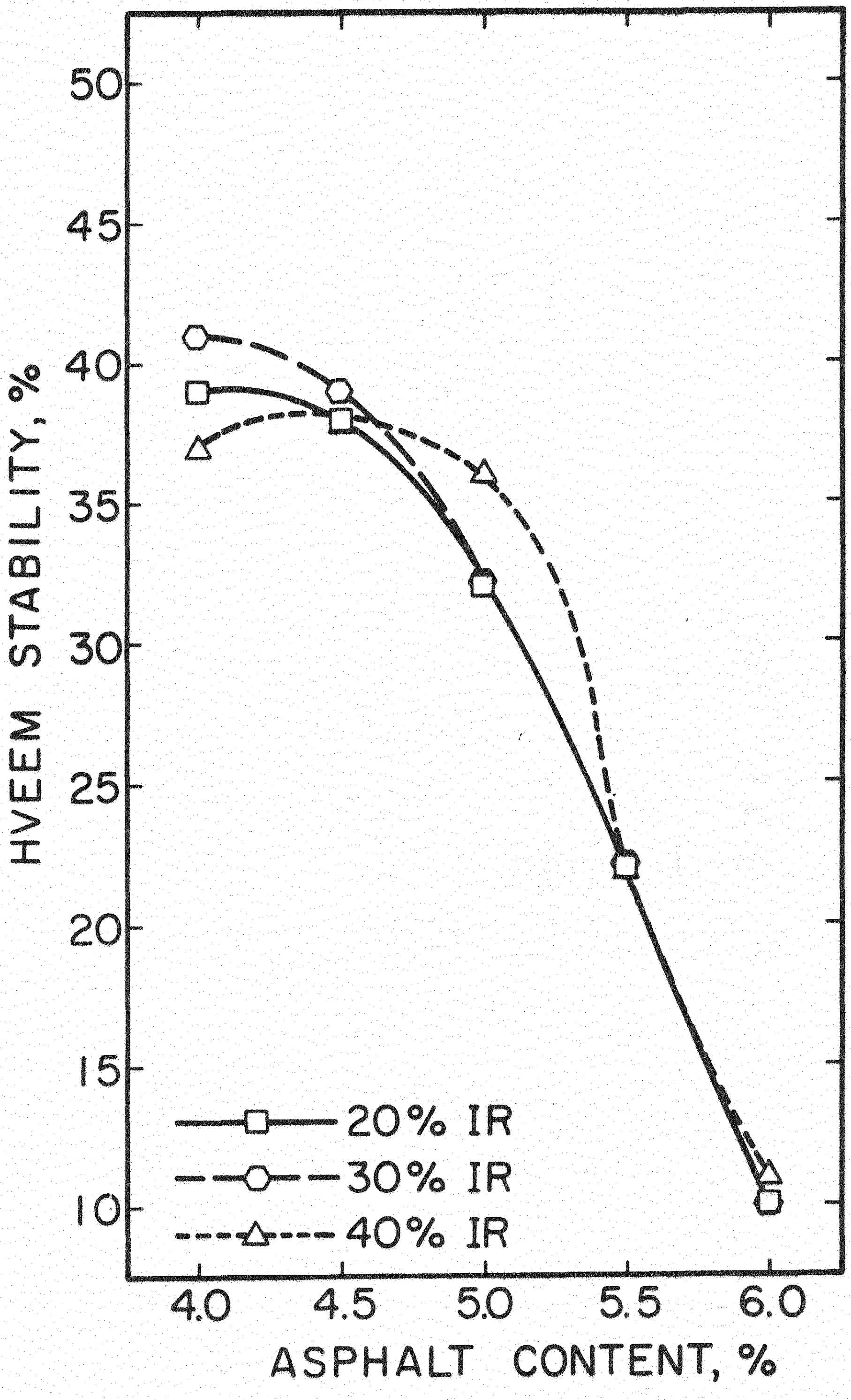


Figure 17. Hveem Stability Versus Asphalt Content
Cyril Sandstone

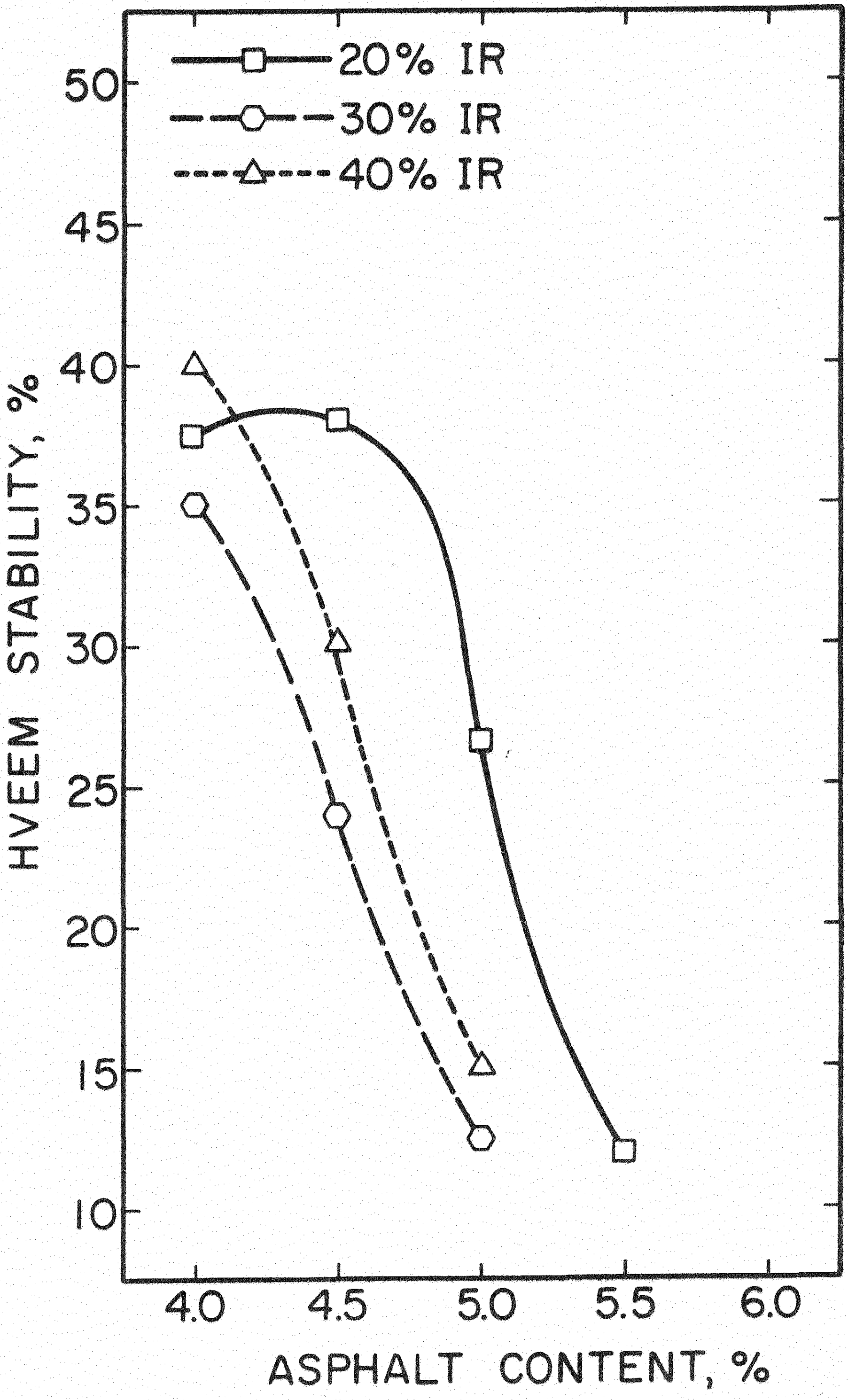


Figure 18. Hveem Stability Versus Asphalt Content
Broken Bow Gravel

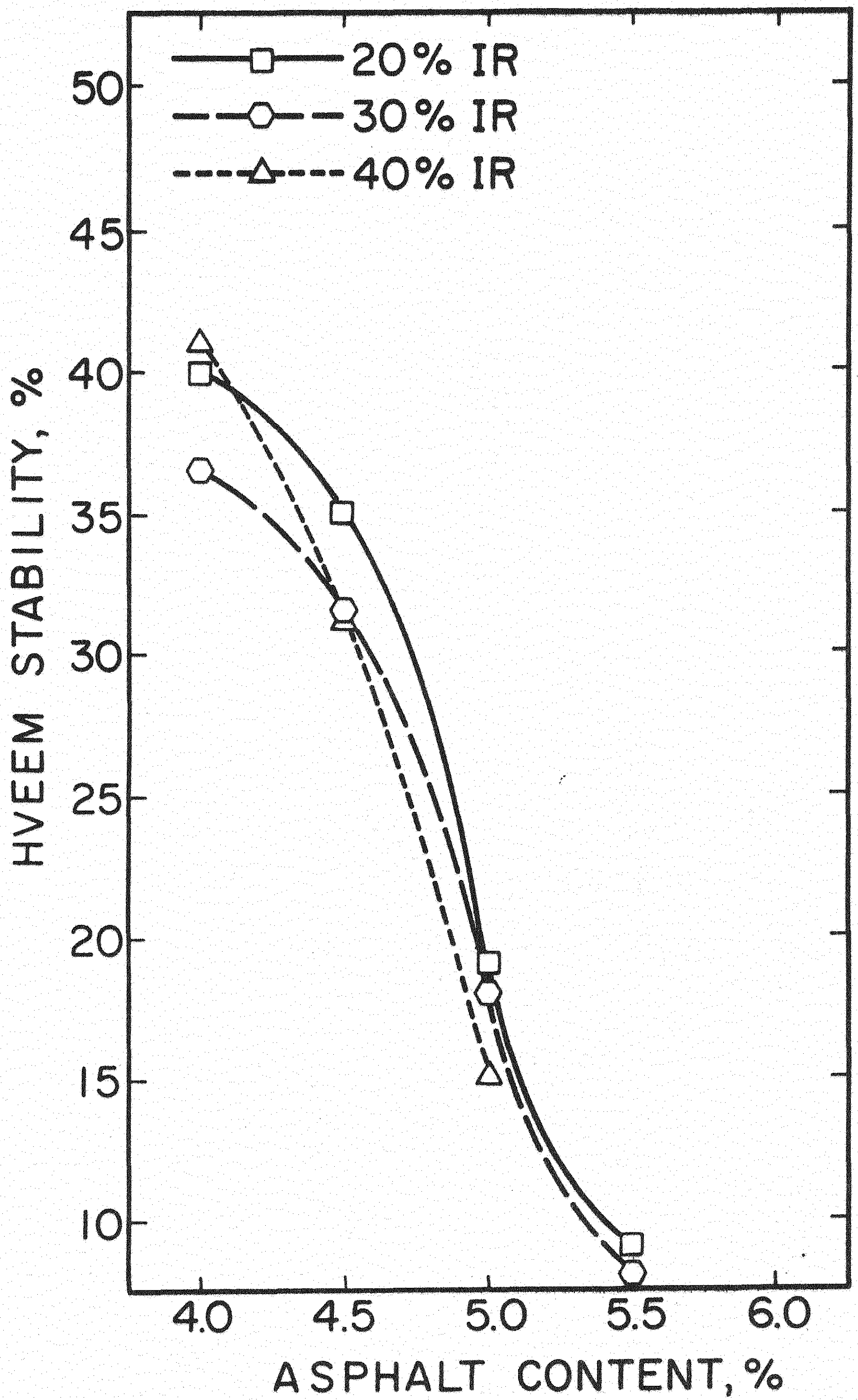


Figure 19. Hveem Stability Versus Asphalt Content
Gore Gravel

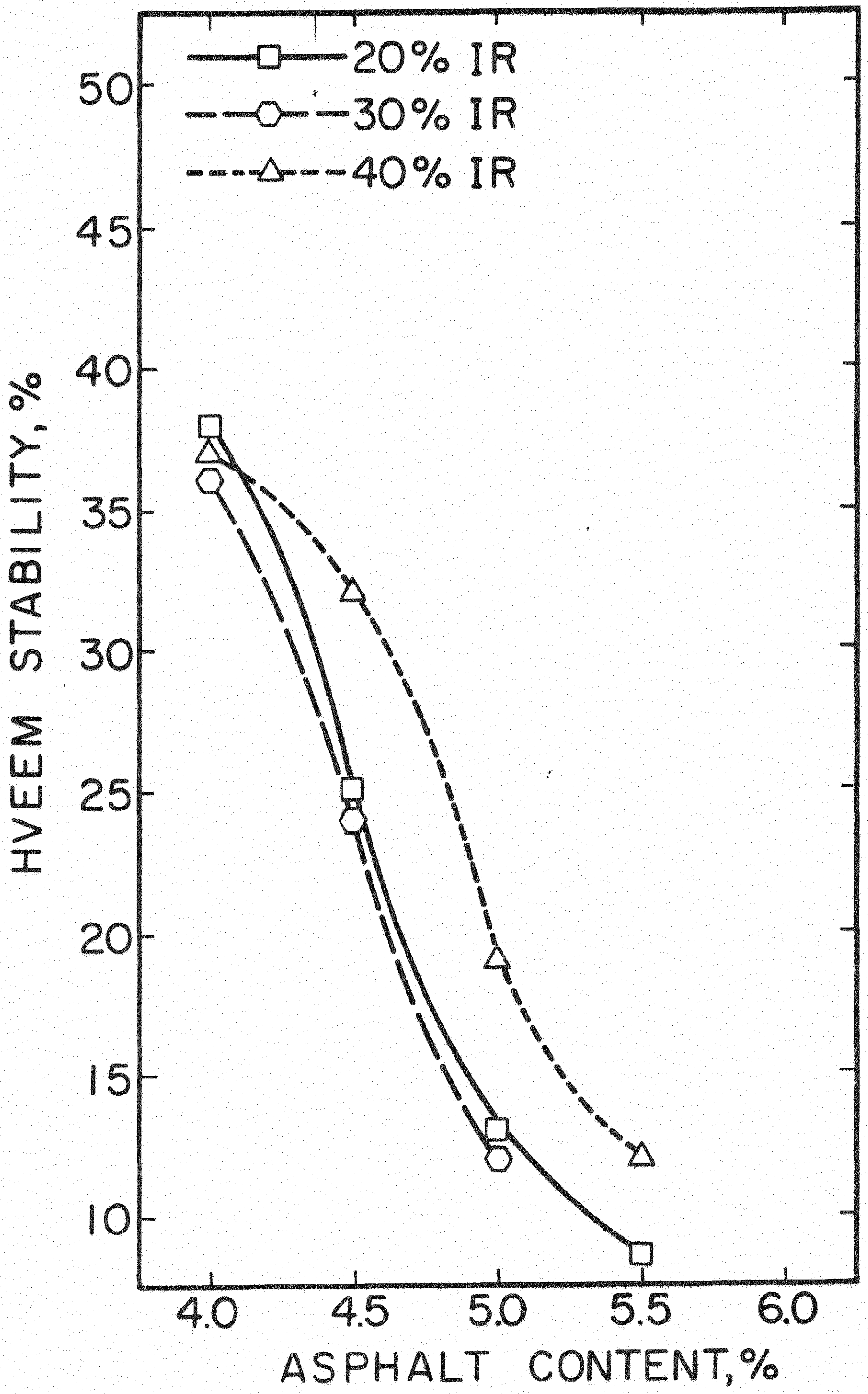


