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RECYCLED ASPHALTIC CONCRETE MIX DESIGN

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16. ABSTRACT

This report is concerned with the design and characterization of recycled asphaltic concrete pavements in the southern part of Arizona. Recycled mixtures designed by ADOT were obtained, reviewed, and compared with properties of the constructed pavement as represented by cores taken from the roadway. Measurements made from the recycled mixtures obtained from the cores consisted of density, Marshall stability and flow, aggregate gradation and specific gravity, and bituminous content and its viscosity. From the measured data, calculations were made for voids in the mineral aggregate and for air voids in the compacted mixture. These measurements and calculations showed that aggregate gradation control was excellent; however, values for air-void content and Marshall flow for the recycled portion of the cores indicated these to be cautiously low and high, respectively. These two measurements were evaluated for designing the new mixture. A theoretical procedure used without the benefit of laboratory testing for the design indicated a good potential for future usage since its results compared favorably with the final recycled mixture placed in the road. Recommendations were to continue the use of asphaltic cement as the recycling agent and to change to an open gradation for the recycled mixture.

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We sincerely value the importance given by ADOT and FHWA to the sponsorship of this investigation. We believe that the evaluation for and design of asphalt pavement recycling are still in a transitory phase.

METRIC CONVERSION FACTORS

1 in. = 0.0254 m

1 psi = 6.89 kPa

1 poise = 0.1 Pa-s

1 pcf = 16.03 kg/m^3

1 lbf = 4.44 N

$^oF = 32 + 1.8^oC$

RECYCLED ASPHALTIC CONCRETE MIX DESIGN

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RECYCLED ASPHALTIC CONCRETE MIX DESIGN

SYNOPSIS

This report is concerned with the design and characterization of recycled asphaltic concrete pavements in the southern part of Arizona. Recycled mixtures designed by ADOT were obtained, reviewed, and compared with properties of the constructed pavement as represented by cores taken from the roadway. Measurements made from the recycled mixtures obtained from the cores consisted of density, Marshall stability and flow, aggregate gradation and specific gravity, and bituminous content and its viscosity. From the measured data, calculations were made for voids in the mineral aggregate and for air voids in the compacted mixture. These measurements and calculations showed that aggregate gradation control was excellent; however, values for air-void content and Marshall flow for the recycled portion of the cores indicated these to be cautiously low and high, respectively. Air voids and voids in the mineral aggregate were used for designing a new recycled mixture. A theoretical procedure developed for design indicated a good potential for future usage since its results compared favorably with the final recycled mixture placed in the road. Recommendations were to continue the use of asphaltic cement as the recycling agent and to change to an open gradation for the recycled mixture.

INTRODUCTION

Following the petroleum embargo in 1973, the construction industry became extremely energy conscious, especially with the use of oil and gas as a fuel. Our concern in this report is principally in highway construction and specifically with the hot-mix recycling of asphaltic concrete pavement surfaces.

The interest in hot-mix recycling seems to have started with the early work done by the Las Vegas Paving Corporation of Las Vegas, Nevada. In September of 1974, the Las Vegas Paving Corporation entered into a contract with the Nevada Highway Department to recycle one mile of asphaltic concrete for replacement on Interstate 15 near Las Vegas (1, 2, 3). In this demonstration project, four inches of surfacing plus one inch of base course were picked up, hot-mixed with asphalt plus an asphalt softening agent and placed on the old roadway to depths of five to seven inches.

Although several problems were encountered in the Nevada Demonstration Project, authors Proudy, Gregory, and Hodges (3) stated the following as benefits of the recycling process:

> "The ability to recycle old or discarded asphaltic pavements can be of significant value in conserving energy, natural resources and in the preservation of our natural landscape.
>
> Some of the more obvious benefits are enumerated below.
>
> 1. Reduces the need for exploring and developing new aggregate sources and conserves existing aggregate sources.
>
> 2. Eliminates the necessity of locating disposal sites for discarded pavements.

3. Conserves expensive and scarce asphaltic products. Recycled asphalt pavement requires about 75% less asphalt cement than does virgin material.

4. Distressed pavements can be recycled in lieu of placing thin overlays that are especially prone to reflective cracking.

5. The structural value of a distressed pavement can be increased by recycling a portion of the underlying base aggregates along with the bituminous pavement.

6. The distressed section of a pavement can be recycled without disturbing the pavement that is in good condition. For example on this project the travel lane was in poor condition while the passing was in good condition with many years of service remaining."

The benefits listed above for recycling asphaltic concrete pavements can be summarized to three aspects of (1) cost reduction, (2) energy saving, and (3) conservation of natural resources in the reconstruction or rehabilitation of an existing road.

The total recycled asphaltic concrete pavement involves the operations of mixture design and construction. Each of these two processes is important for the success of the reconstructed highway; however, in this report we are primarily concerned with the mixture design phase.

Figure 1 is a flow chart showing the steps required for the design of a recycled mixture. It is apparent that the old asphaltic concrete must be sampled to identify its composition. Since many of these surfaces have been maintained, it is important that sufficient samples be taken for proper characterization; unfortunately, except for Reference 12, the literature has not given enough guidelines as to the frequency or number of samples to be taken.

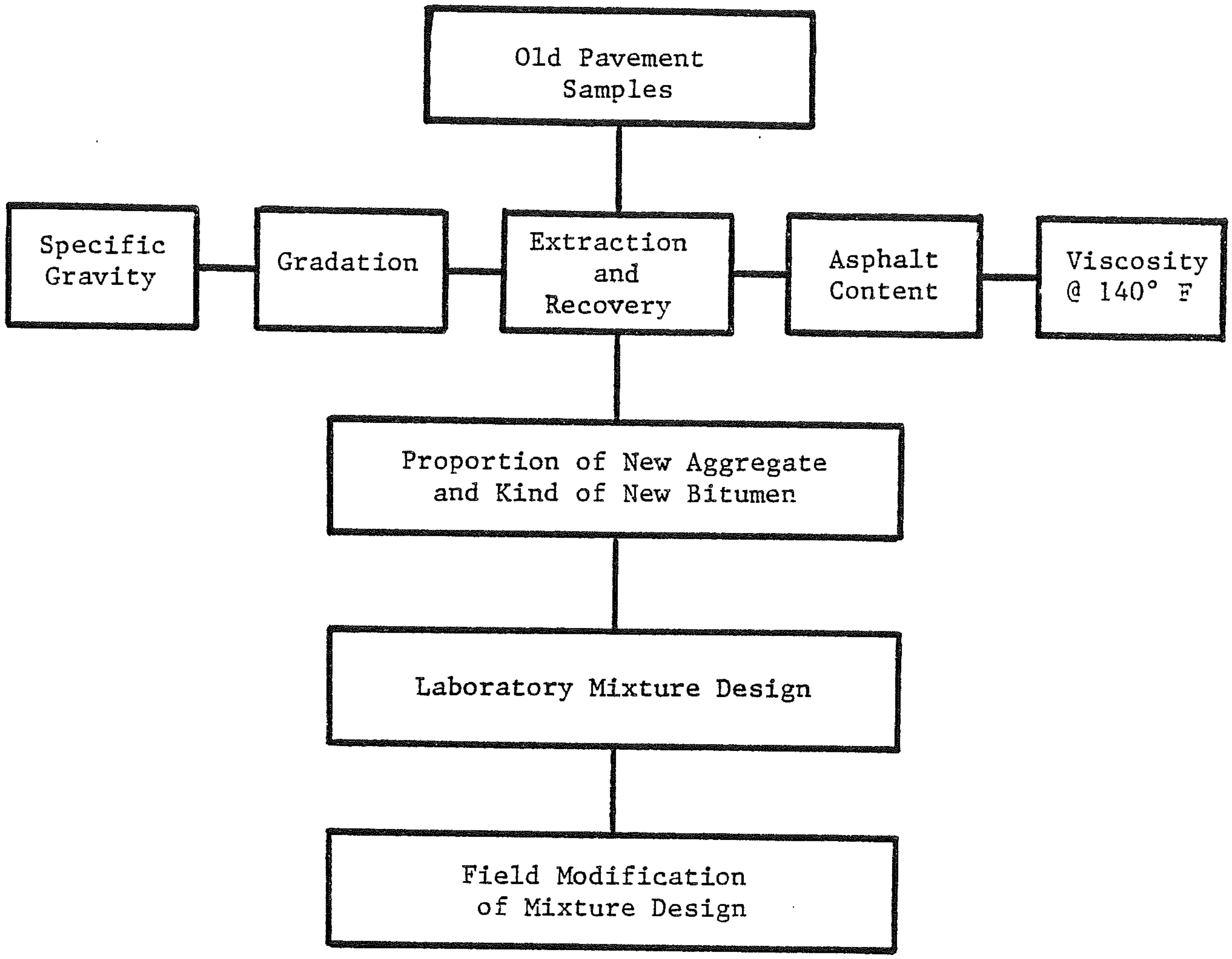

Figure 1. General Flow Chart for the Design of Recycled Hot-Mix Asphaltic Concrete.

The pavement samples have to be broken down in consideration of the construction process for obtaining the reclaimed asphaltic material; that is, how much aggregate degradation will result. The core or chunk samples will be processed through extraction and recovery of asphalt to determine amount and properties of the aggregate and asphalt.

From the extraction-recovery information, one must establish the amount and gradation of virgin aggregate to be added. Also, it will be necessary to select the type and amount of bituminous material to serve as the recycling agent. The types and amounts of virgin aggregate and recycling agent to be used must be established in regards to obtaining desirable mixture characteristics for voids in the mineral aggregate, air voids in the laboratory compacted mixture, viscosity of the binder, and stability of the mixture. The viscosity of the binder is an important function of the recycling agent because a portion will mix with the old asphalt and another portion will serve as the only binder for the virgin aggregate.

Laboratory testing of the mixture to be recycled will zero-in for setting design amounts of virgin aggregate and additional bituminous material.

Finally, as expected, modifications will be made to the laboratory design to meet field conditions but yet holding to specification requirements.

Since 1974 there has been a great amount of effort expended in the development of the total process involved in the hot-mix recycling of pavement and much of the work has been reported in References 4 to

13, inclusive. Many State Departments of Transportation regularly use hot-mix recycling as a standard method for pavement reconstruction; however, according to a recent article in Civil Engineering (11) many states (e.g., Texas) do not use recycling as a standard alternative. Hesitation in accepting recycling appears to come from the lack of long service-time performance and the many changes occurring in construction and design of the recycled pavements.

The objectives of this investigation were to review the data and methods that the Arizona Department of Transportation had used in the evaluation and design of constructed recycled pavements. The work involved obtaining mixture design information for selected recycled pavements, sampling the road sections, observing performance, and comparing physical properties of road cores with their design characterization.

From the data accumulated and using a new asphaltic concrete mixture design method developed at The University, we were to present a modified design procedure for future recycled pavement mixtures.

The reader must realize that the design and construction of the recycled pavements to be reported occurred over a period of years. As a consequence, knowledge was gained for improving the sampling, testing, design, and construction procedures.

PAVEMENT TEST SITE CONDITION AND LOCATION

The Materials Division of ADOT selected seven recycled projects for review for which certain design data were available. Additionally, another pavement that had been elected for future recycling was chosen to be sampled and the recycling mixture to be designed by the Asphalt Laboratory of The University of Arizona.

Table 1 lists the location and name for each of the test sites to be reviewed and the one to be designed. Figure 2 shows the locations of these sites to be predominantly in the southern portion of the State. Initially, there had been two pavement sections in the Flagstaff area; due to problems with facilities for sampling and weather conditions, these two were substituted for two sites in the southern region.

TABLE 1 - Location and Name of Test Sites for Sampling Recycled Asphaltic Pavements

Site No.	Name	Project Mile Post	Lane	Construction Date	Condition
1.	Sentinel	I8-2(76) MP 94	EB Travel	Summer 1978	1/4-inch rutting
2.	Rillito	I10-4(68) MP 247	EB Travel WB Travel	Spring 1980	1/4-7/8 inch rutting
3.	Dateland	I8-1(80) MP 60	WB Travel	Summer 1982	good
4.	Firebird Lake	IR10-3(142) MP 163	EB Travel	Summer 1982	1/2-3/4 inch rutting
5.	Gila Bend	I8-2(80) MP 124	EB Travel WB Travel	Summer 1982	good
6.	Williams Field	IR10-3(148) MP 163	WB Passing WB Travel	Summer 1983	good
7.	Willcox	I10-6 (493) MP 340	- To be sampled and designed by The University of Arizona during Fall of 1984.		
8.	Red Rock	IR10-4(86) MP 229	WB Travel	Spring 1985	good

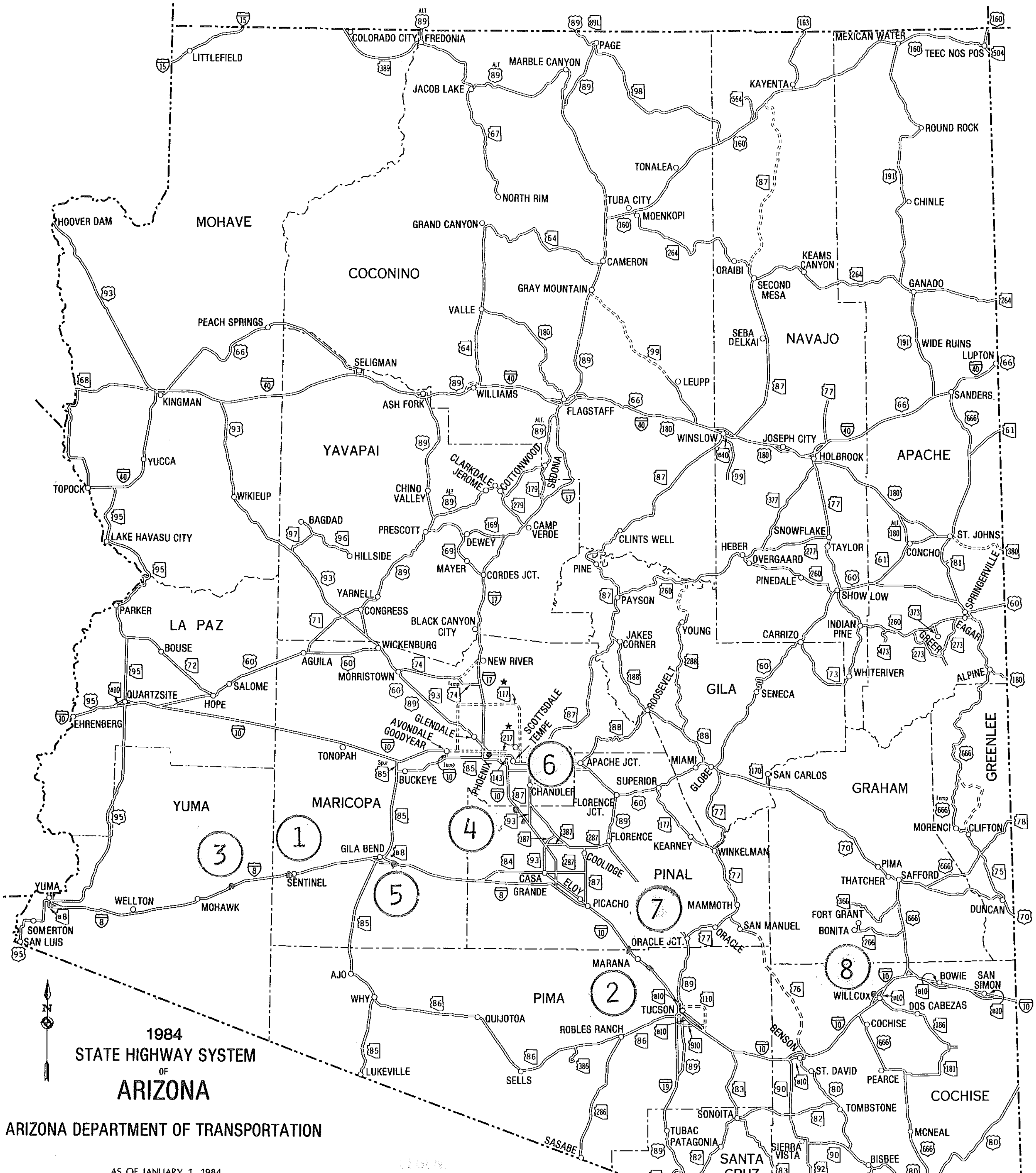

Figure 2. Test Site Locations.

PAVEMENT SAMPLING AND TESTING OF CORES

ADOT personnel helped in the selection of the specific location for coring the pavements and also furnished the traffic control. The cores were cut through the total pavement surface thickness with a four-inch diameter bit. The hole was kept clean of grindings and also dry through the use of forced nitrogen into the center of the bit. At least six cores were taken across the lane -- two each at the outer-wheel path (OWP), between-wheel path (BWP), and the inside-wheel path (IWP).