Figure 20. Hveem Stability Versus Asphalt Content
Hugo Chert Gravel

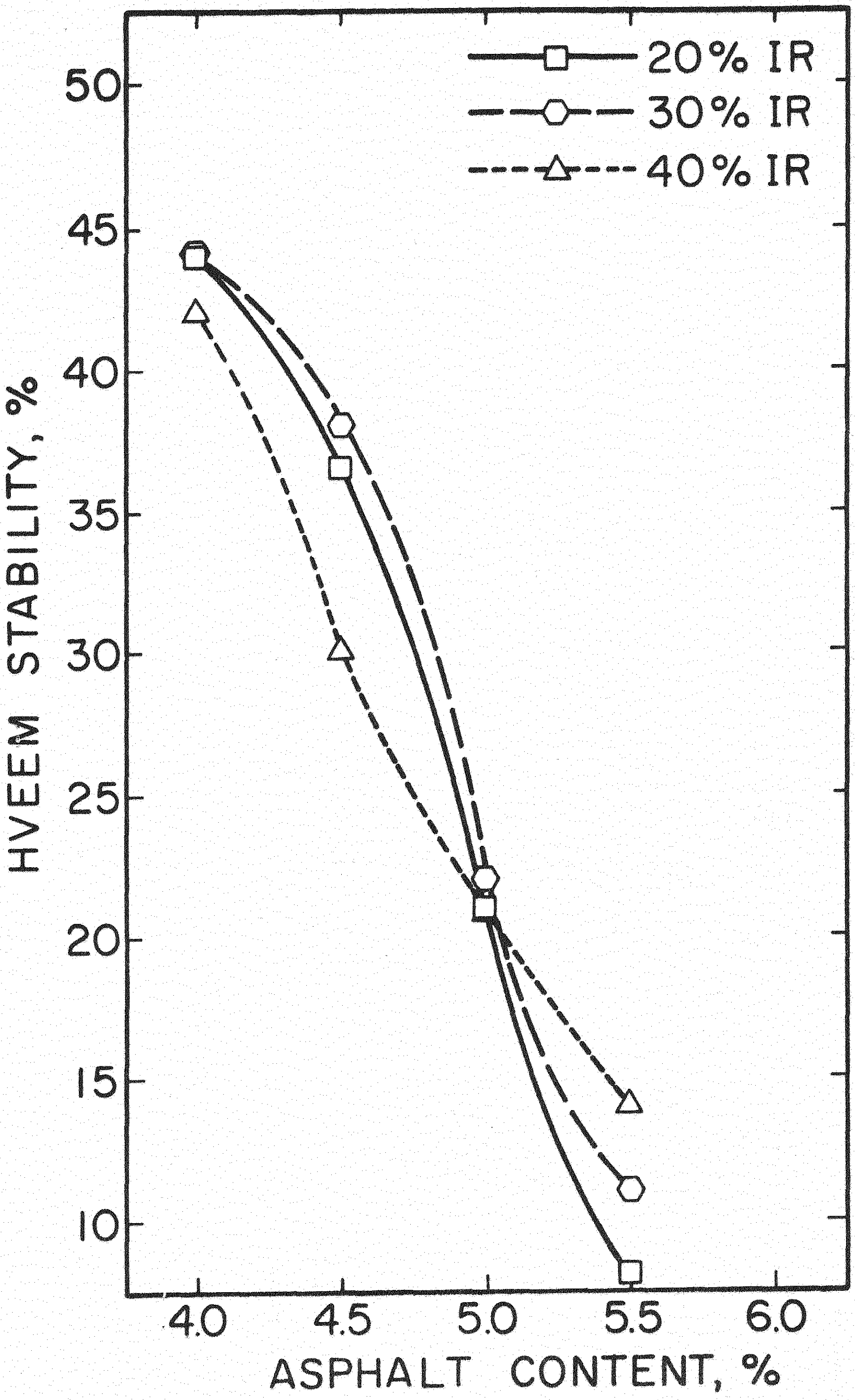


Figure 21. Hveem Stability Versus Asphalt Content
Keota Sandstone

APPENDIX C

PLOTS OF HVEEM COHESION VERSUS ASPHALT CONTENT

Figures 22-31

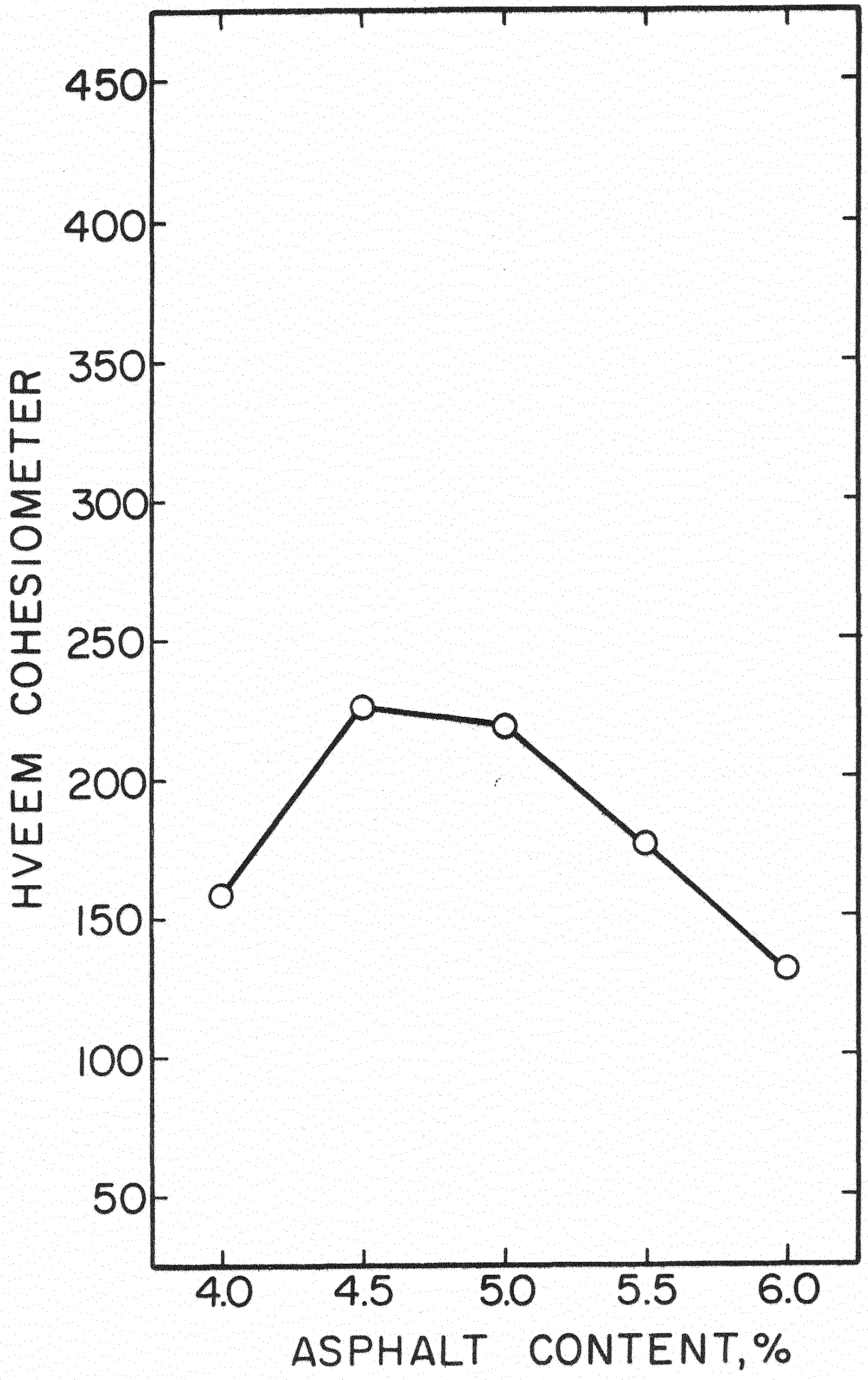


Figure 22. Hveem Cohesion Versus Asphalt Content
Cooperton Limestone

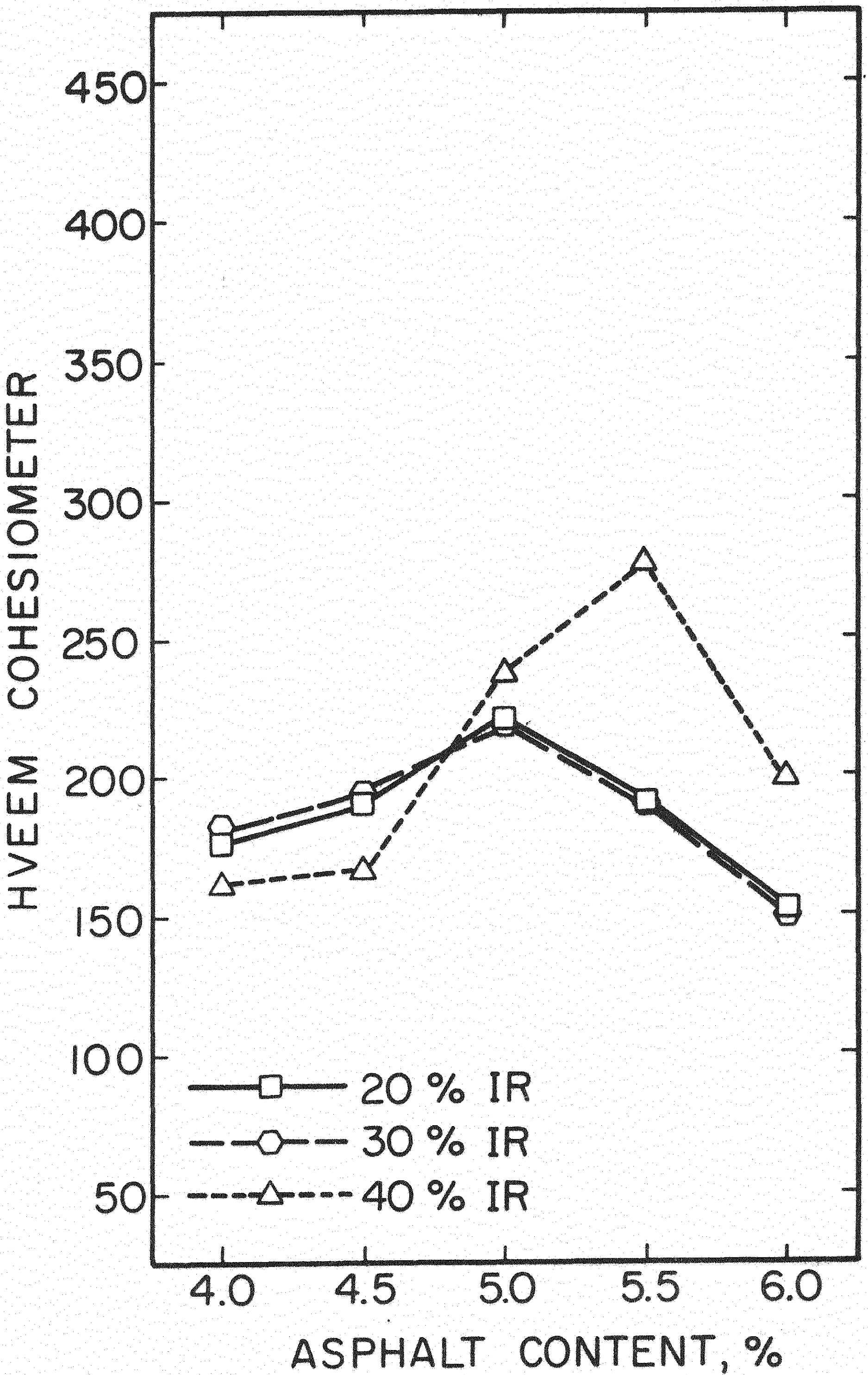


Figure 23. Hveem Cohesion Versus Asphalt Content
Asher Chert Gravel

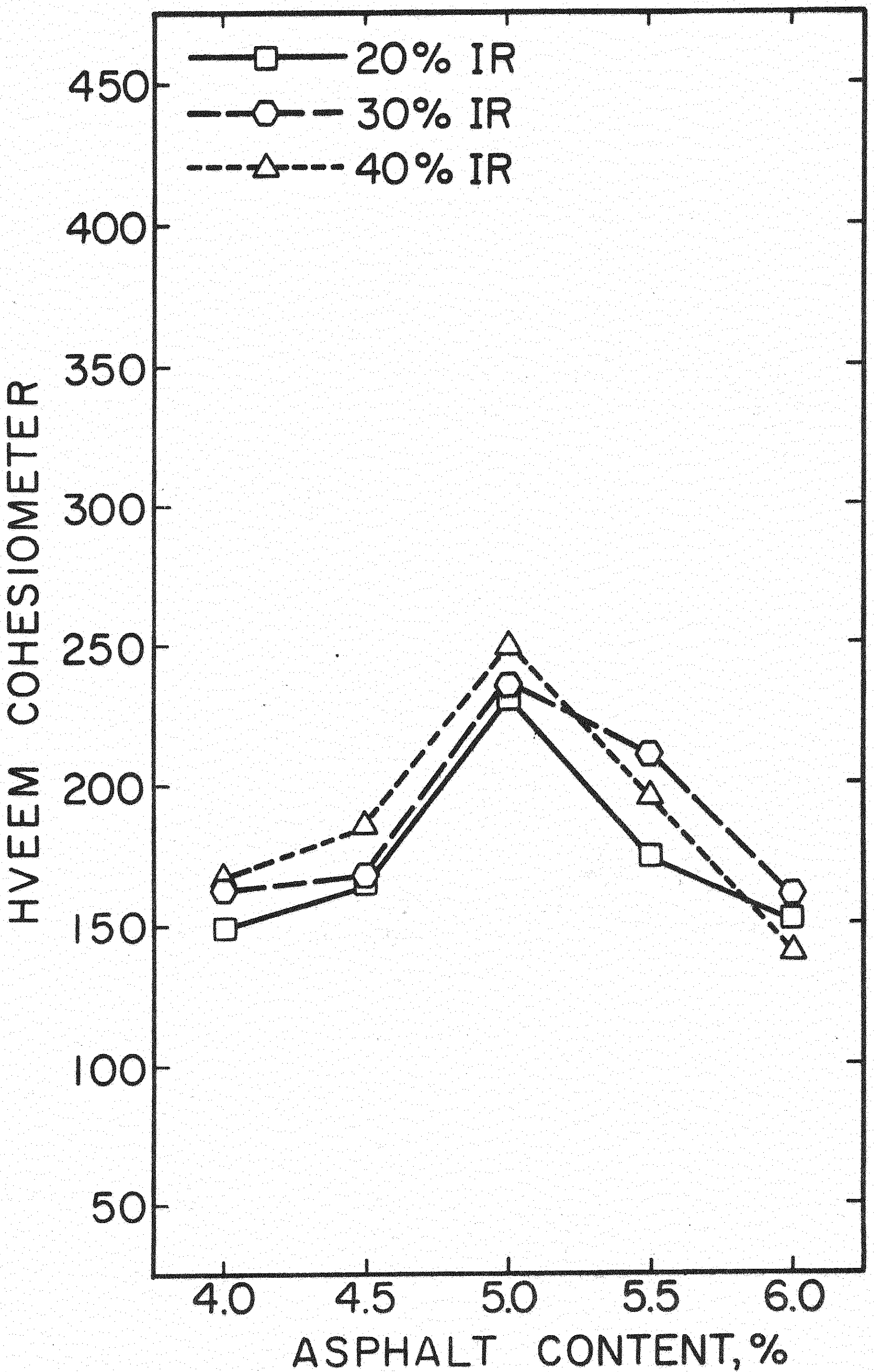


Figure 24. Hveem Cohesion Versus Asphalt Content
Miami Chert Gravel

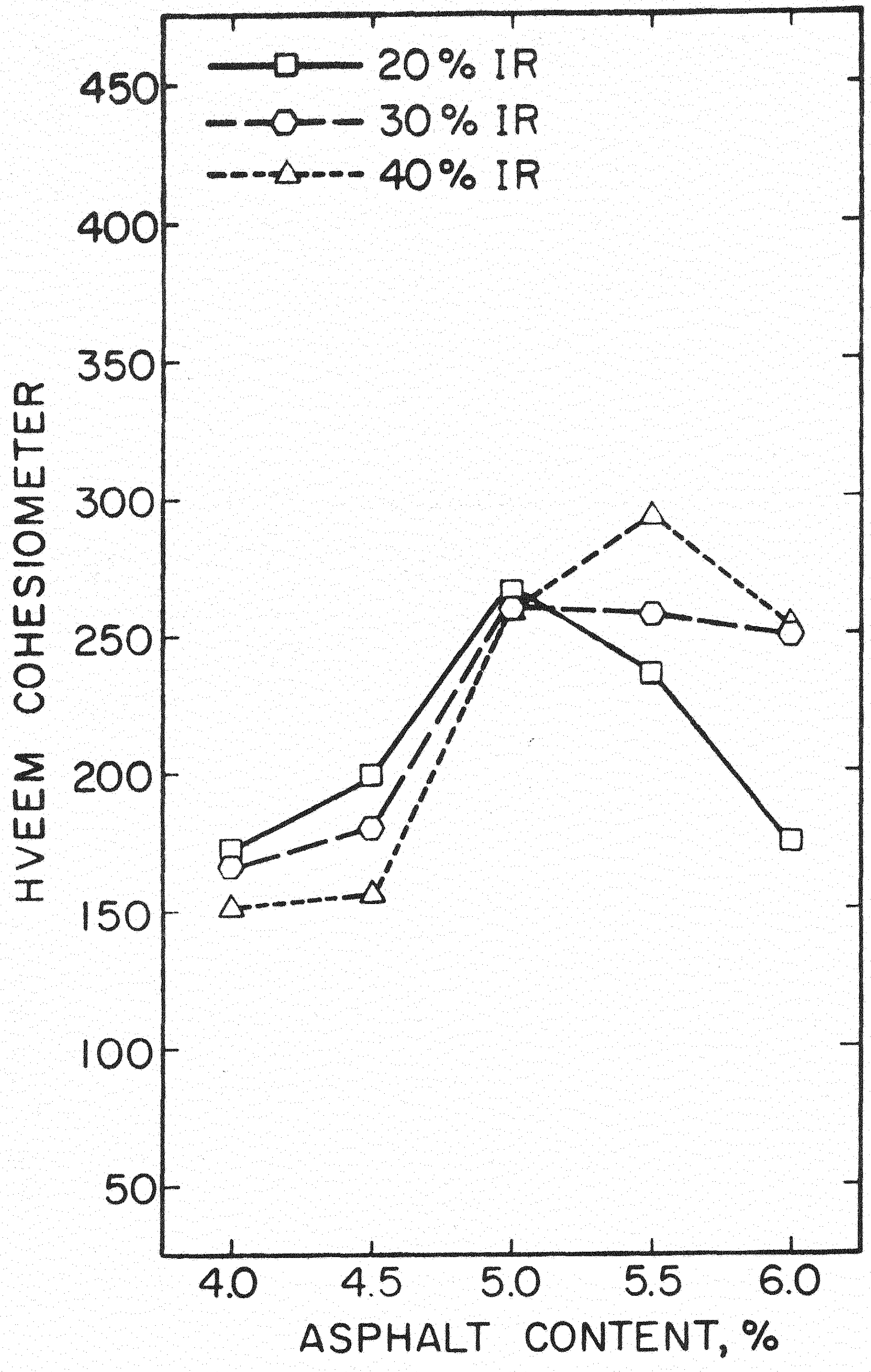


Figure 25. Hveem Cohesion Versus Asphalt Content
Onapa Sandstone