At the Laboratory, the layer containing the recycled mixtures was identified and the core was sawed to yield a test specimen 2-1/2 inches high or the total thickness of the layer. The evaluation and characterization of the recycled mixture consisted of the following sequence:

1. Core Density - AASHTO: T166-78(1982) (14)
2. Core Marshall Stability and Flow - AASHTO: T245-82
3. Extraction of Asphalt - AASHTO: T164-80, Method B
4. Recovery of Asphaltic Binder, Modification of ARIZ 511 (15)
5. Viscosity of Recovered Asphaltic Binder, AASHTO: T202-80
6. Gradation of Aggregate
7. Effective Specific Gravity of Aggregate
8. Calculation of "Measured" Voids-in-the-Mineral Aggregate (VMA) and Total Air Voids
9. Calculation of "Theoretical" VMA and Total Air Voids

The procedures for determining the above-listed items are fairly standard ones; however, a description of the asphalt recovery method is given in Appendix C. For most of the sampled mixtures, the total process of extraction, recovery, and viscosity determination was completed within an eight-hour period. The exceptions to this time period occurred for the two recycled mixtures that contained cyclogen. The extraction process for these mixtures took a minimum of 24 hours, which exceeded our requirement that the recovered asphalt not be in solution with the solvent (methylene chloride) for more than eight hours. The viscosity values for these recovered asphaltic binders are considered to be suspect. The delay in the extraction process is attributed to the plugging of the filter paper by the cyclogen.

For Item 7 aforementioned, the extracted aggregate was mixed with a known amount of asphalt of given specific gravity; the combined specific gravity of the loose mixture was determined (Rice); and from the combination, the effective specific gravity of the aggregate was calculated.

The next section of the report will present the basis and procedure for estimating the optimum asphalt content for either a virgin asphaltic concrete or a recycled mixture.

A NEW MIXTURE DESIGN PROCEDURE

Basis for a Calculated Asphalt Content

The quality of the aggregate and asphalt in a paving mixture are certainly important factors in the service performance of the asphaltic concrete. The contributions of the individual components will not be discussed here; however, we will present a method for establishing a starting or optimum asphalt-content for the laboratory testing of mixtures to determine the design amount of asphalt. The design asphalt content to be used in construction will be established through laboratory tests for stability and durability. A basic thought in this new approach is that the laboratory compacted specimen will be viewed with the potential of having certain properties approaching those of the pavement surface after it has been in service for a period of time (4-5 years) so that the rate of change in properties is not as great as immediately after construction. The procedure is based on controlling the voids in the mineral aggregate (VMA), the amount of air voids (AV) in a compacted mixture, and having an adequate asphalt film thickness on the aggregate. The values of VMA and AV are those that are thought to be necessary for stable paving mixtures that have been in service long enough to have reached a constant amount.

Initial Material Testing

Estimates for the optimum asphalt content would be obtained without physical testing of compacted mixtures. A limited amount of testing for information on material properties will be performed to obtain the following component characteristics:

1. Aggregate - gradation, effective specific gravity, and water absorption must be less than 2.5 percent,

2. Asphalt - specific gravity.

If desired, an estimate (since no direct measurements is available) of the absorption of asphalt by the aggregate can be used in the procedure as will be shown later.

Target Values for Estimating the Asphalt Content

The new approach for calculating the optimum asphalt content has certain criteria or target values that have been selected for controlling (a) the mixture's resistance to rutting and (b) durability. The target values are to provide a balance among VMA, AV, and asphalt film thickness after four to five years of service. These values are as listed in the following:

1. A minimum AV of 2 percent calculated with the effective specific gravity (ESG) of the aggregate. This amount is to preclude bleeding and rutting originating within the asphaltic course.

2. A minimum VMA calculated with the ESG of the aggregate blend. The minimum value of VMA is to provide space in the compacted aggregate to accommodate the 2 percent AV and sufficient asphalt for durability considerations. The suggested VMAs for various maximum aggregate size of a blend are:

   * 15 percent for a 1/2-inch mixture,

   * 14 percent for a 3/4-inch mixture, and

   * 13 percent for a 1-inch mixture.

The maximum aggregate size is established on the basis that approximately 10 percent is retained on the "maximum size" sieve and 100 percent passes the next larger sieve for a standard nesting. The standard nesting is shown on the example discussed later.

3. Asphalt film thickness may range from 8 to 14 microns (4) if total asphalt content is used in the calculation and from 6 to 12 microns if asphalt absorption is considered. Those apshalt film thicknesses have been found in pavement surfaces that have shown good performance.

We reiterate that the target VMA and AV values are end points in the pavement and not for specimens compacted in the laboratory with present-day standard procedures.

The VMA of an aggregate blend is calculated from its gradation using the procedure described by Hudson and Davis (16). We are limiting the procedure to those aggregate blends that have a combined water absorption of less than 2.5 percent and to those that do not have highly textured surfaces such as certain manufactured aggregates and cinders. (Special mixture design criteria are used for these aggregates.) Also, for the present, we have accepted all -#200 sieve-size particles to have a VMA value of 32 percent. We assume this value to be a compromise between the VMA values for one-sized spheres ranging from the loosest (VMA = 47 percent) to the densest (VMA = 26 percent) conditions. The VMA of an aggregate blend is reduced from the 32 percent on the basis of ratios of percentages passing successive sieves from a specific nesting which includes the #200, #100, #50, #30, #16, #8, #4, 3/8", 3/4", and 1-1/2".

The surface area of the aggregate is required for the calculation of asphalt film thickness. The California surface area factors listed by The Asphalt Institute in Reference 18 are applied to amounts passing each of the same sieves listed above for the determination of VMA.

Sample Calculations for Optimum Asphalt Content

As mentioned earlier, no testing of the paving mixture is done. The aggregate and asphalt in the examples are described as follows and listed in Table 2. Also, the asphalt has a specific gravity of 1.020 and the aggregate blend has a "maximum" particle size of 3/4-inch.

A computer program has been developed for the calculations of VMA, SA, and also the total asphalt content by weight of mixture, as well as the asphalt film thicknesses that correspond to variable amounts of air voids (17). The film thickness is calculated using the effective asphalt content.

Input into the program are as follows:

1. Percentages passing the corresponding sieves,
2. Effective specific gravity of the aggregate blend,
3. Specific gravity of the asphalt, and
4. An assumed value for asphalt absorption of the aggregate.

Copies of computer printouts for the three trials listed are shown in Tables 3, 4, and 5 on the following pages.

Table 2 - Aggregate Characteristics

	Gradation		
	Total Percent Passing		
Sieve Size	Trial 1	Trial 2	Trial 3
1.5"	100	100	100
0.75"	93	86	90
0.375"	77	66	70
#4	65	52	55
#8	49	37	41
#16	35	24	26
#30	24	12	16
#50	25	5	9
#100	9	2	5
#200	5	1	2
Effective Specific Gravity	2.680	2.680	2.680
Asphalt Absorption (Assumed), %	0.6	0.6	0.6

Examination of Table 3 for Trial 1 shows the calculated final value of VMA to be 14.5 percent, which meets the criterion calling for a minimum value of 14.0 percent. If we believe that 14.5 VMA is too close to the minimum recommended, but acceptable, we can compensate by selecting an asphalt content corresponding to an air void value of 3 percent. That asphalt content would be 4.9 percent and the effective film thickness would be 7.4 microns.

If we were uncomfortable with the VMA value of 14.5 percent, then we would have opened the gradations perhaps to that shown as in Trial 2. Table 4 shows that the VMA was 18.1 percent and the SA was 12.4 square feet per pound. An upper limit for VMA has not been recommended;

Table 3 - Computer Output for Trial No. 1

Sieve Size	Percent Passing (P)	R	Voidage Reduction Factor (F)	Aggregate Voidage	Surface Area Factor	Surface Area (Sq Ft/Lb)
200.000	5.0	0.00	0.000	32.00	160.	8.00
100.000	9.0	1.80	0.940	30.08	60.	5.40
50.000	15.0	1.67	0.922	27.72	30.	4.50
30.000	24.0	1.60	0.911	25.24	14.	3.36
16.000	35.0	1.46	0.893	22.55	8.	2.80
8.000	49.0	1.40	0.891	20.09	4.	1.96
4.000	65.0	1.33	0.893	17.93	2.	1.30
0.375	77.0	1.18	0.917	16.44	0.	2.00
0.750	93.0	1.21	0.909	14.93	0.	0.00
1.500	100.0	1.08	0.969	14.48	0.	0.00

TOTAL SURFACE AREA = 29.32

Air Voids (Percent)	Apshalt Content (Percent)	Film Thickness (Microns)
2.00	5.26	8.09
3.00	4.86	7.36
4.00	4.45	6.63
5.00	4.05	5.91
6.00	3.63	5.18

EFFECTIVE SPECIFIC GRAVITY = 2.680
ASPHALT SPECIFIC GRAVITY = 1.020
ASPHALT ABSORPTION VALUE = 0.600

Table 4 - Computer Output for Trial No. 2

Sieve Size	Percent Passing (P)	R	Voidage Reduction Factor (F)	Aggregate Voidage	Surface Area Factor	Surface Area (Sq Ft/Lb)
200.000	1.0	0.00	0.000	32.00	160.	1.60
100.000	2.0	2.00	0.965	30.87	60.	1.20
50.000	5.0	2.50	0.013	31.28	30.	1.50
30.000	12.0	2.40	0.005	31.42	14.	1.68
16.000	24.0	2.00	0.965	30.31	8.	1.92
8.000	37.0	1.54	0.902	27.35	4.	1.48
4.000	52.0	1.41	0.891	24.37	2.	1.04
0.375	66.0	1.27	0.899	21.90	0.	2.00
0.750	86.0	1.30	0.894	19.59	0.	0.00
1.500	100.0	1.16	0.927	18.15	0.	0.00

TOTAL SURFACE AREA = 12.42

Air Voids (Percent)	Apshalt Content (Percent)	Film Thickness (Microns)
2.00	6.98	26.63
3.00	6.58	24.84
4.00	6.17	23.05
5.00	5.76	21.25
6.00	5.35	19.46

EFFECTIVE SPECIFIC GRAVITY = 2.680
ASPHALT SPECIFIC GRAVITY = 1.020
ASPHALT ABSORPTION VALUE = 0.600

however, as can be shown, the asphalt film thickness for up to 6 percent air voids is excessive at 19.5 microns and thus would be considered inadequate since the air-void values would be too high at lowered asphalt content and film thickness.

Trial 3 is suggested as a compromise in between the other two gradations. Table 5 shows a VMA of 16.2 percent for this aggregate blend. In reference to the criterion for film thickness, the data indicate an asphalt content of either 4.9 or 5.3 percent which correspond to final air-void values of 5.0 and 4.0 percent, respectively.

Table 5 - Computer Output for Trial No. 3

Sieve Size	Percent Passing (P)	R	Voidage Reduction Factor (F)	Aggregate Voidage	Surface Area Factor	Surface Area (Sq Ft/Lb)
200.000	2.0	0.00	0.000	32.00	160.	3.20
100.000	5.0	2.50	0.013	32.43	60.	3.00
50.000	9.0	1.80	0.940	30.48	30.	2.70
30.000	16.0	1.78	0.937	28.56	14.	2.24
16.000	26.0	1.63	0.915	26.13	8.	2.08
8.000	41.0	1.58	0.907	23.71	4.	1.64
4.000	55.0	1.34	0.891	21.13	2.	1.10
0.375	70.0	1.27	0.898	18.98	0.	2.00
0.750	90.0	1.29	0.896	17.02	0.	0.00
1.500	100.0	1.11	0.953	16.21	0.	0.00

TOTAL SURFACE AREA = 17.96

Air Voids (Percent)	Apshalt Content (Percent)	Film Thickness (Microns)
2.00	6.06	15.61
3.00	5.66	14.40
4.00	5.26	13.19
5.00	4.85	11.98
6.00	4.43	10.77

EFFECTIVE SPECIFIC GRAVITY = 2.680
ASPHALT SPECIFIC GRAVITY = 1.020
ASPHALT ABSORPTION VALUE = 0.600

Table 6 - Summary Data from New Design Method.

Gradation	VMA, %	Void, %	Content, BTW, %	Film Thickness
Trial 1	14.5	3.0	4.9	7.4
		4.0	4.5	6.6
Trial 2	18.1	6.0	5.3	19.5
Trial 3	16.2	4.0	5.3	13.2
		5.0	4.9	12.0

Table 6 shows a summary listing of the salient values of the calculations discussed above.

Now, one must select a specific value of asphalt content for initiating laboratory stability testing, which usually includes a minimum number of mixtures at plus-and-minus 0.5 percent asphalt from the calculated optimum amount. For the gradations shown, we would recommend as follows:

a. Trial 1 - 4.9 percent

b. Trial 2 - Not acceptable

c. Trial 3 - 5.3 percent

It is apparent that due to acceptable ranges of VMA, AV and film thickness and their interrelation, a certain amount of experience in mixture design is required to select the calculated optimum amount of asphalt for the paving mixture. Since the recommended minimum and maximum values for the design parameters are for a potential end point condition in a road, one must accept that values for VMA and AV for laboratory design must be different to allow for traffic compaction of the mixture.

Basis for Selection Design Asphalt Content

Data resulting from measurements of cores taken from existing pavements have indicated certain relationships between performance and values of VMA and AV. Additionally, it has been found that core densities were higher than the corresponding laboratory compacted values. The recommendations made for selecting a design asphalt content are based in consideration of laboratory duplication of pavement densities. However, at the present, this duplication is not possible, yet we must now make specific recommendations for laboratory mixture design criteria.

The mixture design criteria are based on the following assumptions:

1. Mixing temperature of 275-285°F followed with loose curing of mixture for 15 hours at 140°F.
2. Compaction temperature of 250°F with 75 B/F of the Marshall mechanized device to meet ADOT procedure.
3. The aggregate blend will have a water absorption value of less than 2.5 percent.
4. The effective specific gravity of the aggregate will be used and determined with the Rice value for the loose cured mixture.

The requirements of the compacted mixture for selecting the design asphalt content are as listed below and are for aggregate blends of 3/4- and 1/2-inch "maximum" particle size:

1. Hveem Stability, 140°F, dry, min 40

2. Marshall Stability, 140°F wet, lb. min 1500

3. Marshall Flow, 140°F wet, 0.01 in. 8-16

4. Air Voids, % 4-6

5. VMA, %, min

   (1/2-inch aggregate) 17

   (3/4-inch aggregate) 16

The Hveem stability (a measure of frictional strength) is to be performed before the Marshall test (a measure of tensile strength). Its minimum value of 40 is set temporarily until sufficient data are obtained for determining effects of Marshall compaction.

If laboratory specimens do not meet design criteria or if there is a change in gradation, then the mixture should be re-examined with the calculations of the theoretical procedure. It is anticipated that a new design procedure will need adjustments as information is obtained for its verification in estimating the design asphalt content.

Construction Control of Paving Mixtures

The present ADOT procedures for the construction control of paving mixtures are considered appropriate. However, since some additions have been proposed for the laboratory design practice, these have to be reconciled in the control measurements. The present controls and additions are as follows:

1. Aggregate gradation.

2. Asphalt content.

3. Compaction of mixture on the roadway must be such that air-void content value is a maximum of nine percent based on the effective specific gravity of the aggregate; i.e., the "Rice" specific gravity of the mixture.

4. Stability control of the paving mixture to be based on a 1,500-pound Marshall.

The following portion of the report is concerned with discussions of the original design considerations for the recycled mixtures and also comparisons with the physical properties of cores taken from the recycled pavements.

DISCUSSION OF TEST RESULTS FROM RECYCLED PAVEMENT CORES

The following paragraphs present a description of the design considerations and comparisons for each of the seven test sites sampled. Data for each of these are located in the tables of Appendix A.

It is of general knowledge that laboratory design properties of asphaltic concrete are not always the same as for the mixture produced at the plant for construction. These differences are due to variations in proportioning of aggregates and asphalt, as well as the specific source of asphalt used. Additionally, because of construction difficulties (e.g., related to compaction and placement), the aggregate blend and asphalt content may be modified but yet satisfy specification requirements.

In requesting laboratory design data for the various recycling projects, ADOT would submit a minimum of three Laboratory Bituminous Mixture Design forms showing results of tests that had been performed on materials used or anticipated for use on the project. Table A-1 in Appendix A presents a typically completed form. In some cases, the laboratory design mixtures were changed as the project progressed. As a consequence, we were forced or compelled to select the laboratory work sheet that contained data most comparable to the physical properties of the cores taken from the roadway. This method of selecting the laboratory design data could lead to making comparisons of results that did not correspond to the same material and proportions used for both the laboratory and field.

The comments of pavement surface condition shown on Table A-2 are based on observation and measurements made at the location for coring. The rutting observed may or may not have originated in the recycled mixture evaluated, since none of these was a surface course and no measurements were made of any of the surface courses. However, it is noted that there was no bleeding of the surface near the cored sites.

Test Site No. 1 - Sentinel

ADOT's investigation of the old roadway showed the primary type of distress was that of alligator cracking with a minor amount of rutting. The decision to rehabilitate by hot-mix recycling was based on (a) limitation of funds, (b) failures were predominantly in the travel lane, and (c) the desire to evaluate the recycling method (6).

The Sentinel test site was one that was added at a later date as a replacement to one that had been eliminated from the Flagstaff area. As a consequence, there was a lack of documented data on pavement sampling and after construction evaluation.

Upper Lift

Listings of the design and core data are shown in Table A-2, parts 1 and 2, which are located at the end of the report in Appendix A. The tables show that cores were taken across the eastbound travel lane at two locations approximately 0.7 of a mile apart.

Part 1 of the table has the results of measurements made on the second layer (below the ACFC) which was 1-1/8 inch in thickness. Marshall stability and flow were of comparable values for the two locations; however, there was a difference in asphalt content and, therefore, also air-void content. The notation next to "Remarks" states that there was rutting of the pavement surface, and we generally associate bleeding and rutting distresses when air-void contents are below two percent, if calculated with the effective specific gravity of the aggregate.

If bulk specific gravity of the aggregate is used for the determination of air-void and VMA, then these values would be lower than if the effective specific gravity had been used for those calculations.

Design and core values for stability and flow should not be compared directly since different methods were used for those measurements. However, comparisons can be made between density and air-void values.

Analysis of the data suggests that rutting at the time of sampling was caused by the following conditions:

1. The Asphalt Institute recommends a minimum VMA of 14 percent for a 3/4-inch gradation (18). Measured VMA's ranged from 14.3 to 17.6 percent, the corresponding air-void content from 0 to 2.3 percent, and binder content ranged from 6.2 to 7.6 percent. It seems that the 2.5 percent of added cyclogen resulted in too high a binder content and perhaps resulting in too low a binder viscosity.
2. It is interesting to note that the theoretically calculated VMA values (13.3 to 13.8%) suggested a gradation that could not have tolerated an added binder content of 2.5 percent.

Gradation curves of the ADOT design and of the extracted aggregate from the cores taken are shown on Figure B-1 of Appendix B. The figure shows the close agreement in gradation between the design and the mixture produced at the plant. The core aggregate indicates a finer gradation at the coarse end of the curve. This characteristic is expected because mainly the plus 3/8-inch aggregate would be reduced by coring and trimming the core to size for stability testing.

Lower Lift

In Table A-2, part 2, the data shown are for the third layer from the surface of the road. The test results are comparable to those of part 1, except that densities and measured air voids are a bit lower. As a consequence, Marshall stabilities are lower and flow values are higher for the cores. The variations in stability may be due to the use of height correction factors, since the upper lift cores were 1-1/8 inch thick and the lower ones were 2-1/2 inches.

In Figure B-2, one can see the similarity between the design and extracted gradations.

The major difference between the two recycled layers was in the value of the viscosity of the recovered binder. The large difference between the viscosity values of approximately 1,200 and 13,000 poises is attributed to the recovery procedure which held the asphaltic binder in solution with the methylene chloride for periods ranging from three to seven days.

A review of the ADOT design data suggests that the quantity of the recycling agent used was based on the testing of specimens to meet strength and voids criteria. However, the type and proportion of recycling agent to old asphalt was obtained through the use of a blending chart such as the one illustrated on Figure B-3. The example for usage shown on the chart indicates that the viscosity of the old asphalt had a value of 10^5 poises and the viscosity of the recycling agent was equal to approximately 60 poises. The desired viscosity was one comparable to new asphalt having a value of 2×10^3 poises. From the 2×10^3 viscosity mark, a line was drawn horizontally to intersect the one connecting the first two given viscosities. Then from the intersection point a vertical line was drawn to show that new binder was to consist of 45 percent of the recycling agent.

Test Site No. 2 - Rillito

ADOT's early examination of this roadway indicated that the Rillito pavement was severely distressed in the travel lanes in the form of cracking, flushing, and rutting. Recycling of the old pavement surface provided an economical means of rehabilitation, since it minimized the thickness of new overlay that would have been necessary. A 10-mile stretch was to be recycled and sampling was of 6-inch diameter cores taken one per mile.

West Bound

Data for the design and core testing values are given in part 3 of Table A-2. The ADOT design was performed using the Hveem procedure and criteria applicable in 1979. Of immediate note is the reported laboratory VMA value of 13.2 obtained with the Hveem design, and so the design should have been questioned. Perhaps, it is fortuitous, but it is interesting to note the similarity in values for VMA obtained in the laboratory specimen, pavement cores, and by calculation with the theoretical procedure. The low values of VMA and air-void content, along with the five percent binder content are sufficient to explain why the pavement surface showed rutting and bleeding distress at the time it was sampled in 1983.

However, it must be pointed out that the recycled material was the third layer down from the surface and so the distress observed could have been due to failure of the virgin mixture placed above the recycled material.

As noted earlier, viscosity values for the recovered binder are suspect because of the long period of time required to extract it from the paving mixture.

The gradation curves presented in Figure B-4 show that the construction gradation was a bit coarser (less fines) in the sizes smaller than the No. 4 sieve. However, the differences between design and construction values were generally within construction tolerance limits.

East Bound

The design and core results data are given in part 4 of Table A-2 and graphed in Figure B-5. In general, the same comments as made for the west bound lane are applicable to this sampling. The low VMA and air-voids values seem to warrant the rutting of the pavement surface. The curves of Figure B-5 indicate that field adjustments were made to the gradation between the construction of the east and west bound lanes.

The mixture for both directions of the roadway was composed of 70 percent recycled asphaltic material and 30 percent virgin aggregate. The selections of the amount and gradation of the virgin aggregate appear to have been made on the basis of minimizing air pollution problems during construction and obtaining a combined aggregate resulting in a dense gradation.

Test Site No. 3 - Dateland

The recycled mixture was designed in 1982 using the Marshall method and AC-20 as the added bitumen. Also, a 50/50 blend of old asphaltic concrete and virgin aggregate was selected for the new material. It would appear that by this date ADOT had accepted a recycled mixture design using approximate 50 percent virgin aggregate and asphaltic cement for the recycling agent. The data on part 5 of Table A-2 show that the recycled mixture was used in the trench and overlayed with new asphaltic concrete. Sampling of the pavement surface at the age of two years showed no distress at either of the two sites that were cored.

The results of testing for the two sections as presented in the table indicate some differences in density and air-void content. From the curves of Figure B-6 it can be seen that the gradation of the extracted aggregate from the cores taken at mile post 64.0 was more open than that at mile post 59.0. As a consequence, the air-voids were lower for the cores from mile post 59.0 and would be susceptible to failing in strength. This concept is reinforced with the high flow values ranging from 15 to 24 units.

Marshall stability values were relatively high for all of the cores taken. The high flow values are of some concern related to a rutting failure; however, the mixture was placed in a trench section and the shear stress level may be low enough to preclude a shear failure by rutting.

Test Site No. 4 - Firebird Lake

The design data for this recycled mixture are comparable to that for Test Site No. 3 - Dateland, except that the Marshall stability was quite high at 5,132 pounds. Inspection of the roadway at the time of coring showed distress of rutting at both the inner wheel and outer wheel paths. Measured values for VMA and air-void would not serve as evidence or reason for the distress since both are at accepted levels. However, the calculated values based on the new theory were low enough to suspect distress by rutting.

Test Site No. 5 - Gila Bend

Inspection of the data presented in Table A-2, part 7, shows that cores were taken from the traffic lanes for both east and west bound directions. Reviews of the laboratory design sheets and responses to the questionnaire did not yield sufficient information to specifically identify the cores that represented a recycled mixture or a virgin mixture. Our review of the core data suggest that the west bound lane contained the recycled material because of the higher values of -#200 fines and higher recovered binder viscosity. The gradation curves of Figure B-8 indicate that the gradations of the aggregate for the two sites were quite similar, especially for the fines below the No. 16

sieve. But the coarse portion of the gradation for the material in the west bound lane corresponds closer to the ADOT design than did the aggregate from the east bound lane.

The measured values for VMA and air-void content, 14.0 and 2.0 percent respectively, suggest that the virgin mixture in the east bound lane would be more susceptible to rutting and bleeding than the recycled mixture placed in the west bound lane. However, as noted, there was no rutting. It should be mentioned that the construction has been in service approximately one and one-half years at the time of sampling, and also that the layers in question are approximately two inches below the surface of the pavement.

Test Site No. 6 - Williams Field

The test site was sampled across the total width of the west bound lanes; however, only one 4-inch core was taken at the OWP and BWP of the passing lane. The portions of the passing lane cores evaluated for gradation, stability, and asphalt content were the third layer down from the surface. The results of those measurements plus the viscosity of the recovered asphalt (+ 300,000 poises) indicate that the old paving material had been tested from the passing lane. These data are shown on Table A-2, part 8, and Figure B-9.

Inspection of the table shows that the project was constructed during the summer of 1983 and was cored for evaluation in December of 1983. The roadway was trafficked for about 6 months, mainly during the cool period of the year. Although the measured VMA's and air-void content were at relatively safe values of 14.5 and 3.8 percent, respectively, we would be concerned that the recycled layer will be over compacted by traffic within a period of time during the summer months of 1984 and 1985. The result of this overcompaction would manifest itself as rutting of the surface.

Test Site No. 7 - Red Rock

A portion of I-10 between Red Rock and Picacho was reconstructed with its recycled mixture during April to June of 1985. Two layers of the recycled mixtures were sampled in March of 1986. Characteristics of the cores taken are shown in Table A-2, Part 9 along with ADOT's design properties. As noted, cores were taken on the west bound traffic lane, and the trench and surface layers were tested separately. The properties of the cores represented those after about one year of traffic loadings.

Inspection of the table and Figure B-10 shows that there were some acceptable differences between individual values of gradation for the cores and the design values. However, these differences went from positive at the coarse end to negative at the fine end of the distribution. These differences resulted in variations for calculated and also measured values for VMA and AV between the design gradation and core gradations.

At less than one year of traffic, the average core density is equal to the laboratory compacted density. The core data show that the average measured VMA was 15.7 percent and for the calculated potential VMA, it was 14.1 percent. The AV values were 5.9 and 4.2 percent respectively. Should traffic for the next several years reduce the VMA of the 1986 cores to the calculated potential value by 1.6 percent, then the air-void value of the cores will be reduced to 4.3 percent. The implication of this result is that failure of the mixture by bleeding or rutting is not likely since the final AV is greater than 2 percent. However, there is a note of caution in that the theoretical calculations show that at an asphalt content of 4.3 percent the film thickness is on the low end of our criterion; especially in using AC-40 as the added binder. The concern in this case is that the recycled paving mixture will be brittle and susceptible to cracking.

An attempt was made to design the new paving mixture with the recycled aggregate and the virgin aggregate used in the ADOT design. The following listings in Table 7 show the gradations considered and the resulting potential VMA values. As can be seen, no combination of recycled and virgin aggregates would yield the desired theoretical gradation.

TABLE 7: Gradations and VMA Values for Test Site 7

Sieve Size	Recycled Aggregate	Virgin Aggregate	U. of A. Theoretical Aggregate	ADOT 50/50 Design
		Total Percent Passing		
1-1/2"	100	100	100	100
3/4"	99	96	100	98
1/2"	91	71	95	81
3/8"	85	61	90	73
4	66	47	75	57
8	48	35	60	43
16	35	24	40	32
30	23	14	20	21
50	14	6	15	13
100	10	2	10	9
200	7.2	1.3	6	7
Potential VMA %	13.5	18.0	15.9	13.3

A review of the data presented in all parts of Table A-2 shows that most of the recycled mixture samples came from a trench section rather than from a surface course. Table 8 shown on the next page is a summary of the range of values for VMA, air content, and binder content obtained for the core samples from the seven sites. Also, Figure 3 shows the limits of all aggregate gradations extracted from the cores.

As can be seen from the table and gradation bands, most mixtures could be classified as being of maximum density and with the amount of binder used, would be susceptible to flushing as indicated by the low values of air-void content.

Test Site No. 8 - Willcox

Specifically, this was not a test site as described for the other six. For this location, the old pavement surface material was analyzed by this laboratory for developing a recycling mixture design prior to construction.

Sixteen 6-inch diameter cores were sent to this laboratory for evaluation of a seven-mile stretch of I-10 Bypass of Willcox. Figure 4 shows the scheduling of the cores and Table A-3 in Appendix A shows results obtained from the extraction and recovery procedures. As indicated in the table, the cores were divided into three groups for testing; that is, the upper one-half inch of the asphaltic concrete friction course (ACFC), the next two-and-one-half inches of asphaltic concrete, and the upper three inches of the core containing both the ACFC and the asphaltic concrete.

TABLE 8 - Range of Values for Recycled Pavement Cores

Test Site	Recycle/Virgin Ratio	Recycling Agent	VMA %	Air Void, %	Binder Content, %
Sentinel, Upper	100	Cyclogen	14-17	0-2	6.2-7.6
Sentinel, Lower	100	Cyclogen	15-16	0-1	6.6-7.2
Rillito, WB	70/30	Cyclogen	14-15	0-1	4.9-5.3
Rillito, EB	70/30	Cyclogen	12-13	0-2	4.8-5.1
Dateland	50/50	AC	15-18	3-6	4.7-5.7
Firebird Lake	50/50	AC	15-16	3-4	4.9-5.5
Gila Bend	50/50	AC	13-17	2-5	4.8-5.1
Williams Field	50/50	AC	14-15	3-5	4.6-4.8
Red Rock	50/50	AC	14-17	4-7	4.2-4.8

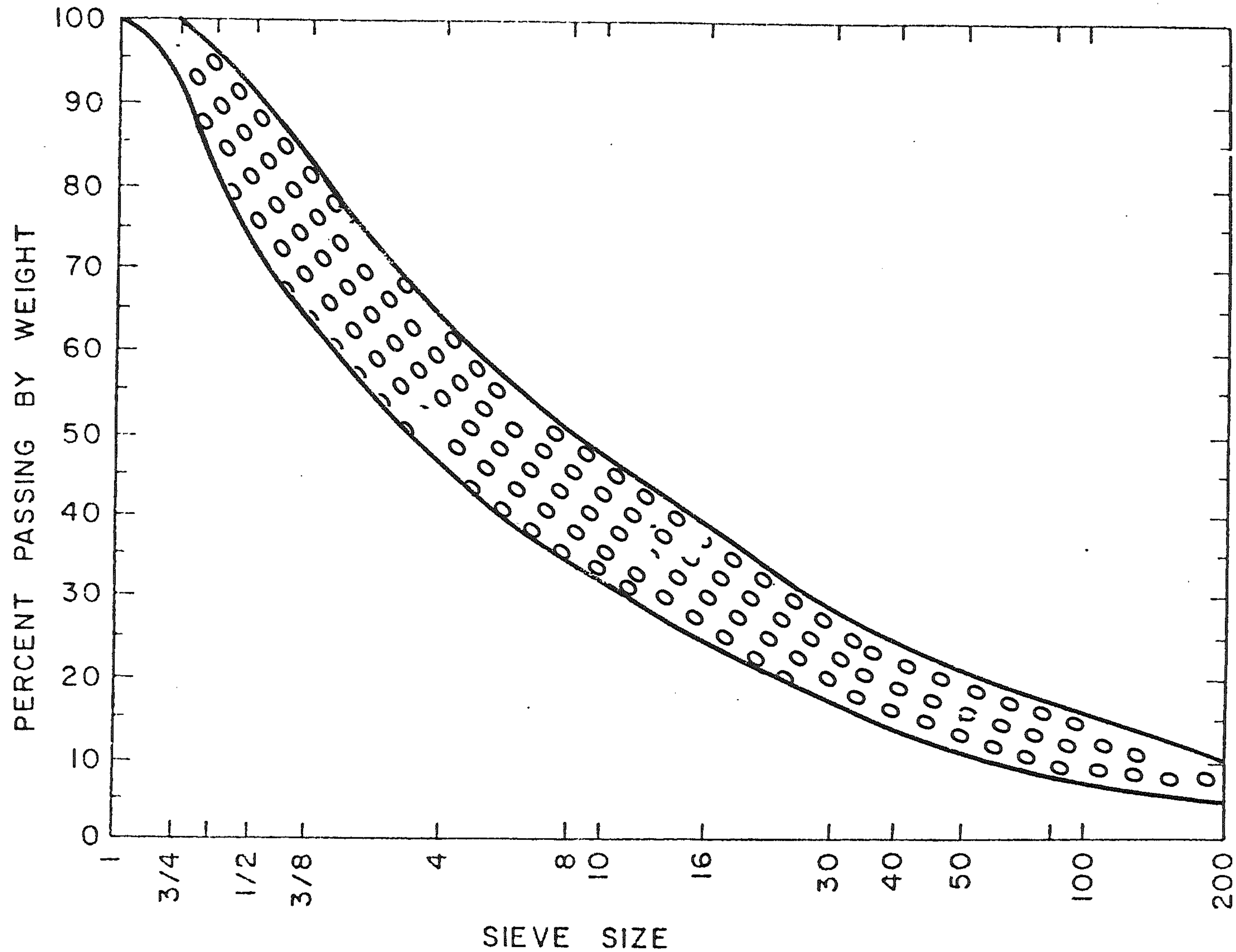

Figure 3. Limits of All Gradations from Extracted Cores.