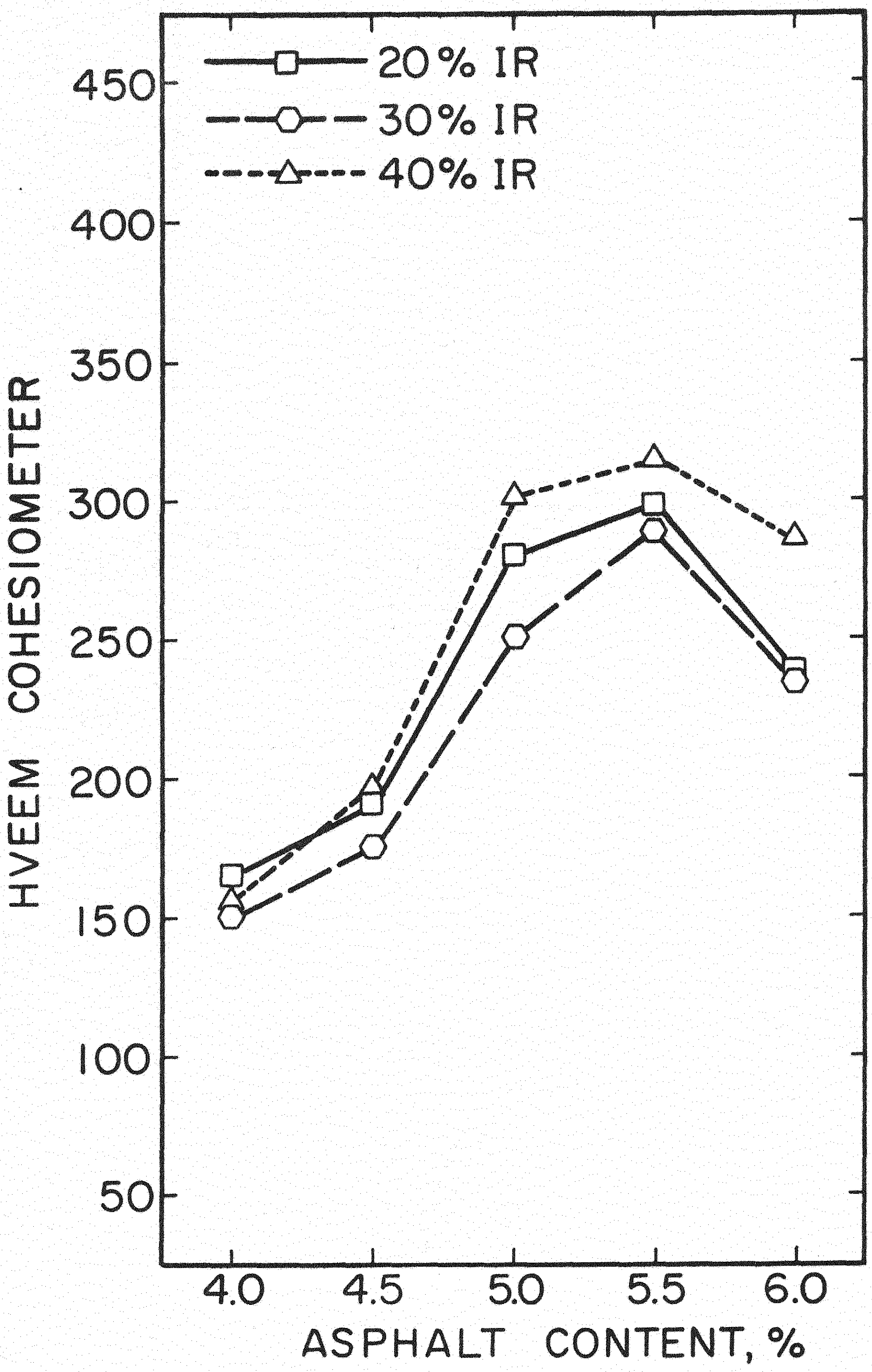


Figure 26. Hveem Cohesion Versus Asphalt Content
Stringtown Limestone

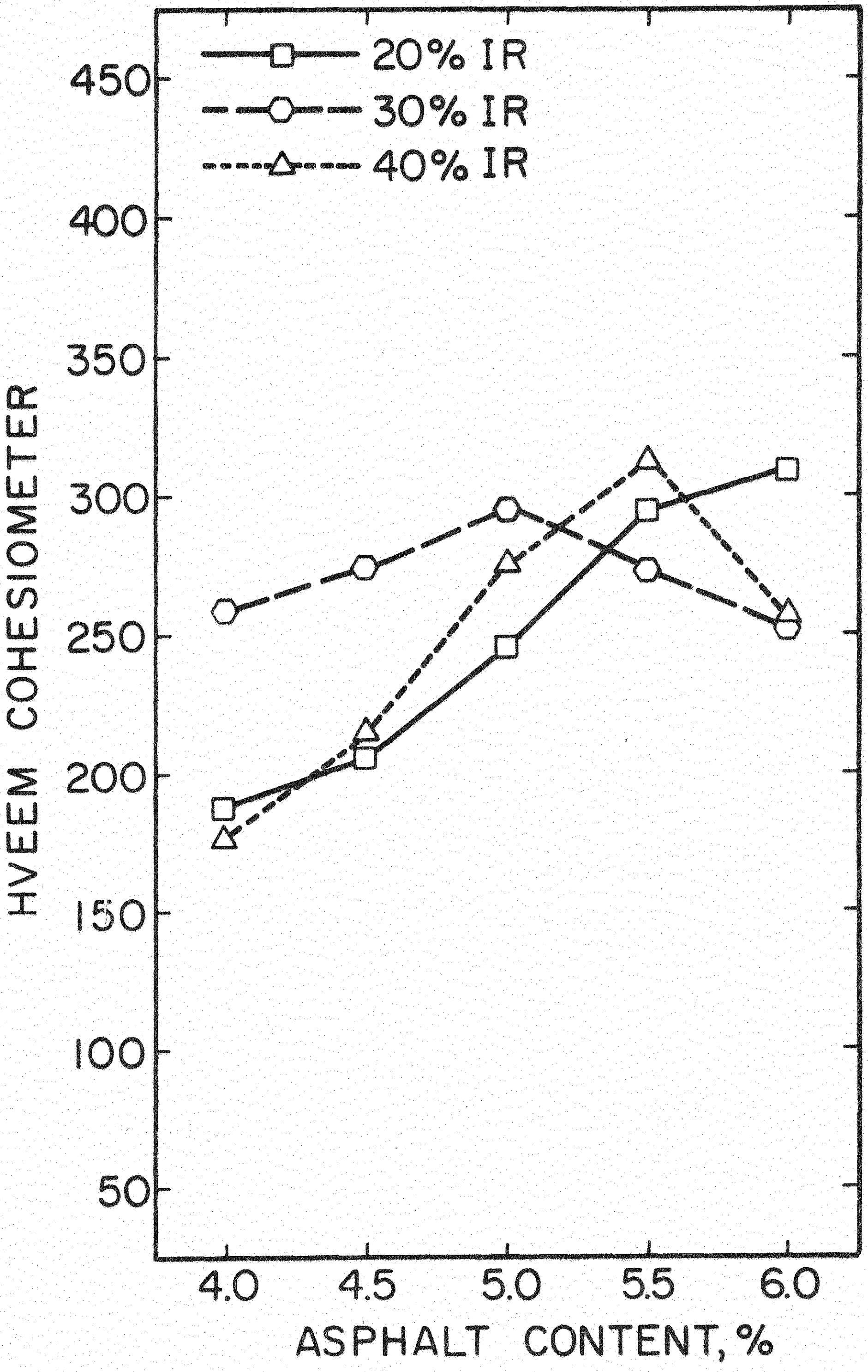


Figure 27. Hveem Cohesion Versus Asphalt Content
Cyril Sandstone

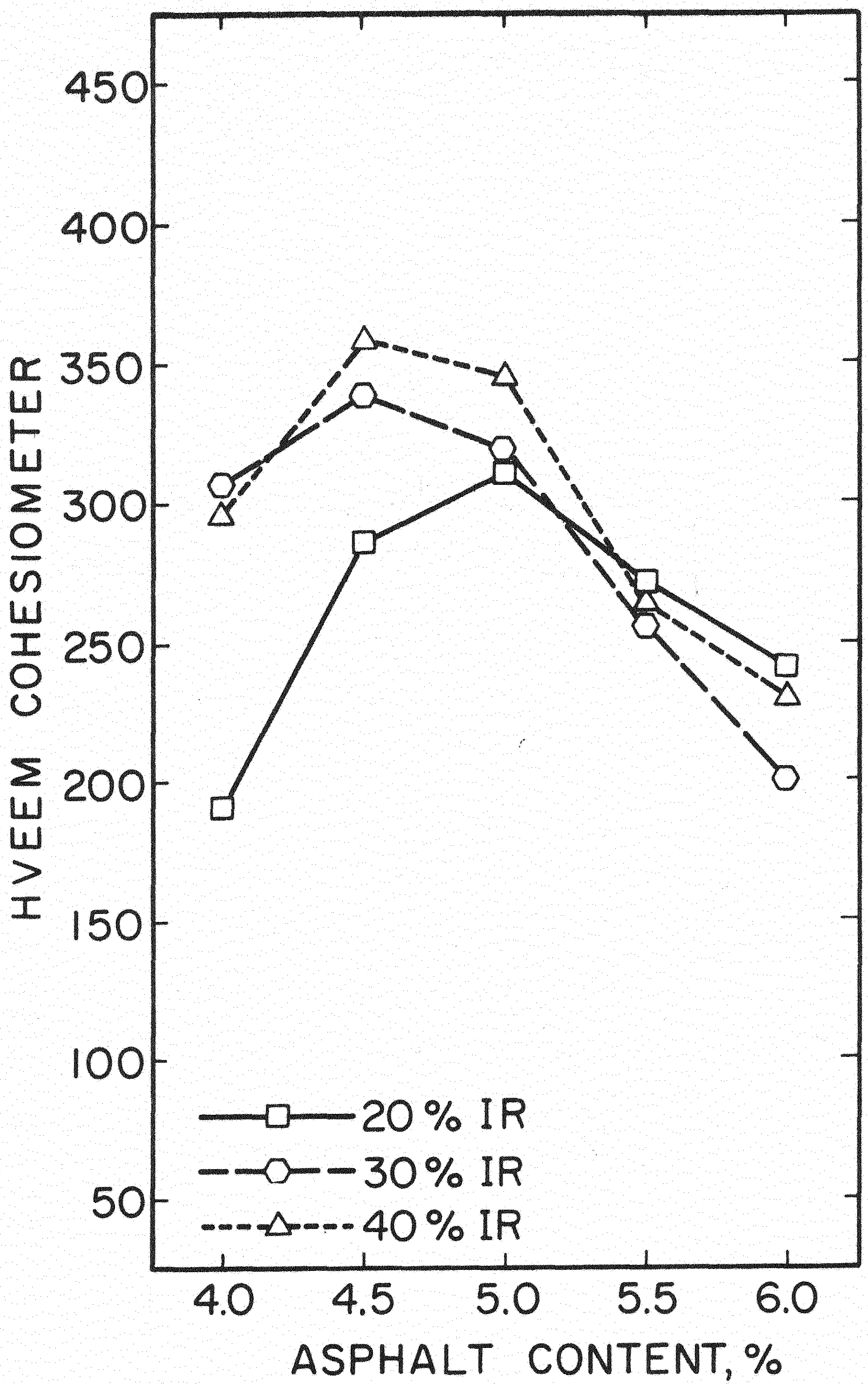


Figure 28. Hveem Cohesion Versus Asphalt Content
Broken Bow Chert Gravel

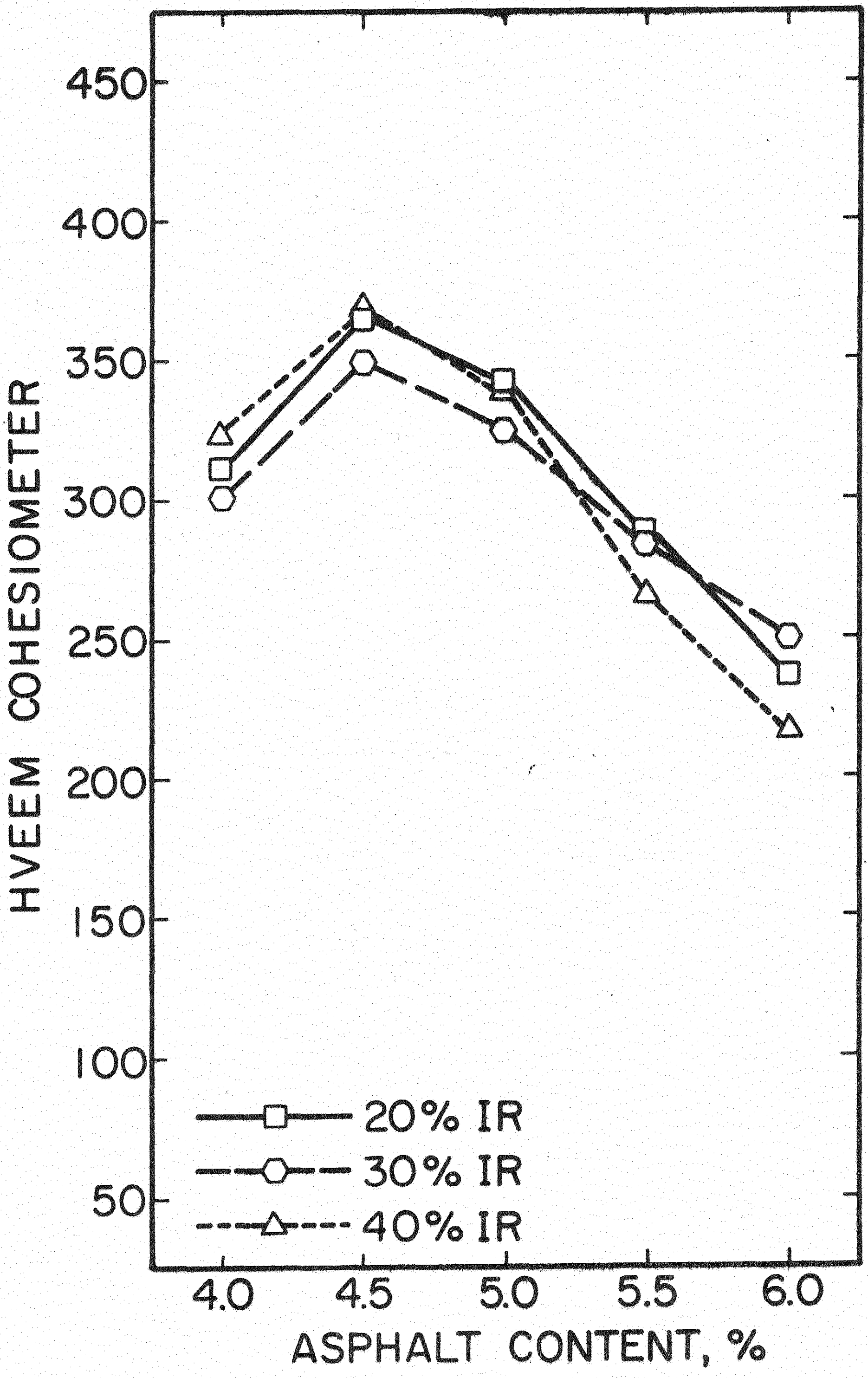


Figure 29. Hveem Cohesion Versus Asphalt Content
Gore Gravel

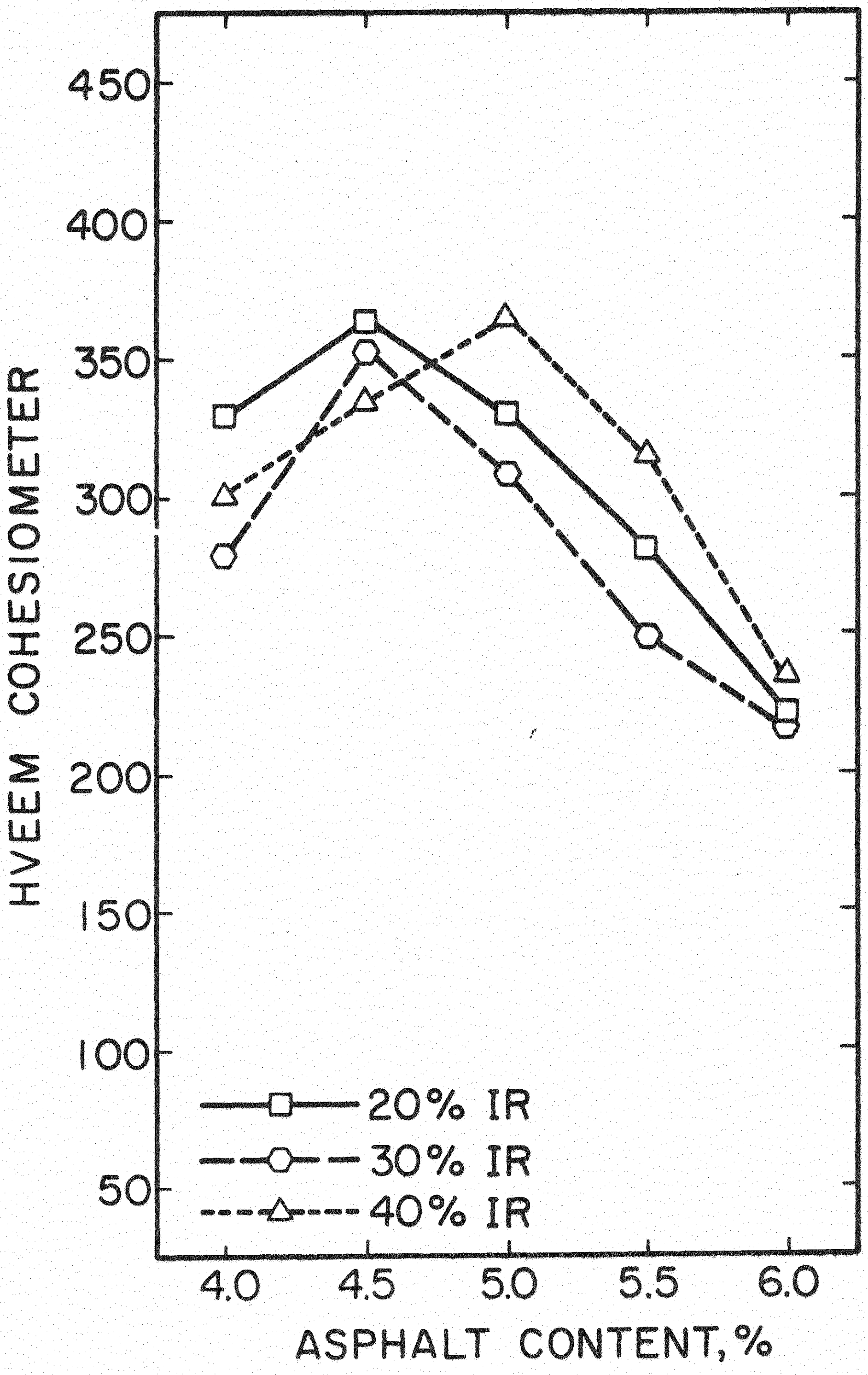


Figure 30. Hveem Cohesion Versus Asphalt Content
Hugo Chert Gravel

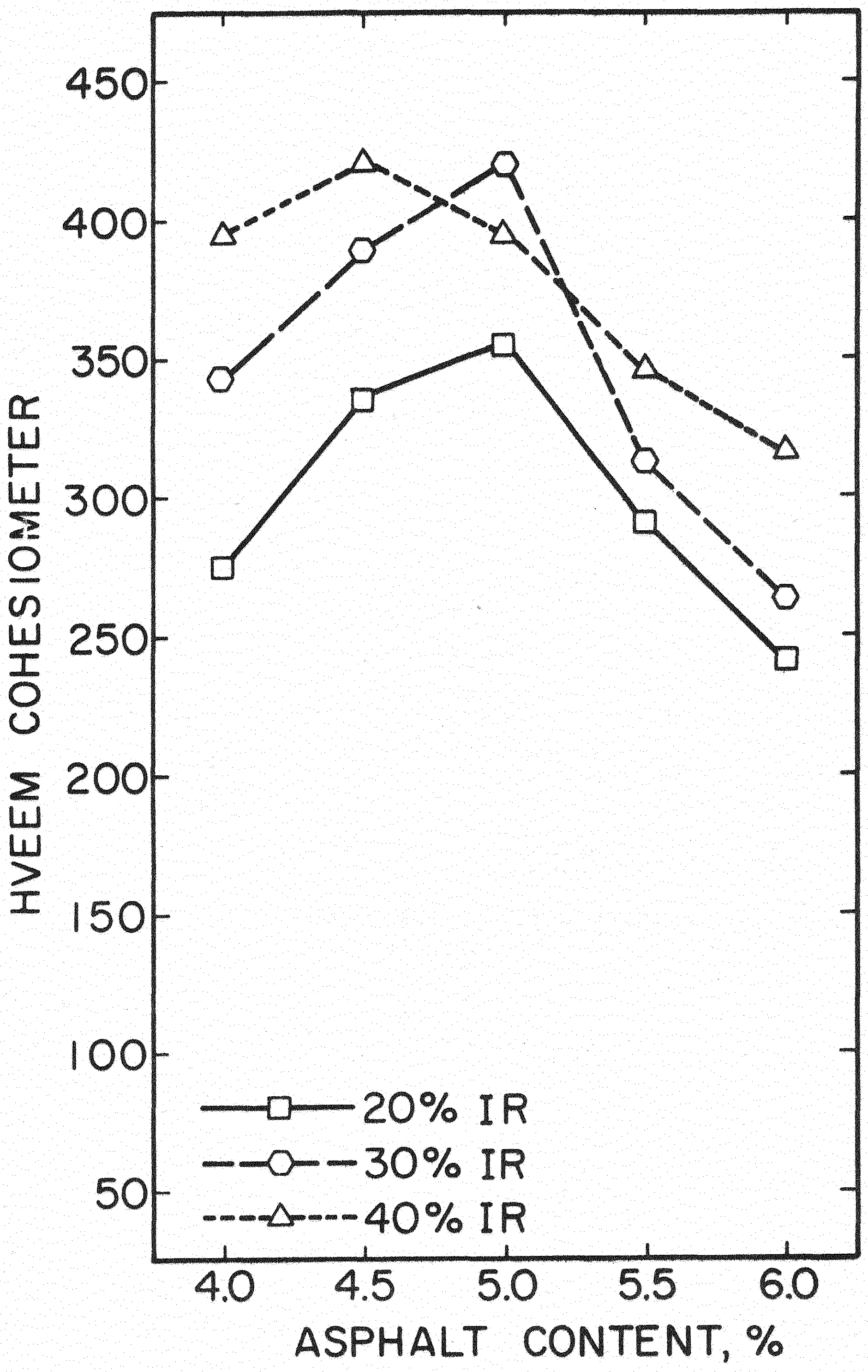


Figure 31. Hveem Cohesion Versus Asphalt Content
Keota Sandstone

APPENDIX D

PLOTS OF PERCENT DENSITY VERSUS ASPHALT CONTENT