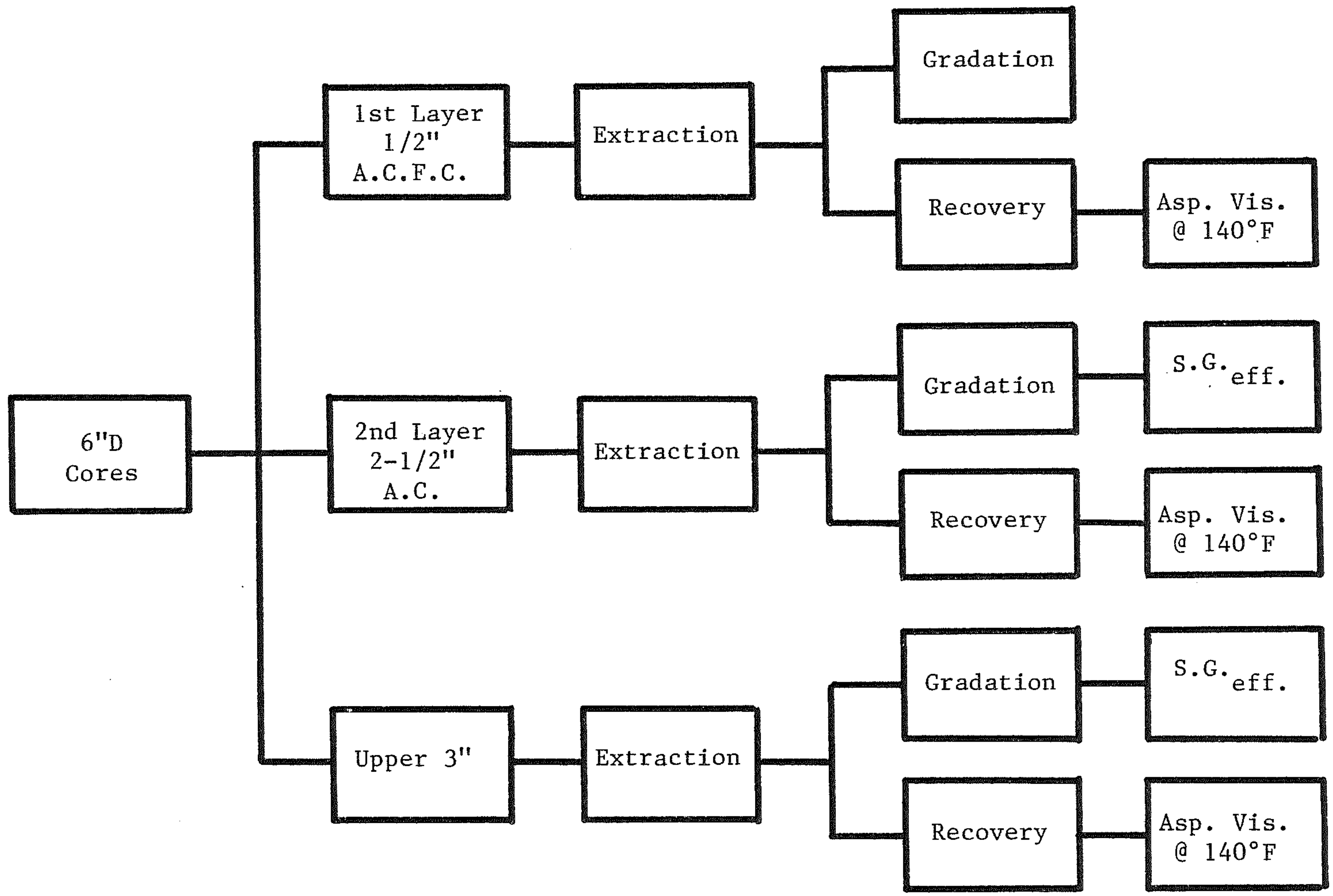

Figure 4. Flow Chart for Cores Taken From I10-Willcox Bypass

The cores were sliced to the thicknesses mentioned in order to determine if it would be practical to combine the separate aggregate gradation for each layer to determine the best thickness combinations of both layers with a new virgin aggregate. ADOT had established that the old pavement was going to be milled down a thickness of three inches which then fixed the ratio of ACFC to old asphaltic concrete, but then there would be no proof that this would be the optimal usage and design of the recycled mixture.

Measurements on the components of the cores were made for aggregate gradation, aggregate specific gravity, asphalt content, and asphalt viscosity at 140°F. Examination of the data shows that the variability of gradation in the upper three inches of the cores, as well as the asphalt content, was surprisingly low. However, it is noted that the gradation of the upper one-half inch of the cores was not representative of values specified for an ACFC.

In the desire to characterize each layer of the cores individually, there was concern over the capability of separating the ACFC from the next lower layer. The following listings (below) show the results obtained from combining the ACFC with the two-and-one-half inch layer of asphaltic concrete below it and then comparing the asphalt content and gradation values with those obtained from the upper three inches of the cores. The calculations are based on the assumption that the ACFC constituted 25 percent of the upper three inches of the cores if based on density rather than on dimension.

Asphalt Content:

Combining Two Layers	$\frac{0.75 \times 6.87 + 2.25 \times 5.1}{3}$	=	5.5
One Three-Inch Layer		=	5.7

Gradation:

	Total Percent Passing Sieves									
	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200
Combining Two Layers	100	90	82	62	44	32	24	17	12	8.6
One Three-Inch Layer	100	91	85	63	44	31	23	16	11	8.1

It is recognized that only one mixture has been treated as indicated above, and so the close agreements are not completely acceptable without reservation. However, the similarities are thought to be significant enough for future consideration in using the characterization of the separate layers for developing both recycled mixture and structural designs which include variable depth of milling of the old pavement.

Design for the Recycled Mixtures

The following paragraphs describe the process used to develop a recycled mixture design for the Willcox Bypass. Also, there will be shown a comparison between mixture design values based on theory with those obtained by field adjustments of ADOT design for construction conditions.

The design problem was to obtain the amount and gradation of the virgin aggregate to be combined with the old pavement surface milled from the pavement. Also, it was necessary to determine the kind and amount of asphaltic binder to be added to the total mixture. The ADOT design was to use a 50/50 mixture of the upper three inches of the old pavement and virgin aggregate on a weight basis and AC-20 for the binder; therefore, those conditions were used by this laboratory for the design based on theory and without laboratory testing. The theory used was that described earlier for the computation for VMA and control of both AV and asphalt film thickness. The combined total aggregate gradation, JMF, was selected to meet the limits of ADOT's old MA-3 gradation and also yield a theoretical value of VMA greater than 15 percent. Having a specific gradation and that the virgin aggregate was to be 50-percent of the total blend, then the virgin aggregate gradation was calculated. The calculated VMA for the JMF was 16.4 percent. In consideration that the old pavement cores had 5.7 percent asphalt and the desire to control air-voids in the mixture after traffic compaction, the calculated amount of added asphalt was 2.5 percent by total weight of mixture. The basic numbers for gradation and asphalt content calculated are given below.

Top Three Inches of Pavement Cores

Sieve	Gradation 3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200	Asphalt Content, %
% Passing	100	91	85	63	44	31	23	16	11	8.1	5.7

Theoretical Virgin Aggregate for 50/50 Blend

Sieve	Gradation 3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200	Asphalt Content, %
% Passing	100	97	95	87	76	49	21	14	9	2	

Theoretical-Job-Mix Formula

Sieve	Gradation 3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200	Asphalt Content, %
% Passing	100	94	90	75	60	40	22	15	10	5	2.5

It should be recognized that the gradation for the virgin aggregate was a theoretical one and might not be met with aggregates available near the construction area.

After the contractor had established his stockpile of virgin aggregates, samples were sent to the central laboratory of ADOT and also to this laboratory. The stockpile aggregates were combined to approach the theoretical gradation for the virgin aggregate. The final laboratory JMFs for ADOT and the University of Arizona are as follows:

JMF for Recycled Mixture

	Gradation										Asphalt
Sieve	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200	Content, %
ADOT Lab, % Passing	96	82	76	55	40	28	20	12	8	6.1	1.3
UofA Theory, % Passing	100	96	92	73	48	38	26	14	9	6.4	2.5

The reader is reminded that in the above comparison, the University of Arizona's (UofA) job-mix-formula was based on theoretical considerations without the benefit of mixing and strength testing of the recycled mixture.

Verification of the Designed Recycled Mixtures

The data available for verification of the designed mixtures are extremely limited, especially since there were many adjustments made in the field to the ADOT JMF. The listing in Table 9 for gradation and asphalt content is insufficient to reach definite conclusions, but should serve to illustrate the strong capability of the theoretical procedure for mixture design.

TABLE 9 - Gradation and Asphalt Content for Recycled Mixtures

Sieve	Theoretical UofA JMF	Laboratory ADOT JMF	Field Laboratory Extraction	UofA Extraction of Cores Taken by ADOT A(4"D)	B(6"D)
3/4"	100	96	96	98	95
1/2"	96	92	88	92	86
3/8"	92	76	82	86	80
#4	73	55	60	65	62
#8	48	40	48	51	49
#16	38	28	--	39	37
#30	26	20	--	29	28
#40	--	--	24	--	--
#50	14	12	--	18	18
#100	9	8	12	13	13
#200	6.4	6.1	8	8.4	8.3
Total Asphalt Content,%	5.3	4.0	3.9	4.9	4.8

Discussions with field personnel of ADOT indicated that the original ADOT JMF was too harsh and the required field compaction could not be achieved. Corrective action was taken to modify the virgin aggregate gradation by increasing the amount of the -#4 fines and increasing the specified asphalt content. Examination of Table 9 shows that the corrective action taken approached the JMF suggested by the University of Arizona which was based on only theoretical considerations.

CONCLUSIONS AND RECOMMENDATIONS

The examination of the few recycled pavements included in the program did not yield a great deal of data. However, the information obtained was quite consistent, and thus would seem to warrant the following conclusions:

1. The majority of the recycled material had been placed in the trench and not as a surface course. As a consequence, they were not subjected to the higher pavement stresses.
2. The majority of the cores taken from sections containing cyclogen had low values for air voids in that they ranged from 0 to 2 percent.
3. The Marshall stability values of the cores were satisfactory in that they exceeded 1500 pounds; however, some of the flow values were high - in excess of 16 hundreths of an inch.
4. Although the surface courses were not evaluated, it is considered very likely that the recycled asphaltic concrete sampled contributed to rutting of the pavements. This statement is made in view of some of the low values for air voids, high values for flow, no bleeding, and that most of the sections had not received more than two years of traffic. Rutting of pavements is a concern when the depression is larger than from 0.2 in. (20) to 0.4 in. (21), since hydroplaning can occur when water ponds at a depth of 0.2 in. and structural cracking can be initiated along the ridge of a 0.4 in. rut.

5. The literature review and ADOT's experiences show that the Marshall method of asphaltic mixture design can be used for recycled mixtures. The data show that the density of all of the recycled cores was higher than the laboratory design density obtained with either the Hveem or Marshall compaction.

6. Design and construction aggregate gradations were quite similar, thus indicating good control on the proportioning of the aggregate and recycled material bins. In general, the gradations were such that minimum desired theoretical VMA values were approached in the pavement cores.

Recommendations offered are based on the program and are listed below:

1. A minimum of two 6-inch diameter cores of the full depth should be taken per lane mile. Enough of the old pavement should be taken in order to quantify the amount and kind of materials in each layer considered for recycling.

2. ADOT should consider utilizing other than the recent 50-50 proportioning of old pavement and virgin aggregate for recycled mixtures. The ratio of the quantities for the two materials would be dependent on the quantity and variability of the old pavement material available for the new construction.

3. The continued use of asphalt cements of grades AC-10 or 20 is recommended, especially when the amount of virgin aggregate exceeds about 30 percent of the total mixture.

4. The selection of the gradation of the virgin aggregate and/or new asphalt content should be such that VMAs and air-voids be higher than found in the pavement cores. We believe the core (3/4" aggregate) VMAs should have been a minimum of 15 percent and the air-voids at a minimum of two percent for the older pavements and higher for the newer ones in order to accommodate traffic compaction. Although the data are somewhat limited, they suggest that the present criteria for virgin asphaltic concrete may not be totally applicable for recycled mixtures. As a consequence, we recommend that the directions taken by this research be continued for <u>surface</u> mixtures to obtain design parameter values for comparing laboratory and field conditions.

5. The University of Arizona's procedure for mixture design has not been fully verified; however, we recommend that it be used jointly with ADOT's method in future recycling jobs.

REFERENCES

1. "Recycling of Asphaltic Concrete," a report by R.M.I. Systems of Las Vegas, Nevada, 1974.

2. "Nevada Recycles a Highway," Paving Forum, Winter, 1974.

3. "Recycled Asphalt Concrete," Implementation Package 75-5, U.S. Department of Transportation, Federal Highway Administration, September 1975.

4. Dunning, R. L. and R. L. Mendenhall, "Design of Recycled Asphalt Pavements and Selection of Modifiers, ASTM Special Technical Publication 662, December 1977.

5. "Recycling Materials for Highways," The Asphalt Institute, Manual Series No. 20, August 1981.

6. Ritter, John B., "Pavement Recycling Project I8-2(76), Yuma County Line-Gila Bend," Demonstration Project 39, NEEP 22 Arizona, Arizona Department of Transportation, October 1978.

7. "Symposium-Recycling of Asphalt Mixtures," Proceedings, Association of Asphalt Paving Technologists, 1979.

8. "Proceedings of the National Seminar on Asphalt Pavement Recycling" TRR-780, Transportation Research Board, 1980.

9. "Guidelines for Recycling Pavement Materials," NCHRP Report 224, Transportation Research Board, 1980.

10. Zywiak, Stephen J., "Recycling Asphalt Pavements in the State of New Hampshire," FHWA Demonstration Project No. 39, FHWA-DP-39-33, April 1982.

11. "Pavement Recycling: Stopgap or Remedy," Civil Engineering, September 1984.

12. Kennedy, Thomas W. and Roberts, Freddy L., "Quality Assurance Considerations in Design of Recycled Asphalt Mixture; TRR 885, Transportation Research Board, 1982.

13. Brown, Elton, R., "Evaluation of Properties of Recycled Asphalt Concrete Hot Mix," Technical Report GL-84-2, U.S. Army Engineer Waterways Experiment Station, 1984.

14. American Association of State Highway and Transportation Officials, AASHTO Materials, Part II Tests, 13th Edition, 1982.

15. State of Arizona Department of Transportation, Materials Testing Manual, 1984.

16. Hudson, S. B. and Davis, R. L., "Relationship of Aggregate Voidage to Gradation," Proceedings, Association of Asphalt Paving Technologists, 1965.

17. Jimenez, R. A. and DaDeppo, D. A., "Asphaltic Concrete Evaluation in Arizona," Report in Progress for Arizona Department of Transportation.

18. "Mix Design Methods for Asphalt Concrete," Manual Series No. 2, The Asphalt Institute, 1979.

19. "Hot-Mix Recycling Manual," Asphalt Division, Chevron, U.S.A., 1981.

20. Monismith, C. L. and McLean, D. B., "Design Considerations for Asphalt Pavements," Report No. TE 71-8, University of California, Berkeley, 1971.

21. Barksdale, Richard D., "Practical Application of Fatique and Rutting Test on Bituminous Mixes," Proceedings, Association of Asphalt Paving Technologists, 1978.

APPENDIX A

LIST OF TABLES

Table A-1. Typical Asphaltic Mixture Design Data from ADOT.

MATERIALS DIVISION

LABORATORY BITUMINOUS MIXTURE DESIGN
Pit No. RECYCLE AC W/ AB

Received | Test Commenced | Material Recycle AC w/AB
Identification | Sampled | Lab. No. 77-31662
Project No. I-8-2(56)
Quantity
Location of Supply
Project Name | Contractor
Specifications Governing

	MATERIALS SURVEY(PE) PRELIMINARY DESIGN GRADING						AS PRODUCED (CONSTRUCTION) FINAL ADJUSTED DESIGN GRADING				
	ORIG. PIT AVERAGE		CRUSHED GRADATION		ADJ. CRUSH GRADATION		AS CRUSHED ON PROJECT		AS ADJ. IN LABORATORY		SPEC. LIMITS SOUGHT
Sieve	% Ret	% Pass	% Ret	% Pass	% Ret	% Pass	Composite		Composite		
3" Slot								%		%	
3"								%	AC	75 %	
2½"			EXTRACTED					%	AB	25 %	
2"	AB		AC					%		%	
1½"							% Ret	% Pass	% Ret	% Pass	
1"	0	100	0	100					0	100	
¾"	4	96	2	98					3	97	
½"	11	85	11	87					11	86	
⅜"	10	75	11	76					10	76	
¼"	10	65	11	65					11	65	
No. 4	7	58	8	57					8	57	
8	12	46	16	41					15	42	
10	3	43	3	38					3	39	
16	6	37	8	30					8	31	
30	6	31	8	22					7	24	
40	3	28	2	20					2	22	
50	4	24	3	17					3	19	
100	7	17	6	11					7	12	
200	5	12	4	7					4	8	

SP. GR. Coarse Aggr. 2.527 | SP. GR. Fine Aggr. 2.607 | SP. GR. Comb. 2.573
Absorp. Coarse Aggr. 2.72 % | Absorp. Fine Aggr. 1.75 % | Absorp. Comb. 2.17 %
O.D. SP. GR. 2.460 | O.D. SP. GR. 2.562 | O.D. SP. GR. 2.518

	LL	PL	PI	Swell 24 hr.	Swell 48 hr.	S.E.
Natural Fines	%	%		in.	in.	
Crushed Fines	%	%		in.	in.	

ABRASION% Loss | 100 Rev. | 500 Rev.

REMARKS

DESIGN BASED ON CONCEPT OF RECYCLING 3INCHES OF AC AND 1 INCH OF AB.

AB CKE DEMAND 6.1% X 25% = 1.5   2.5-1.5 = 1.0 AVAILABLE FOR SOFTENING

1.0/1.0+3.75 = 20% BLEND
COMPARED TO
2.5/6.25 = 40% BLEND

DESIGN DATA

Specimen	A	B	C	D	E	F	Design Sp
Temperature							
Bulk Density Lbs. Per Cu. Ft.			136.9	139.5			
Bit. Grade Cyclogen M	1.0	1.5	2.0	2.5			
% of Bit. .75 X 5.0 = 3.75	5.75	5.25	5.75	6.25			
Density Lbs. Per Cu. Ft.							
Stabilometer	49	53	52	48			
Cohesiometer	206	174	169	293			
Voids			6.5	4.1			
V.M.A.			17.9	16.8			
% Voids Filled			63.7	75.6			
Effective Asphalt Total Mix							

Sample	Air PSI	H_2O PSI	Retention	
No. 1	334	93	28 %	2.5 % Cyclogen M Asphalt
No. 2			%	% Anti-Strip Agent
No. 3			%	% Dry Lime
No. 4			%	% Dry Cement

Max. Density 145.5 Lbs. Per Cu. Ft. (Rice Method) 6.25
Asphalt 0.50 % Absorp. on dry aggregate ESTIMATED
Film Strip. (Course Aggr.) =

C.K.E. Values F= C=
Theoretical Bit. Ratio for Grade Bitumen %
Kc= Kf= Km=

Static Density @ psi& % Bit= Lbs. Per Cu. Ft.

Recommended Bitumen Content Considering All Test Data %

Sieve	% Pass	Tol. % Total Wt. Pass	
1"	100		
¾"			
½"			
¼"			
No. 4			
8			
40			
200			

Table A-2. Core Data from Recycled Pavements, Part 1, Sentinel.[a/]

Test Site No. I-8-2(76) Mile Posts 93.7 & 94.4 Lane EB Travel

Construction Date Summer 1978 Layer 2nd Thickness 1 1/8"

Sample Date 5/30/84 Remarks: No bleeding, 1/4" rutting in OWP & IWP.

	Sieve No.	Mile Post 93.7 EB OWP	Mile Post 93.7 EB BWP	Mile Post 93.7 EB IWP	Mile Post 94.4 EB OWP	Mile Post 94.4 EB BWP	Mile Post 94.4 EB IWP	ADOT's Design 100/0
GRADATION, TOTAL PERCENT PASSING	1"	--	--	--	100	--	--	100
	3/4"	100	100	100	98	100	100	97
	1/2"	89	86	95	92	90	94	86
	3/8"	83	76	82	84	80	84	76
	#4	63	56	62	64	62	62	57
	#8	46	42	45	46	46	45	42
	#16	32	31	33	32	33	32	31
	#30	26	25	26	26	26	26	24
	#50	20	20	21	20	21	20	19
	#100	16	15	16	16	16	16	12
	#200	10.2	9.6	9.6	10.2	10.2	10.1	8
Marshall Stability, wet, lb		4,210	4,090	4,070	4,000	4,170	3,820	Hveem 49
Flow, .01 in.		11	12	10	11	14	15	--
Density, pcf		145.5	142.0	145.0	145.0	144.0	144.5	139.5
Eff., S.G.[b/]		2.531	2.536	--	2.328	--	--	--
VMA, %	Meas.	14.30	15.71	14.60	17.64	16.98	17.45	16.8
	Calc.	13.65	13.30	13.74	13.78	13.48	13.71	
Air Voids, %	Meas.	0	1.82	0	0.36	2.30	1.38	4.1
	Calc.	0	0	0	0	0	0	
Binder Content, % BTW		6.9	6.2	6.8	7.6	6.4	7.0	2.5[c/]
Recovered Viscosity @ 140°F, p.				1,750		1,317	828	Cyclogen M

a/ Average of at least two 4"D cores.

b/ From mixture with 5% asphalt having specific gravity of 1.018.

c/ Added bitumen.

Table A-2. Part 2, Sentinel.

Test Site No. I-8-2(76) Mile Posts 93.7 & 94.4 Lane EB Travel

Construction Date Summer 1978 Layer 3rd Thickness 3"

Sample Date 5/30/84 Remarks: No bleeding, 1/4" rutting in OWP & IWP.

	Sieve No.	Mile Post 93.7 EB OWP	Mile Post 93.7 EB BWP	Mile Post 93.7 EB IWP	Mile Post 94.4 EB OWP	Mile Post 94.4 EB BWP	Mile Post 94.4 EB IWP	ADOT's Design 100/0
GRADATION, TOTAL PERCENT PASSING	1"	100	100	100	100		--	100
	3/4"	98	98	98	98		100	97
	1/2"	90	94	90	88		94	86
	3/8"	89	87	86	78		85	76
	#4	80	87	86	78		85	76
	#8	43	45	46	42		44	42
	#16	32	32	34	32		32	31
	#30	26	26	28	25		26	24
	#50	20	20	22	20		20	19
	#100	14	14	16	14		14	12
	#200	9.0	9.0	9.8	8.8		9.3	8
Marshall Stability, wet, lb		3,710	3,010	3,380	3,370	2,830	3,040	Hveem 49
Flow, .01 in.		13	16	15	11	17	14	--
Density, pcf		144.0	143.0	144.0	142.0	137.5	142.0	139.5
Eff., S.G.		2.531	2.518	--	2.536	--	--	--
VMA, %	Meas.	15.00	15.82	15.24	16.39		16.26	16.8
	Calc.	13.28	13.85	14.01	13.33		13.50	
Air Voids, %	Meas.	0.02	0	0	1.05		1.35	4.1
	Calc.	0	0	0	0		0	
Binder Content, % BTW		6.8	7.2	7.1	6.8	--	6.6	2.5
Recovered Viscosity @ 140°F, p.				12,980				Cyclogen M

Table A-2. Part 3, Rillito.

Test Site No. I-10-4(68) Mile Post 247 Lane WB Travel

Construction Date Spring 1980 Layer 3rd Thickness 3"

Sample Date 12/14/83 Remarks: M.P. 247.7--some bleeding, 3/8" rutting in travel lane. M.P. 247.6--no bleeding, 1/4" rutting in travel lane.

	Sieve No.	Mile Post 247.7 WB OWP	BWP	IWP	Mile Post 247.6 WB OWP	BWP	IWP	ADOT's Design 70/30
GRADATION, TOTAL PERCENT PASSING	1"	100	100	100	100	100		100
	3/4"	99	96	93	97	97		94
	1/2"	84	80	79	78	79		75
	3/8"	75	69	71	70	69		68
	#4	56	51	53	50	49		54
	#8	40	37	39	37	36		43
	#16	28	27	27	26	25		31
	#30	18	18	18	17	17		22
	#50	12	11	12	11	11		13
	#100	9	9	8	8	7		8
	#200	5.8	5.5	5.7	5.8	5.5		5.3
Marshall Stability, wet, lb		2,260	1,960	1,550	1,810	1,770		Hveem +35
Flow, .01 in.		6	11	9	12	13		--
Density, pcf		147.5	145.0	145.5	145.5	147.5		145.2
Eff., S.G.		2.596	2.596	2.596	2.598	2.598		--
VMA, %	Meas.	13.8	14.9	14.9	13.9	13.7		13.2
	Calc.	12.9	12.9	13.0	12.6	12.5		
Air Voids, %	Meas.	1.4	3.5	2.9	2.3	2.4		4.1
	Calc.	0.2	1.2	0.6	0.8	1.0		
Binder Content, % BTW		5.3	5.0	5.3	5.0	4.9		1.3
Recovered Viscosity @ 140°F, p.		--	1,040	--	--	865		Cyclogen M

Table A-2. Part 4, Rillito.

Test Site No. I-10-4(68) Mile Post 248.0 Lane EB Travel

Construction Date Spring 1980 Layer 3rd Thickness 2.5"

Remarks: No bleeding, 7/8" rutting in right & left wheelpaths.

	Sieve No.	Mile Post 248.0 EB OWP	BWP	IWP	Mile Post OWP	BWP	IWP	ADOT's Design 70/ 30
GRADATION, TOTAL PERCENT PASSING	1"	100	100	100				100
	3/4"	94	93	93				94
	1/2"	75	69	76				75
	3/8"	67	57	68				68
	#4	49	41	49				54
	#8	36	31	36				43
	#16	25	23	26				31
	#30	17	17	18				22
	#50	11	11	11				13
	#100	8	8	8				8
	#200	6.9	5.8	5.9				5.3
Marshall Stability, wet, lb		1,600	1,300	1,610				Hveem +35
Flow, .01 in.		12	13	15				--
Density, pcf		149.0	147.0	149.5				145.2
Eff., S.G.		2.591	2.597	--				--
VMA, %	Meas.	12.4	13.4	12.3				13.2
	Calc.	12.8	11.9	12.6				
Air Voids, %	Meas.	0.5	2.3	0.5				4.1
	Calc.	0.8	0.5	0.7				
Binder Content, % BTW		5.1	4.8	5.0				1.3
Recovered Viscosity @ 140°F, p.		380	--	--				Cyclogen M

Table A-2. Part 5, Dateland.

Test Site No. I-8-1(80) Mile Posts 64.0 & 59.0 Lane WB Travel

Construction Date Summer 1982 Layer 3rd Thickness M.P. 64.0--3.1"
M.P. 59.0--3.25"

Sample Date 1/6/84 Remarks: No bleeding or rutting.

	Sieve No.	Mile Post 64.0 WB OWP	Mile Post 64.0 WB BWP	Mile Post 64.0 WB IWP	Mile Post 59.0 WB OWP	Mile Post 59.0 WB BWP	Mile Post 59.0 WB IWP	ADOT's Design 50/50
GRADATION, TOTAL PERCENT PASSING	1"	100	100		100	100	100	100
	3/4"	98	98		96	97	99	99
	1/2"	81	79		79	81	83	78
	3/8"	73	71		71	71	71	69
	#4	59	59		53	53	52	53
	#8	47	47		43	41	41	42
	#16	32	33		33	31	31	33
	#30	25	25		25	23	23	24
	#50	13	13		13	13	13	12
	#100	7	7		8	7	8	6
	#200	5.1	5.1		5.3	4.5	5.6	4.4
Marshall Stability, wet, lb		2,470	2,730	2,330	3,150	3,200	2,930	2,624
Flow, .01 in.		13	18	19	24	16	15	13
Density, pcf		140.5	142.5	140.0	144.0	144.0	143.5	140.8
Eff., S.G.		--	2.601	2.588	--	2.594	2.593	--
VMA, %	Meas.	17.7	16.5	18.5	15.4	15.9	15.5	--
	Calc.	14.4	14.2	--	14.1	14.0	13.3	
Air Voids, %	Meas.	5.7	4.8	5.9	4.1	3.3	4.7	5.9
	Calc.	1.9	2.2	--	2.8	1.4	2.4	
Binder Content, % BTW		5.4	5.2	5.7	4.9	5.5	4.7	2.5
Recovered Viscosity @ 140°F, p.		25,990	--	--	44,620	--	--	AC-20

Table A-2. Part 6, Firebird Lake.

Test Site No. IR-10-3(142) Mile Post 163.5 Lane EB Travel

Construction Date Summer 1982 Layer 3rd Thickness 2.5"

Sample Date 1/11/84 Remarks: No bleeding, 1/2" rutting in OWP, 3/4" rutting in IWP.

	Sieve No.	Mile Post 163.5 EB OWP	BWP	IWP	Mile Post OWP	BWP	IWP	ADOT's Design 50/50
GRADATION, TOTAL PERCENT PASSING	1"	100	100	100				100
	3/4"	93	95	97				99
	1/2"	81	83	86				85
	3/8"	70	71	77				72
	#4	55	55	63				53
	#8	44	43	49				45
	#16	33	31	36				32
	#30	23	21	24				25
	#50	13	12	13				14
	#100	7	7	7				8
	#200	5.1	4.7	4.9				4.5
Marshall Stability, wet, lb		1,710	1,710	1,560				5,132
Flow, .01 in.		7	5	8				14
Density, pcf		148.5	148.5	148.0				144.6
Eff., S.G.		--	2.685	2.679				--
VMA, %	Meas.	15.5	15.7	16.3				--
	Calc.	13.9	13.8	14.8				
Air Voids, %	Meas.	4.2	3.7	3.5				4.1
	Calc.	2.3	1.8	1.7				
Binder Content, % BTW		4.9	5.1	5.5				2.5
Recovered Viscosity @ 140°F, p.		6,280	--	--				AC-20

Table A-2. Part 7, Gila Bend.

Test Site No. I-8-2(80) Mile Post 124.0 Lane WB & EB Travel

Construction Date Summer 1982 Layer 3rd Thickness 3"

Sample Date 1/5/84 Remarks: No bleeding or rutting.

	Sieve No.	Mile Post 124.0 EB OWP	Mile Post 124.0 EB BWP	Mile Post 124.0 EB IWP	Mile Post 124.0 WB OWP	Mile Post 124.0 WB BWP	Mile Post 124.0 WB IWP	ADOT's Design 50/50
GRADATION, TOTAL PERCENT PASSING	1"	100	100	100	100	100	100	100
	3/4"	97	99	99	99	99	99	99
	1/2"	77	81	81	89	88	89	85
	3/8"	67	69	73	79	77	76	74
	#4	50	51	55	59	61	58	58
	#8	41	41	43	47	48	45	44
	#16	33	33	36	35	36	34	33
	#30	25	25	25	26	27	25	23
	#50	13	14	13	17	17	16	13
	#100	8	8	8	11	10	10	7
	#200	4.7	4.7	4.1	6.5	6.3	5.9	4.1
Marshall Stability, wet, lb		3,370	3,150	3,000	3,860	3,510	3,250	4,331
Flow, .01 in.		11	13	13	19	17	11	13
Density, pcf		150.0	149.0	147.0	146.5	146.5	145.5	145.2
Eff., S.G.		2.613	--	2.635	--	2.669	2.657	--
VMA, %	Meas.	13.2	13.9	14.9	16.3	16.4	17.1	--
	Calc.	14.1	14.1	14.9	13.8	14.1	13.7	
Air Voids, %	Meas.	1.8	1.9	3.7	4.7	4.9	5.3	5.1
	Calc.	2.8	2.2	3.7	1.6	2.2	1.4	
Binder Content, % BTW		4.8	5.1	4.9	5.1	5.0	5.1	2.5
Recovered Viscosity @ 140°F, p.		--	10,800	--	25,480	--	--	AC-20

Table A-2. Part 8, Williams Field.

Test Site No. IR-10-3(148) Mile Post 162.9 Lane WB Travel & Passing

Construction Date Summer 1983 Layer 3rd Thickness 3.4"

Sample Date 12/27/83 Remarks: Travel & passing lanes--no bleeding or rutting.