Figures 32-41

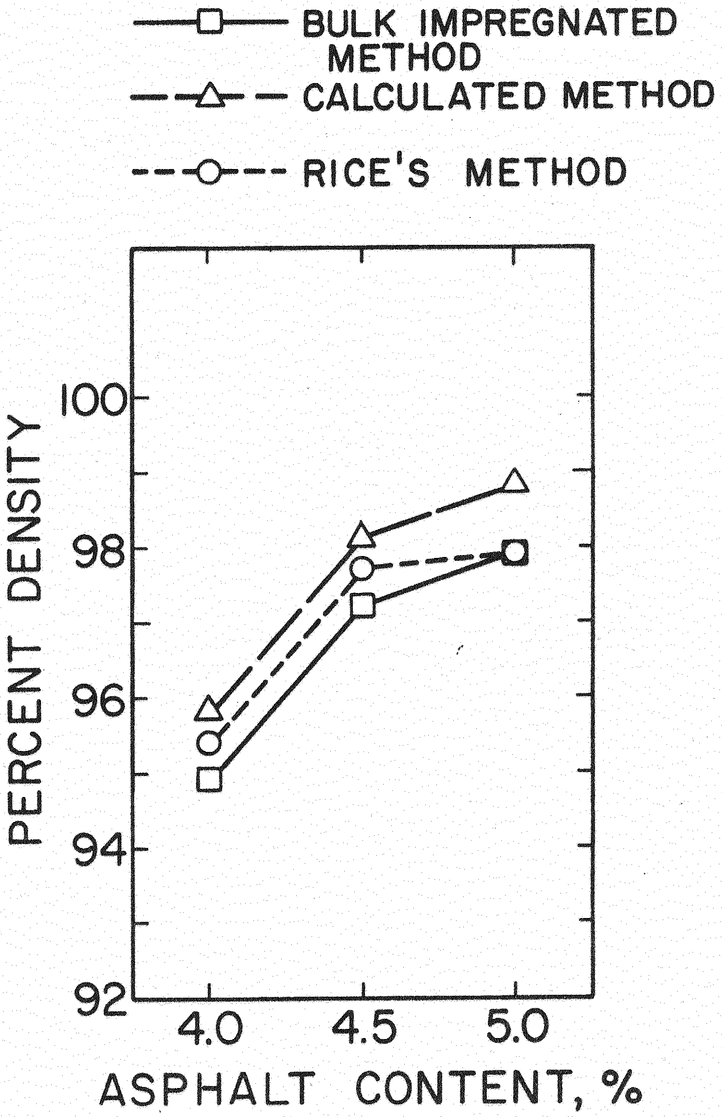


Figure 32. Percent Density Versus Asphalt Content Cooperton Limestone Mixes

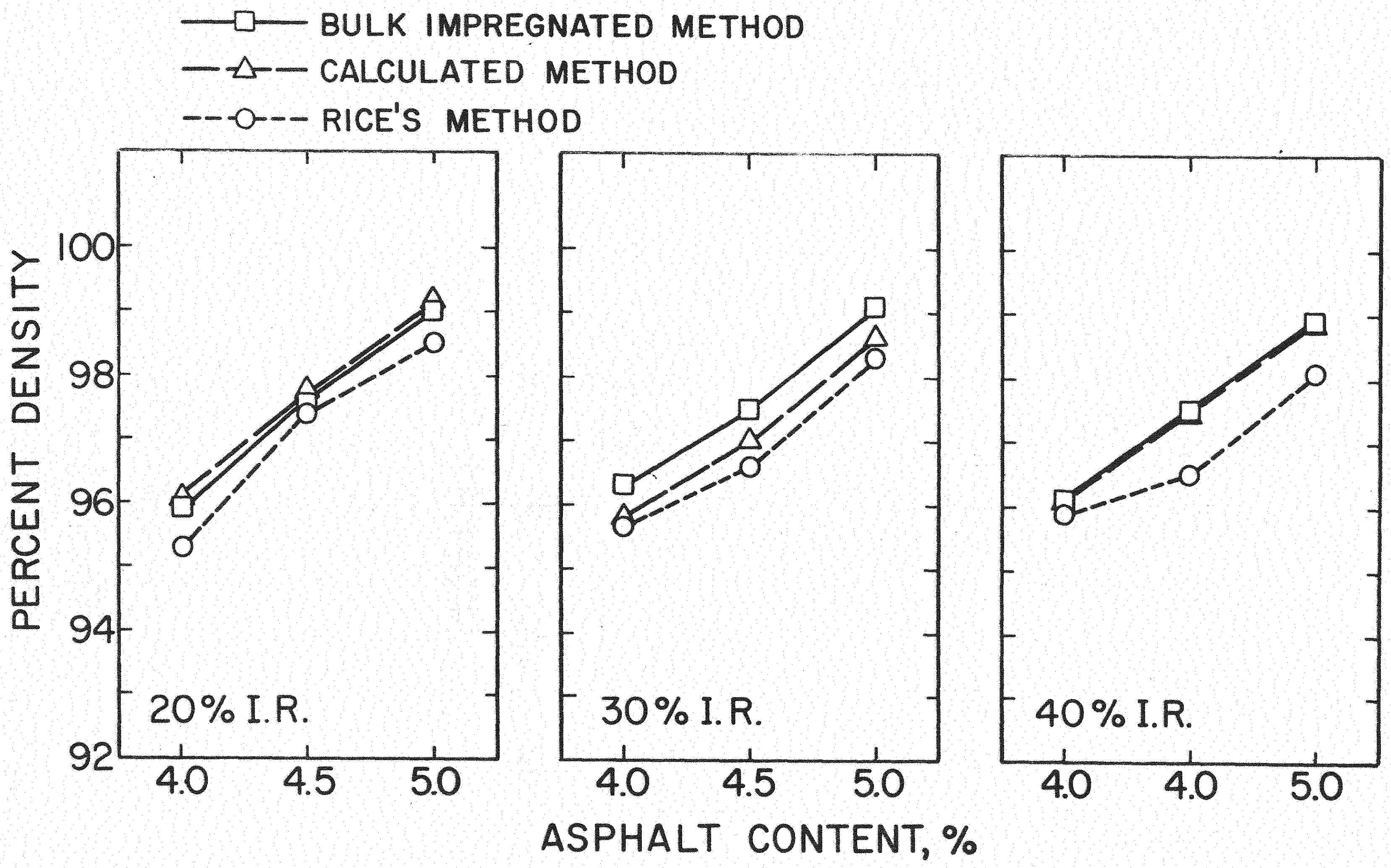


Figure 33. Percent Density Versus Asphalt Content Asher Chert Gravel Mixes

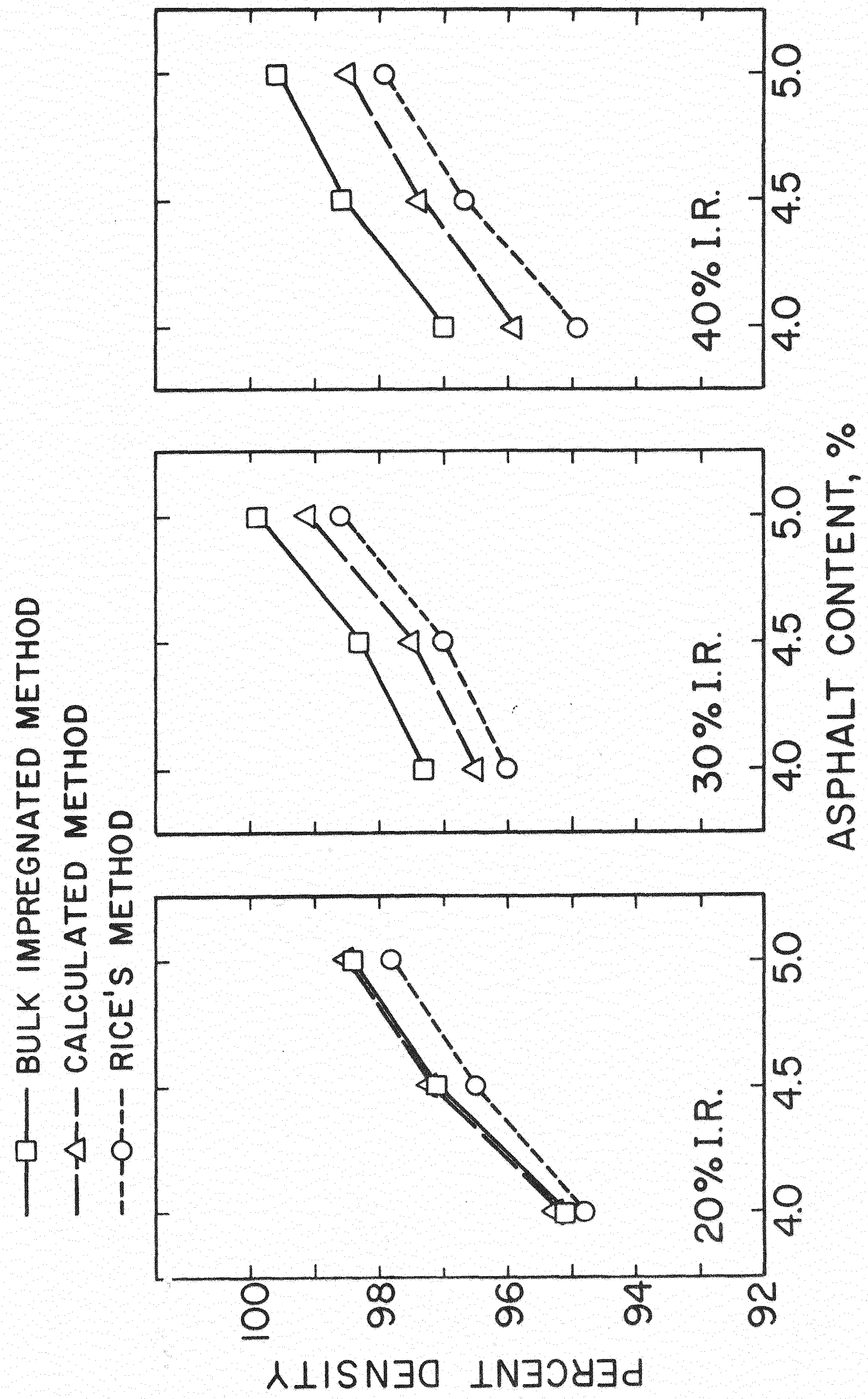


Figure 34. Percent Density Versus Asphalt Content Miami Chert Mixes