	Sieve No.	Mile Post 162.9 Travel OWP	BWP	IWP	Mile Post 162.9 Passing[a/] OWP	BWP	IWP	ADOT's Design 50/50
	1"	100	100	100	--	100		100
	3/4"	98	94	97	100	99		95
	1/2"	84	81	84	93	87		79
	3/8"	74	70	74	84	79		72
GRADATION, TOTAL PERCENT PASSING	#4	54	52	54	66	60		53
	#8	44	44	44	53	49		46
	#16	35	34	35	41	37		37
	#30	26	26	26	29	27		26
	#50	15	14	15	15	14		15
	#100	8	8	8	8	7		9
	#200	5.1	4.8	5.8	5.4	4.1		6.7
Marshall Stability, wet, lb		3,220	3,080	2,750	3,470	3,710		4,750
Flow, .01 in.		19	13	15	26	15		--
Density, pcf		145.5	139.5	144.0	138.0	138.0		143.0
Eff., S.G.		2.596	2.599	--	2.578	2.593		--
VMA, %	Meas.	14.4	14.6	15.1	20.5	19.1		--
	Calc.	14.4	14.6	14.1	15.5	15.8		
Air Voids, %	Meas.	3.3	3.8	4.8	8.2	7.1		5.2
	Calc.	3.3	4.0	3.5	2.6	3.2		
Binder Content, % BTW		4.8	4.6	4.6	5.7	5.5		2.0
Recovered Viscosity @ 150°F, p.		--	--	23,140	348,590	264,210		AC-10

[a/] Old paving material, not a recycled mixture.

Table A-2. Part 9, Red Rock.

Test Site No. IR-10-4(86) Mile Post 229.0 Lane WB Travel

Construction Date Spring 1985 Layer 2nd / 3rd Thickness 2.5" / 3.0"

Sample Date 3/18/86 Remarks: No bleeding or rutting.

	Sieve No.	2nd Layer OWP	2nd Layer BWP	2nd Layer IWP	3rd Layer OWP	3rd Layer BWP	3rd Layer IWP	ADOT's Design 50/50
GRADATION, TOTAL PERCENT PASSING	1"	100	100	100	100		100	100
	3/4"	99	97	99	97		99	98
	1/2"	87	87	89	81		87	81
	3/8"	78	80	82	71		78	73
	#4	53	50	59	47		55	57
	#8	39	43	44	35		41	43
	#16	29	31	32	25		30	32
	#30	21	21	22	17		21	21
	#50	13	14	14	10		13	13
	#100	13	11	10	5		8	9
	#200	6.3	6.4	6.5	4.0		5.0	7.0
Marshall Stability, wet, lb		3,860	2,690	3,010	3,400	5,150	4,470	4,790
Flow, .01 in.		13	9	9	13	13	17	12
Density, pcf		143.5	141.5	140.5	142.5	145.5	144.5	143.4
Eff., S.G.		2.596	2.608	2.585	2.579	--	2.584	--
VMA, %	Meas.	15.2	16.9	17.1	15.1	--	14.4	--
	Calc.	15.2	14.2	13.5	13.7	--	13.7	
Air Voids, %	Meas.	5.7	7.1	6.5	5.9	--	4.1	5.7
	Calc.	5.7	4.4	2.9	4.5	--	3.5	
Binder Content, % BTW		4.2	4.4	4.8	4.1	--	4.5	1.5
Recovered Viscosity @ 140°F, p.		--	--	--	--	--	--	AC-40

Table A-3 - Core Data from the Willcox Bypass

Upper Three Inches

Location, M.P.	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200	S. G. Eff.	Asphalt Content, %	Asp. Vis. p.,140°F
		Aggregate Gradation, Total % Passing											
339	99	86	81	61	43	30	22	15	11	7.5	2.648	5.4	8,929
340	100	95	89	65	46	33	24	17	12	8.4	2.653	5.8	6,186
EB 341	99	90	86	68	48	34	25	17	12	8.6	---	5.7	11,679
342	99	90	84	65	46	33	24	17	12	9.3	2.661	5.9	6.254
343	98	85	80	56	38	28	21	15	11	7.5	---	5.3	17,373
344	100	94	89	66	44	31	22	15	10	7.1	2,657	5.7	8,806
344	100	93	86	63	43	32	24	17	12	8.2	2.667	5.8	6,200
343	100	94	85	63	44	33	25	18	12	8.3	---	5.8	6,554
WB 342	100	92	84	61	41	30	23	16	11	8.1	---	5.7	11,386
341	100	92	87	68	49	35	25	18	12	8.7	2.652	5.7	10,147
340	100	91	85	59	37	26	20	15	10	7.3	---	5.7	13,981
339	98	90	85	66	46	32	23	16	11	8.6	---	6.0	3,435
Average	99	91	85	63	44	31	23	16	11	8.1	2.656	5.7	9,244
Std Deviation	1	3	3	4	4	3	2	1	1	0.7	0.007	0.2	3,915

Upper One-Half Inch - A.C.F.C.

Location, M.P.	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200	S. G. Eff.	Asphalt Content, %	Asp. Vis. p.,140°F
		Aggregate Gradation, Total % Passing											
EB 337	--	--	100	87	64	44	29	18	11	8.4	---	7.0	7,293
338	--	100	99	89	68	47	30	19	12	8.7	---	5.7	7,293
WB 338	--	100	99	85	63	45	29	18	12	8.8	---	5.7	5,384
337	--	--	100	55	26	15	11	9	7	5.5	---	8.8	5,384
Average	--	100	100	79	55	38	25	16	11	7.9	---	6.8	6,339
Std Deviation	--	0	1	16	20	15	9	5	2	1.6	---	1.5	1,102

Second Lift - 2-1/2 Inches of A.C.

Location, M.P.	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200	S. G. Eff.	Asphalt Content, %	Asp. Vis. p.,140°F
		Aggregate Gradation, Total % Passing											
EB 337	98	87	77	54	39	29	22	15	10	6.7	---	5.0	3,393
338	98	83	73	53	36	27	20	16	12	8.4	---	5.0	9,509
WB 338	100	88	82	62	44	32	25	18	13	9.5	---	5.6	8,719
337	100	84	71	56	43	34	26	20	14	10.6	2.652	4.9	5,588
Average	99	86	76	56	41	31	23	17	12	8.8	2.652	5.1	6,802
Std Deviation	1	2	5	4	4	3	3	2	2	1.7	0	0.3	2,834

APPENDIX B

LIST OF FIGURES

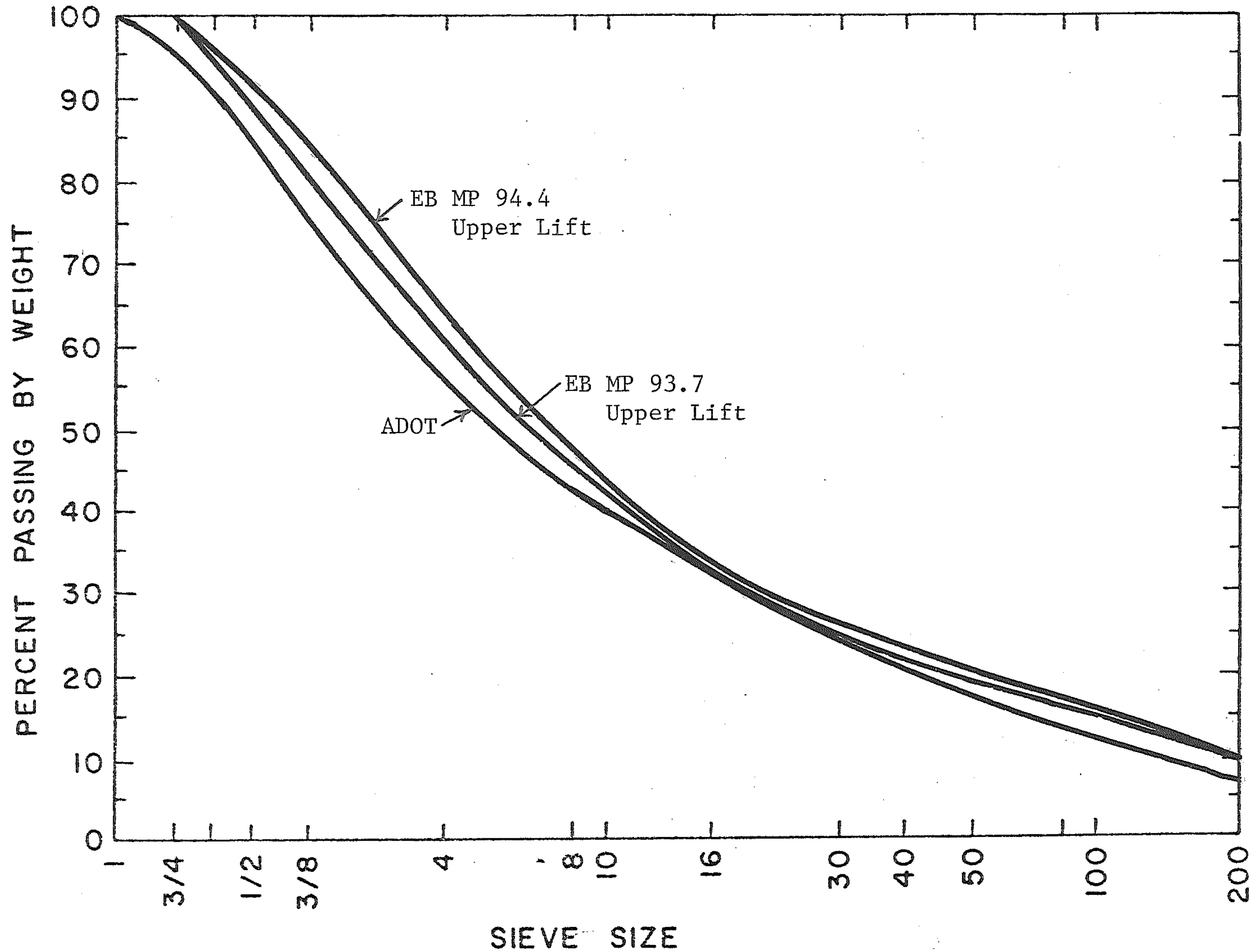

Figure B-1. Design and Extracted Gradations from Recycled Pavements Sentinel, Upper Lift

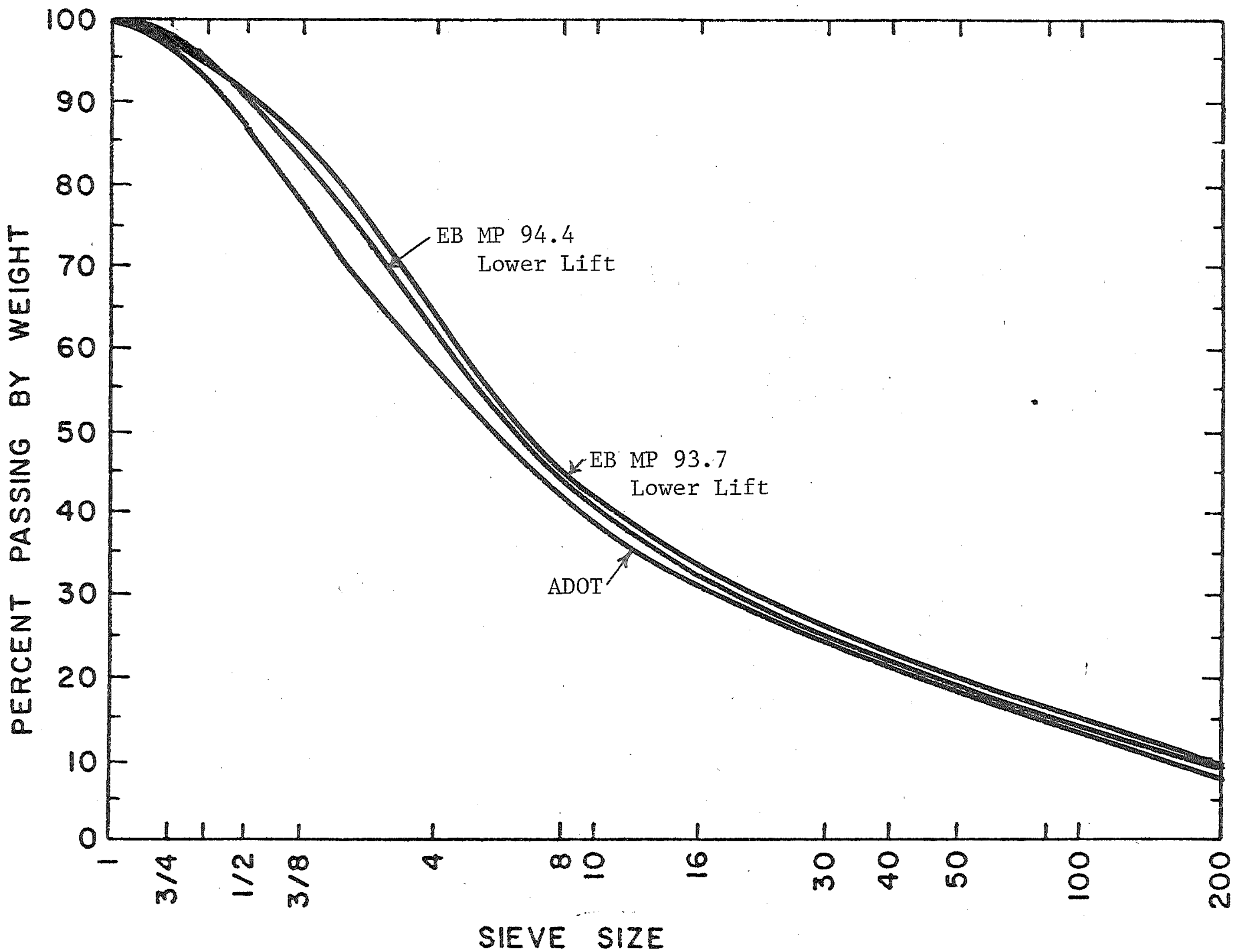

Figure B-2. Design and Extracted Gradations from Recycled Pavements Sentinel, Lower Lift

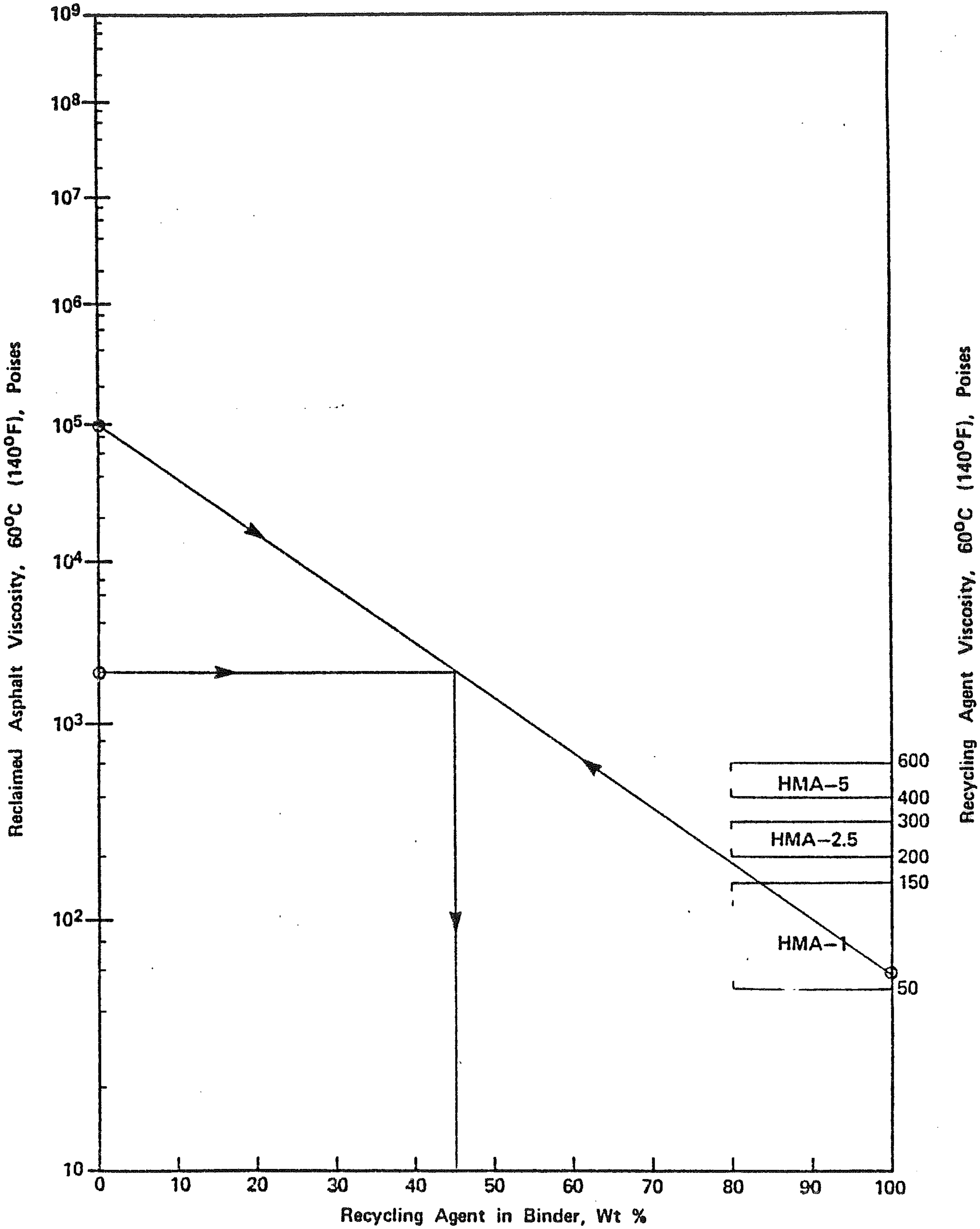

Figure B-3. Viscosity Blending Chart (Reference 19).

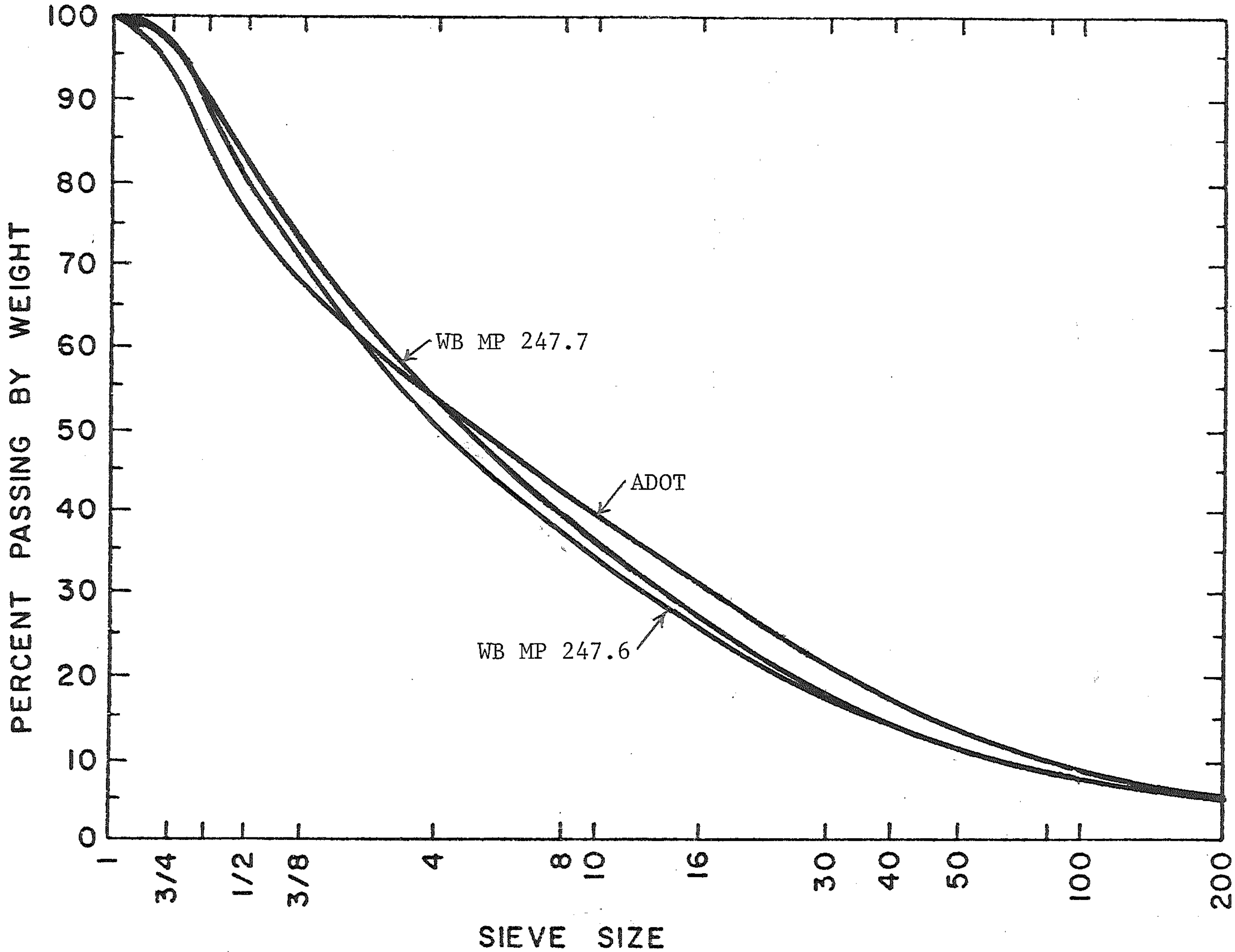

Figure B-4. Design and Extracted Gradation from Recycled Pavement Rillito W.B.

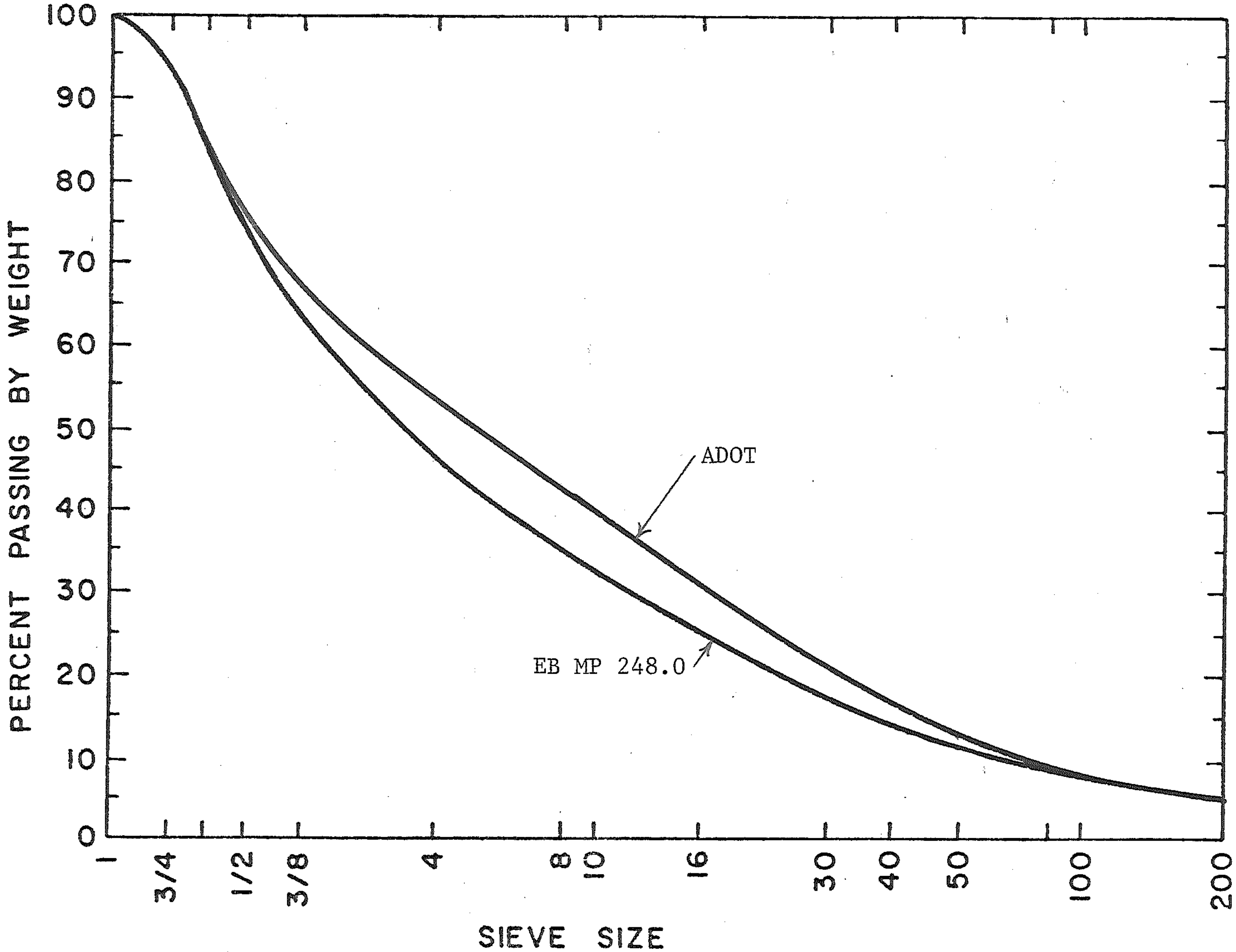

Figure B-5. Design and Extracted Gradation from Recycled Pavement Rillito E.B.