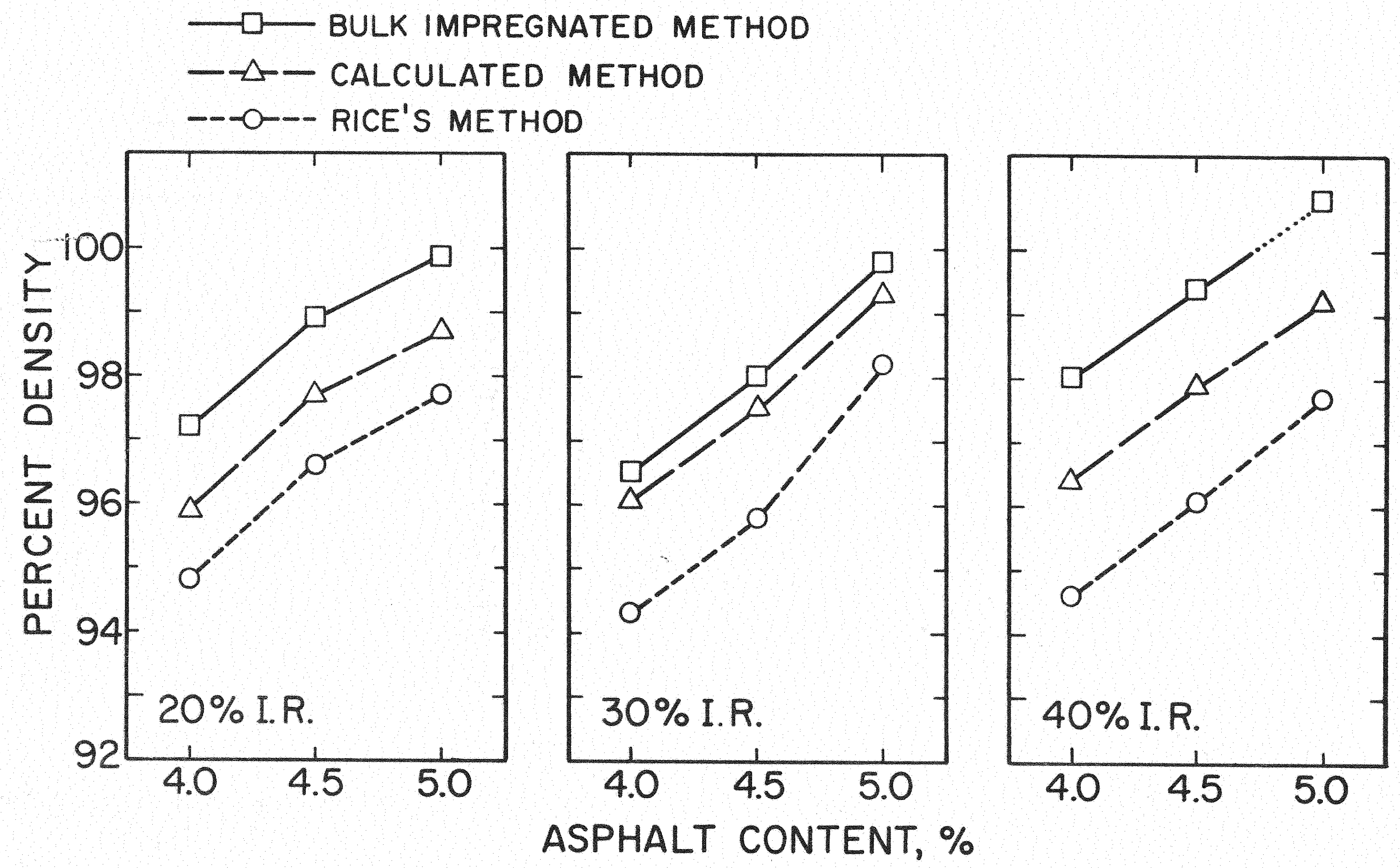


Figure 35. Percent Density Versus Asphalt Content Onapa Sandstone Mixes

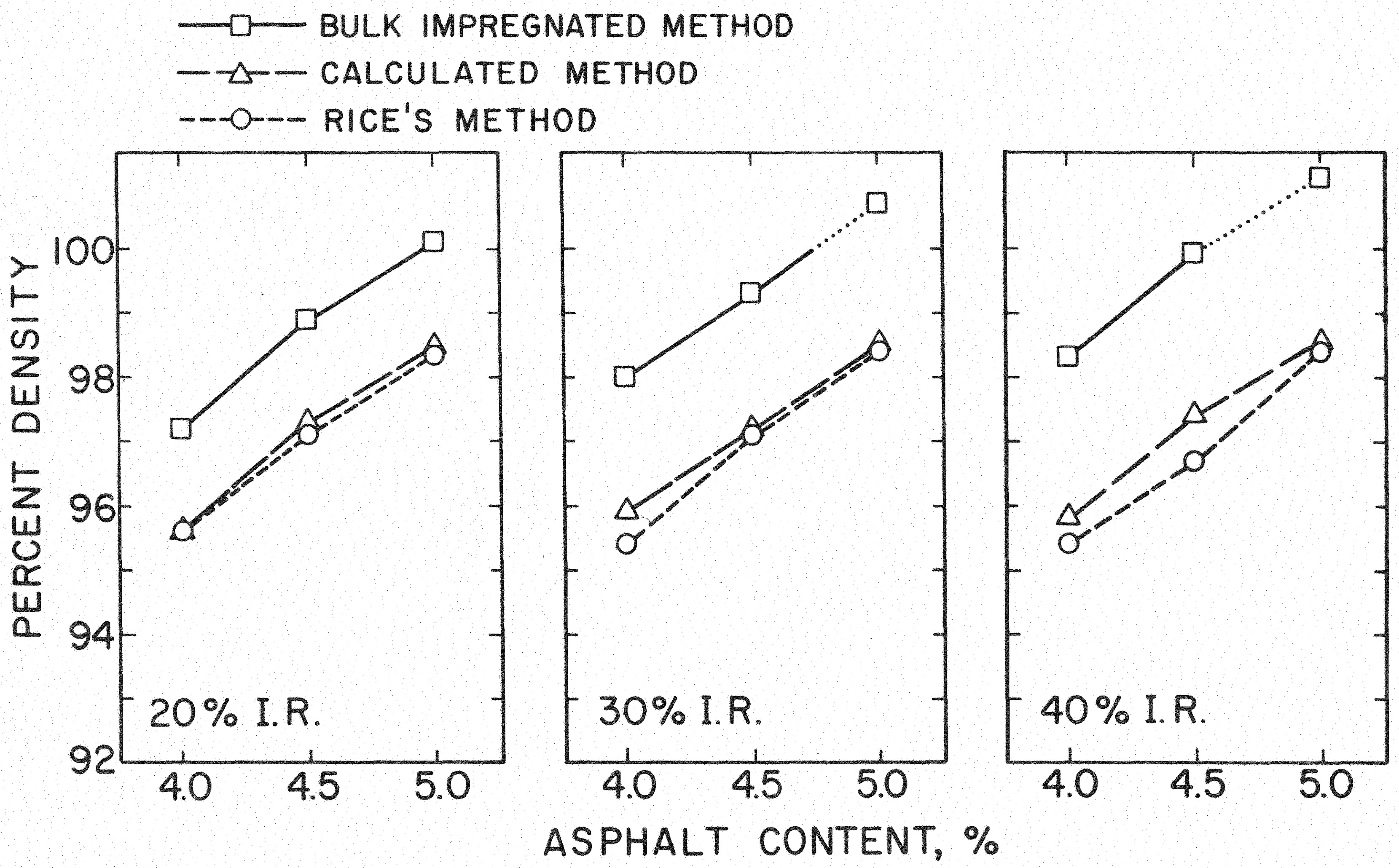


Figure 36. Percent Density Versus Asphalt Content Stringtown Limestone Mixes

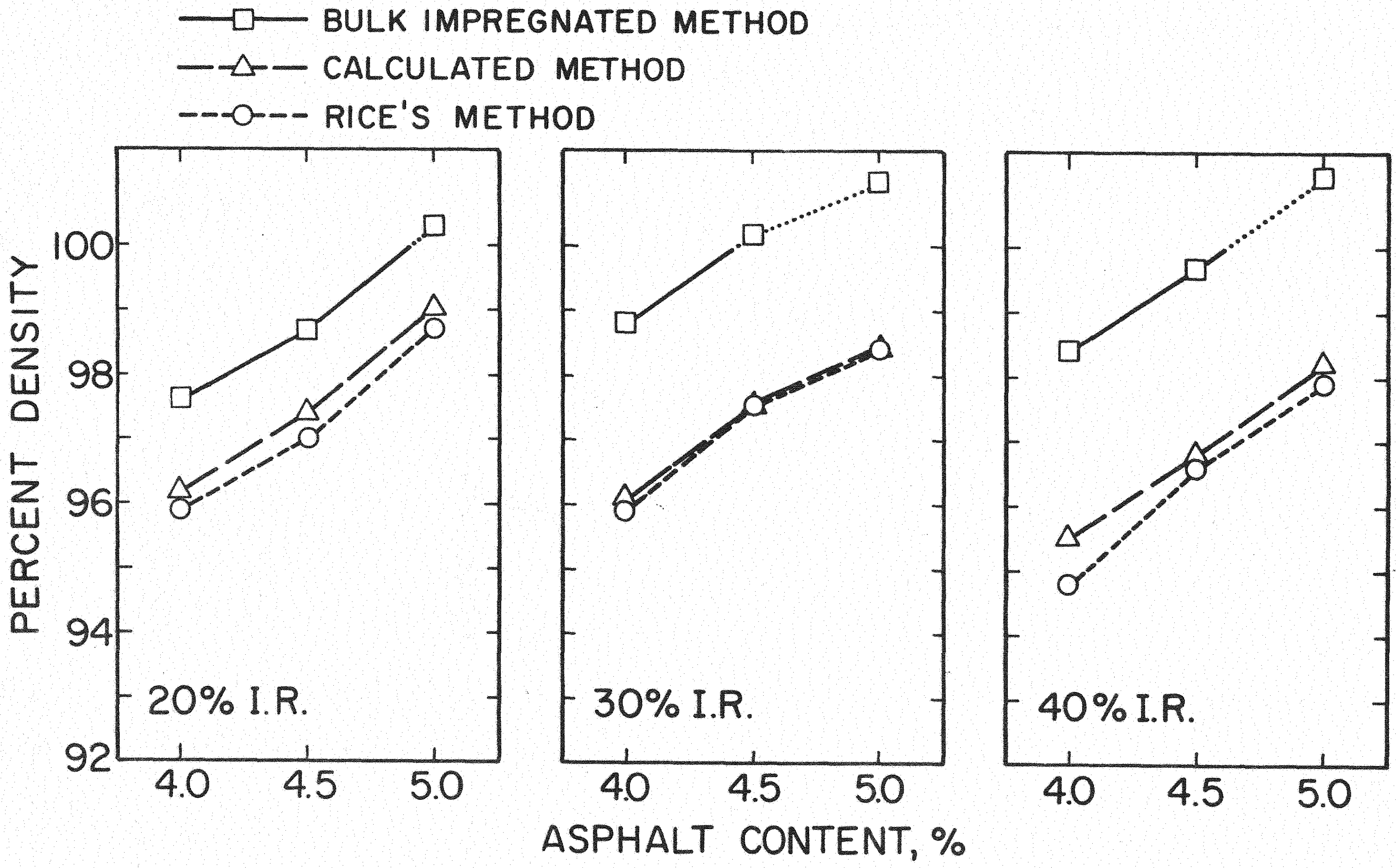


Figure 37. Percent Density Versus Asphalt Content Cyril Sandstone Mixes

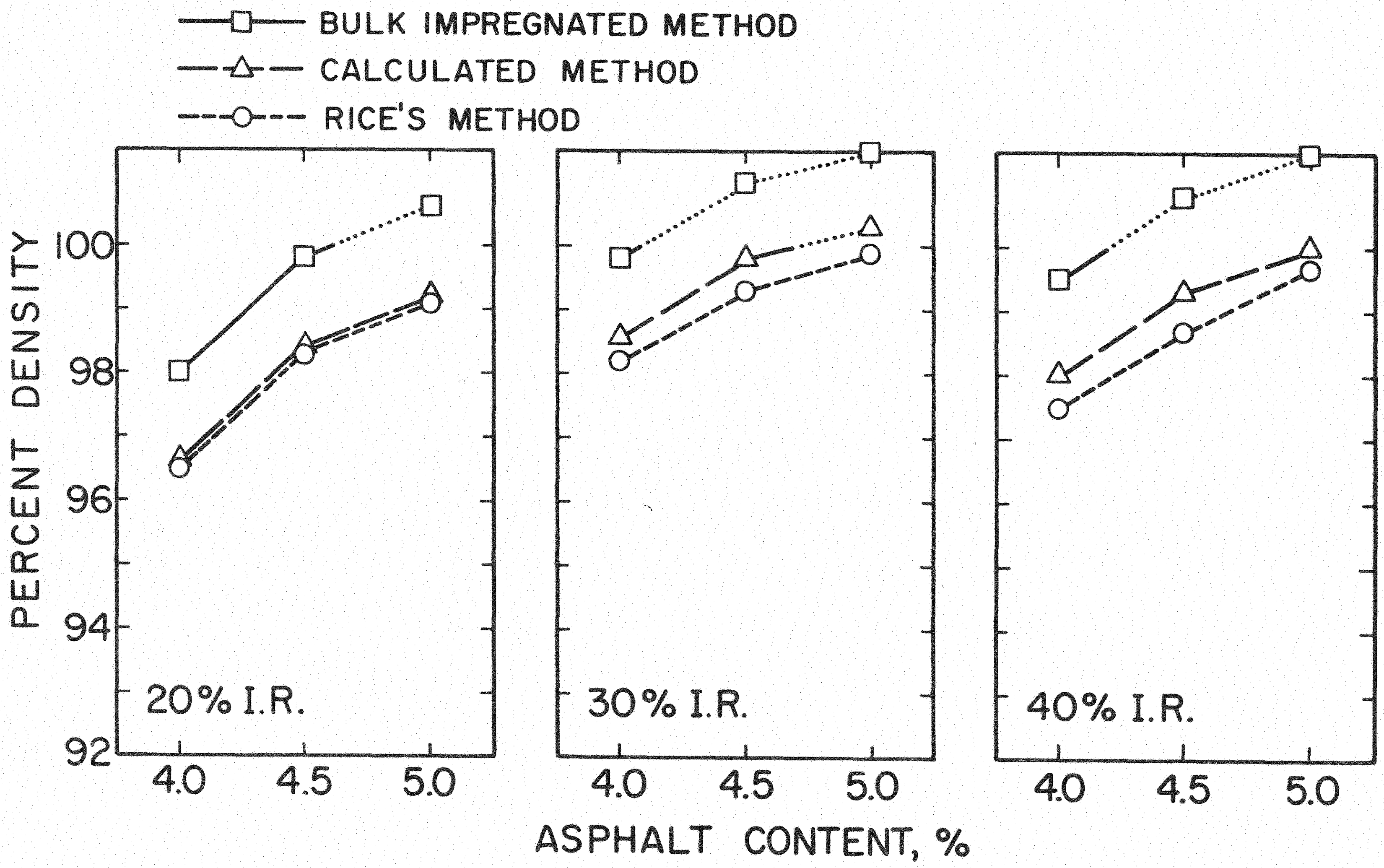


Figure 38. Percent Density Versus Asphalt Content Broken Bow Gravel

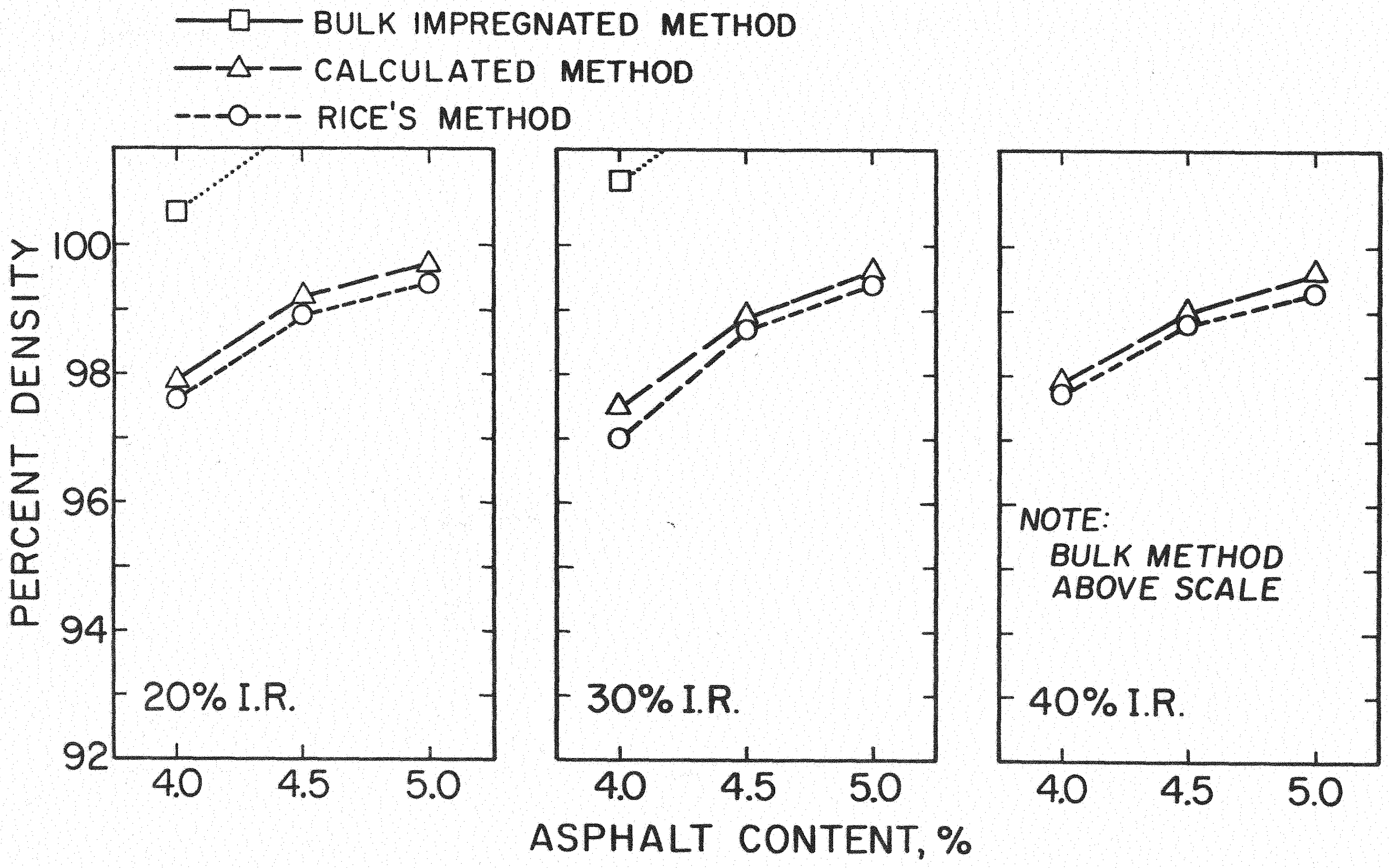


Figure 39. Percent Density Versus Asphalt Content Gore Gravel Mixes

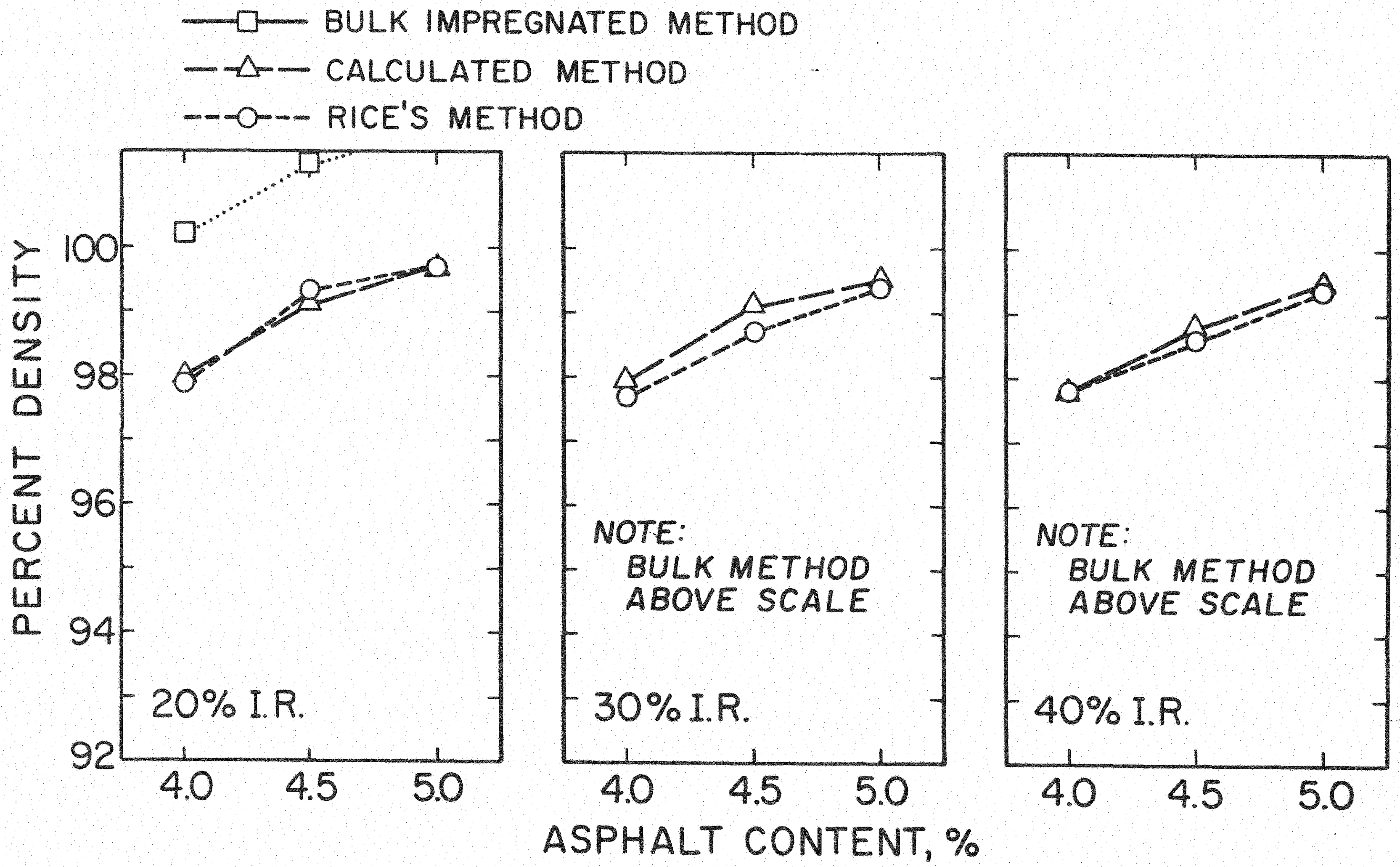


Figure 40. Percent Density Versus Asphalt Content Hugo Chert Gravel Mixes