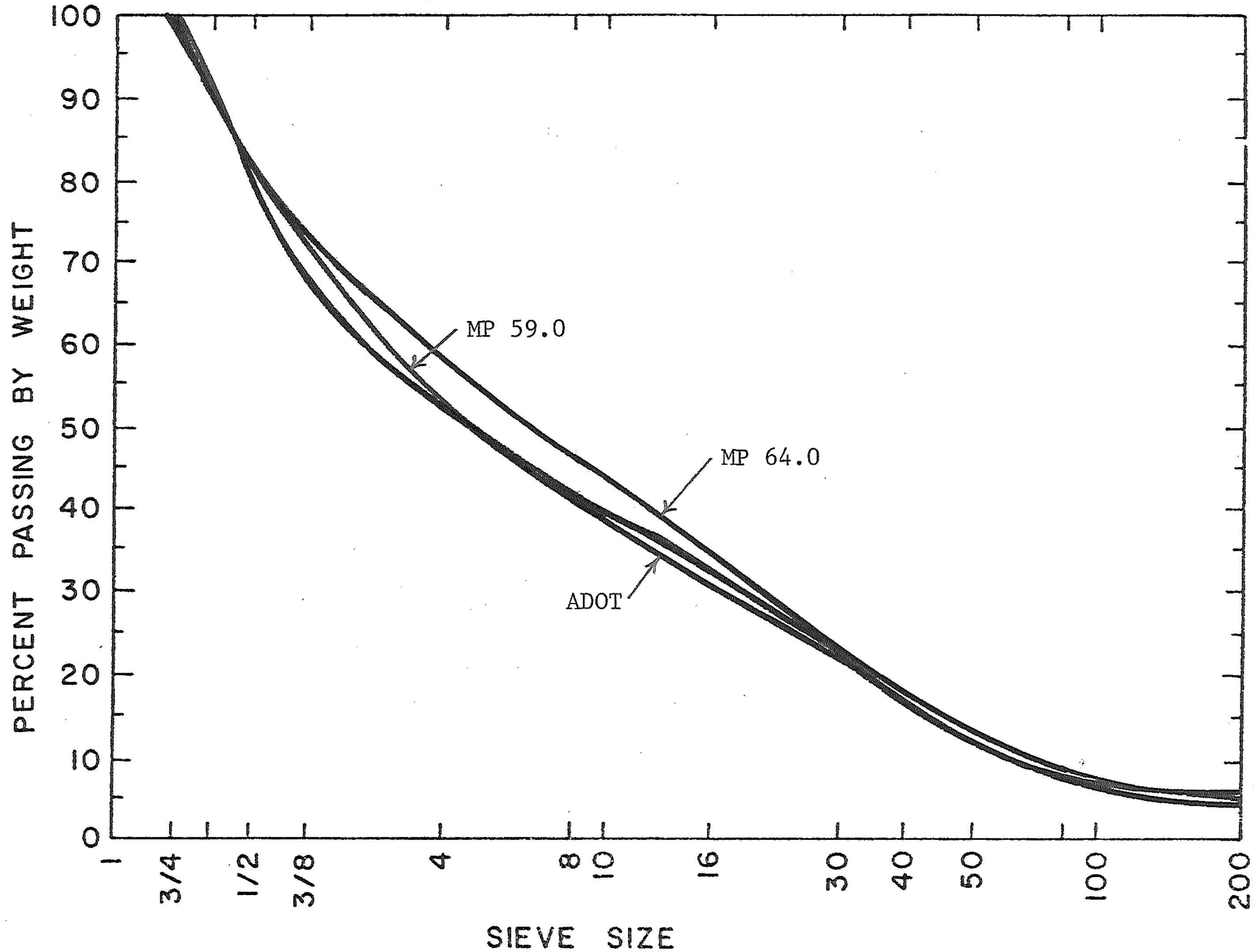

Figure B-6. Design and Extracted Gradations from Recycled Pavement Dateland

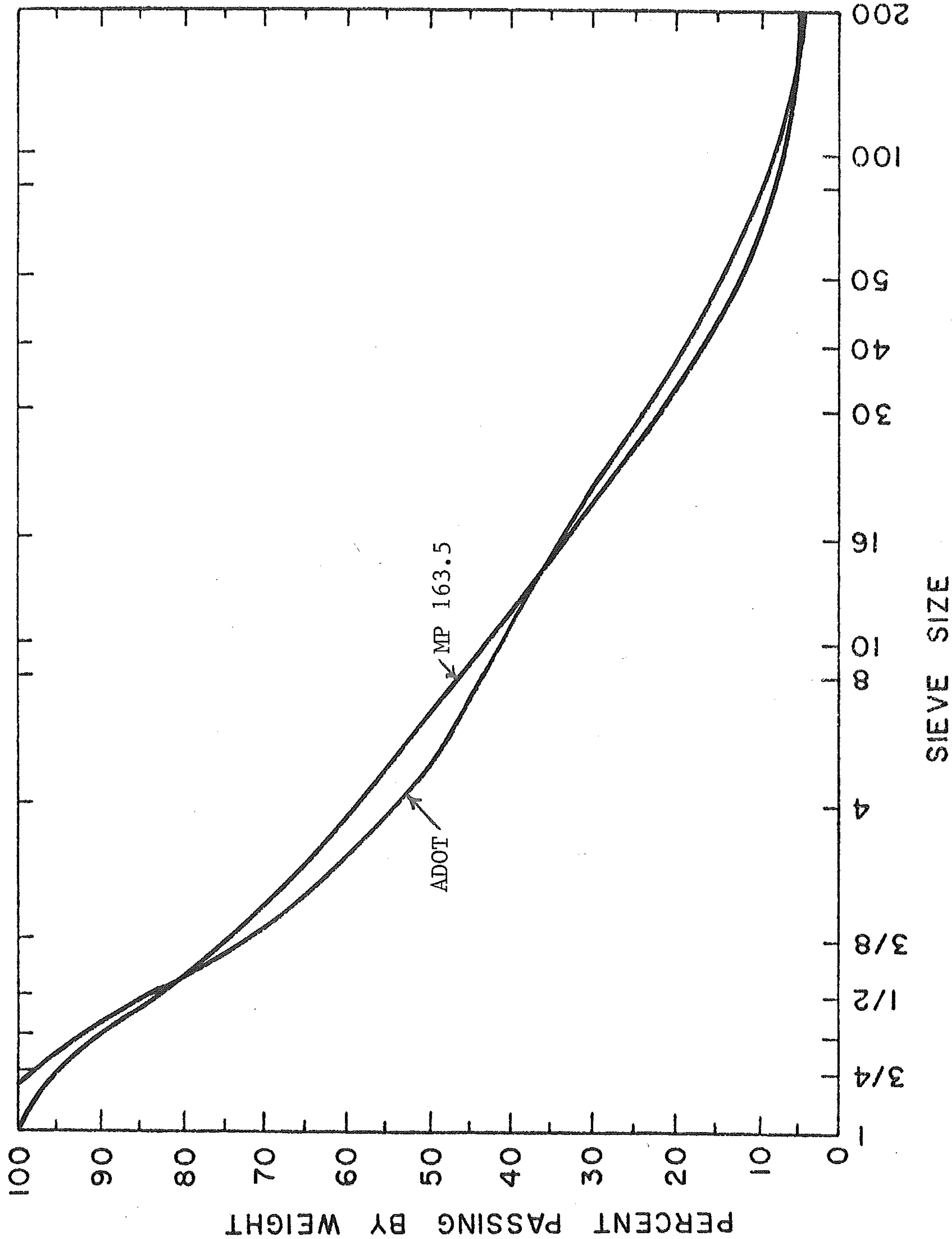

Figure B-7. Design and Extracted Gradations from Recycled Pavement Firebird Lake

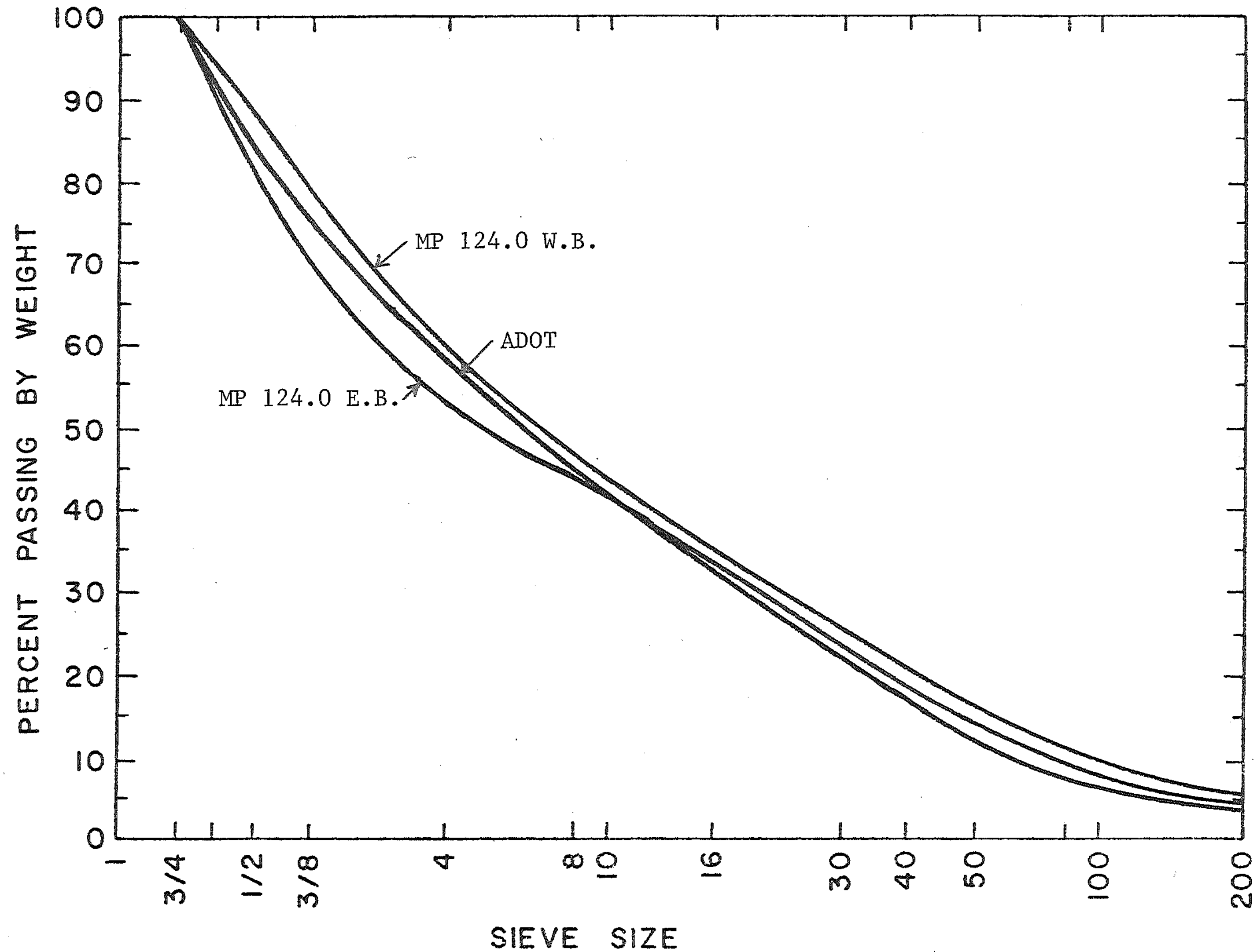

Figure B-8. Design and Extracted Gradations from Recycled Pavement Gila Bend

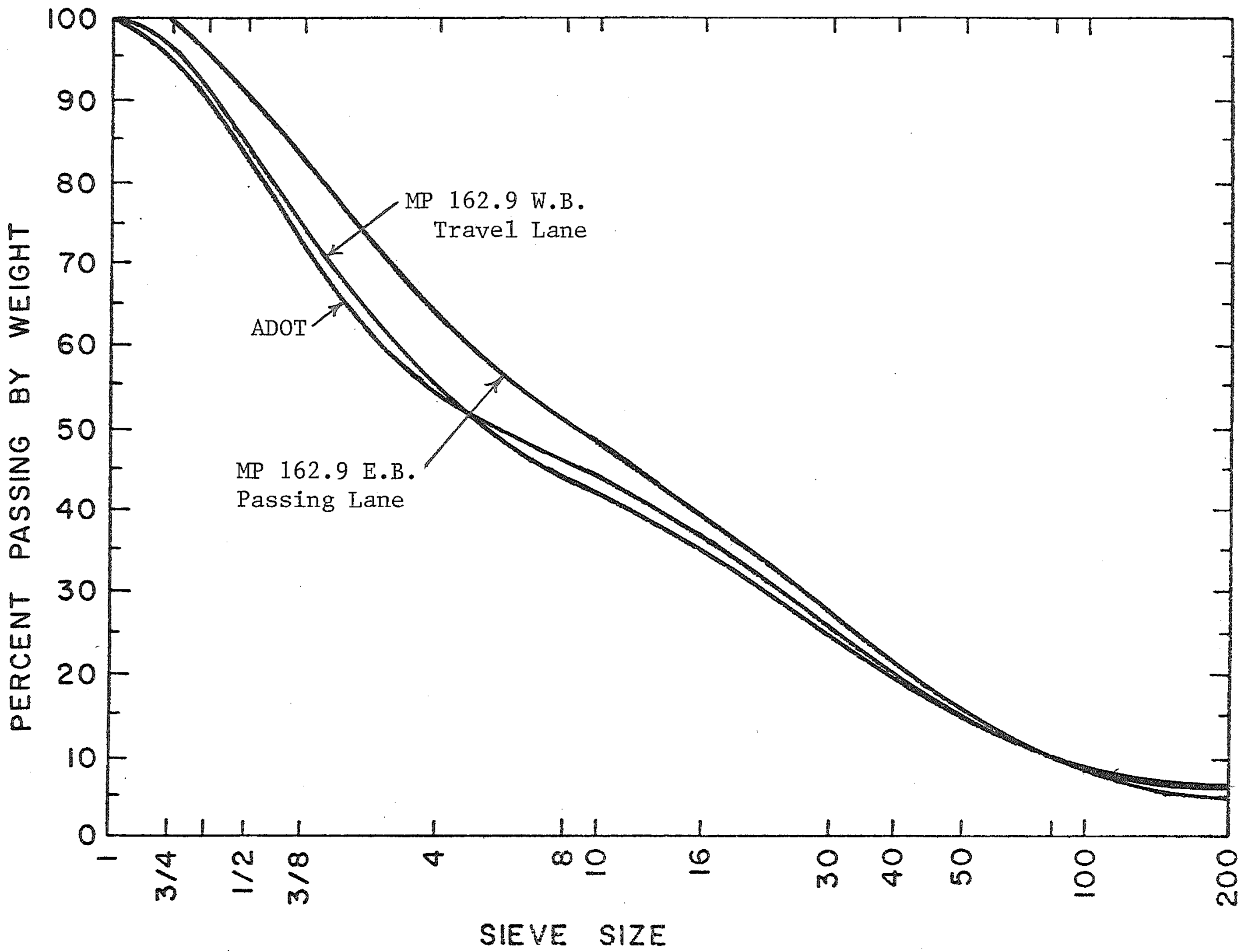

Figure B-9. Design and Extracted Gradations from Recycled Pavement Williams Field

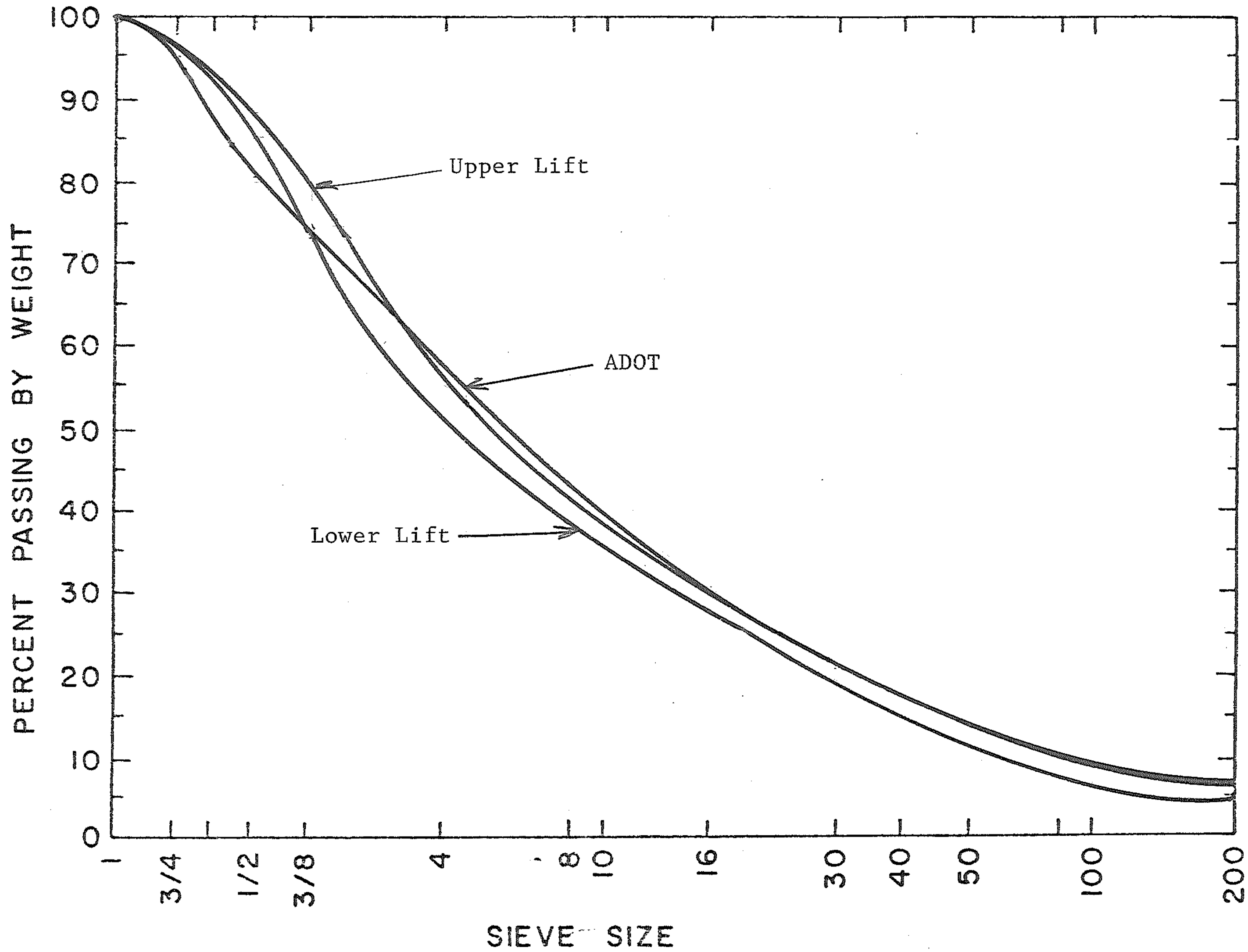

Figure B-10. Design and Extracted Gradations from Recycled Pavement
Red Rock

APPENDIX C

PROCEDURE FOR ASPHALT RECOVERY FROM EXTRACTION SOLUTION

1. SCOPE

1.1 This procedure involves the recovery of asphaltic cement from an extraction solution previously obtained by the Quantitative Extraction of Bitumen from Extraction of Bitumen from Bituminous Paving Mixtures AASHTO Designation: T164-80. The asphalt is recovered with properties substantially the same as those it possessed in the asphaltic mixture. Photographs of the equipment possessed in the asphaltic mixture. Photographs of the equipment are shown in Figure C-1.

2. APPARATUS

2.1 Vacuum pump capable of maintaining a vacuum of 15 to 20 inches of mercury.

2.2 VE-50 "Rotavapor" equipped with water jacketed condenser, see Figure C-2.

2.3 Electric heating mantle to fit a 1,000 ml retort.

2.4 Vacuum gauge with adjustable valve for regulation of vacuum.

2.5 Exhaust hood for removal of methylene chloride vapor.

2.6 Source of cold water for the condenser.

3. PROCEDURE

3.1 The entire procedure, from the start of extraction to the final recovery, must be completed within eight hours.

3.2 Pour the extraction solution (not more than 500 cc at a time) into the 1,000 mL round bottom flask.

3.3 Place the flask on the rotary vacuum evaporator (VE-50 "Rotavapor"). The ground glass connection shall have a light application of high vacuum grease to insure that there is no loss of vacuum at this tapered joint. Secure joint by attaching spring clip.

3.4 Turn on water faucet so that water flows continually through the condenser.

3.5 Adjust VE-50 "Rotavapor" to a speed setting such that, while the vacuum is 15-20 inches Hg, the retort rotates approximately 50-60 RPM.

3.6 Position the heating mantle + 1/8 inch under the rotating retort.

3.7 Energize the heating mantle and the vacuum pump simultaneously.

3.8 Adjust vacuum to 15 inches Hg immediately. During the heating, the vacuum will drop then rise, following evaporation of methylene chloride. Stop the recovery 10 minutes after the vacuum gauge again indicates 15 inches Hg.

3.9 Record time and corresponding vacuum.

3.10 Allow the retort to cool to about 140°F in an upright position when the viscosity is to be determined soon after recovery. Pour the recovered asphalt from the retort directly into the proper Cannon-Manning Vacuum Viscometer to 2 mm of its fill line.

(a) Extraction of Asphalt

(b) Recovery of Asphalt

Figure C-1. Set-Ups for Extraction (a) and Recovery (b) of Asphalt.

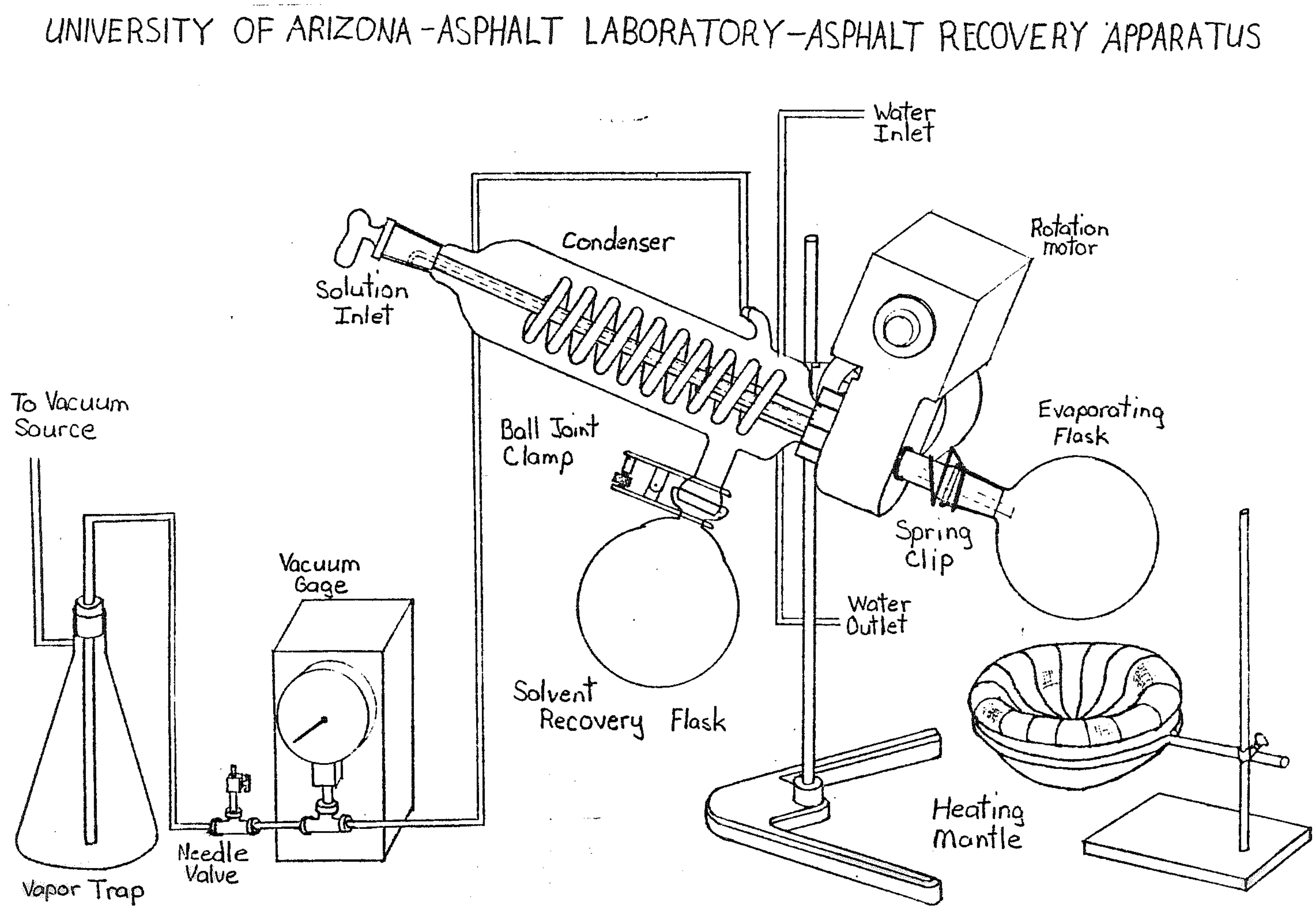

Figure C-2. Set-Up for Asphalt Recovery.