OREGON ASPHALT-CONCRETE B-MIX IMPROVEMENT STUDY

Final Report

HP&R PROJECT NO. 5276

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Prepared for

Federal Highway Administration
Washington, D.C. 20590

June 1991

Technical Report Documentation Page

1. Report No. FHWA-OR-MA-92-10	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Oregon Asphalt-Concrete B-Mix Improvement Study		5. Report Date June, 1991
		6. Performing Organization Code
7. Author(s) Anthony J. George, P.E., Richard L. Dominick, Glenn Boyle		8. Performing Organization Report No.
9. Performing Organization Name and Address Oregon State Highway Division Materials & Research Section 800 Airport Road, SE Salem, OR 97310		10. Work Unit No. (TRAIS)
		11. Contract or Grant No. HPR #5276
12. Sponsoring Agency Name and Address Oregon State Highway Division Materials & Research Section 800 Airport Road, SE Salem, OR 97310		13. Type of Report and Period Covered Final Report April 1988 - June 1991
		14. Sponsoring Agency Code
15. Supplementary Notes		

16. Abstract

The Oregon State Highway Division (OSHD) has experienced rutting and/or ravelling pavements in Oregon Class "B" asphalt concrete in the last two decades. Some of these pavement problems have evolved from material changes or changes in construction practices.

The typical agency reaction to these problems has been to make adjustments in paving mixture components and mixture characteristics. These changes to mixtures have sometimes created unexpected pavement problems.

Test mixtures composed of five different aggregate gradations and up to seven asphalt cement types were fabricated in the laboratory. Several index and performance tests were performed on each mixture. These test results were compared to current OSHD paving mixture design criteria.

This study concluded that a gradation slightly coarser than the maximum density gradation in the 1 - 1/4 inch fraction, and significantly coarser than the maximum density in the 1/4 - 0 inch fraction should improve mixture performance. This study also concluded that conventional asphalt is satisfactory unless environment or construction conditions dictate a need for a modified asphalt.

Oregon has implemented the recommended gradations from this study. Oregon has also discontinued the use of component based modified asphalt specifications for most paving projects. Only projects with harsh environmental conditions employ modified paving asphalt, and they are under a performance based specification.

17. Key Words PAVEMENTASPHALT, CEMENT, GRADATION, VOIDS, STRIPPING, RUTTING, STABILITY, MIX DESIGN, SPECIFIC GRAVITY		18. Distribution Statement	
19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages	22. Price

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

ACKNOWLEDGEMENTS

The authors thank JoAnne Robison, ODOT Research Unit, for typing this report.

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

1.0 INTRODUCTION

Since the early 1970's, the Oregon State Highway Division (OSHD) has experienced problems with pavement performance. At that time, AR viscosity graded asphalt-cement, the vibratory roller, and the drum dryer paving plant were introduced. Heavier loads and higher tire pressures were being experienced on the highways. Rutting problems were observed on some of Oregon's heavily traveled primary highways during the mid 1970's.

The change in the asphalt-cement grading system had an effect on pavement performance. The method of identifying the grade was changed from penetration at 77°F (25°C) on the original asphalt, to the viscosity of the aged residue after it had been aged in a rolling thin film oven. The petroleum industry was struggling to provide a consistent paving asphalt. That became difficult at times since the crude oil being supplied to them was constantly changing due to the Arab embargo of petroleum in 1973.

This crude oil supply problem forced asphalt manufacturers to blend materials from different crude fields that were not always consistent. These new products usually met OSHD consistency specifications, but did not handle in the same way as the old penetration grade asphalt. Selection of the proper grade of asphalt for each situation required some retraining and experience. Some necessary performance requirements of the asphalt also were not measured in the AR grading system.

The introduction of the vibratory roller and the drum dryer asphalt-concrete plant had the most dramatic effect on pavement performance.

The use of the vibratory roller required additional knowledge of the mixture, such as when to operate in the vibratory mode and when to operate it in the static mode. Vibratory rolling also required knowledge of the relationship between vibration frequency, amplitude and mat thickness for proper compaction.

The vibratory roller created another problem by sometimes fracturing the large aggregate. This was particularly troublesome where the material was segregated. In these segregated areas, large stones adjacent to each other would not be cushioned by finer material.

The drum dryer plants introduced other problems. For example, the manufacturer of drum dryer plants insisted that moisture be retained in the mixture to provide a "turbulent action" for coating purposes. The high mixture moisture content and the windrow of the mix combined the heat and moisture in a way that would strip the asphalt from the aggregate.

This "strip ravelling" appearing in the pavement panel as a result of moisture was aggravated by the coarse gradation of the AC mixture in segregated areas. Where there were not enough fines surrounding the larger particles, the moisture would peel or strip the asphalt from the rock particle. These stripped areas would disintegrate rapidly and areas of pavement would come out of the mat or "ravel".

Another problem created by the high moisture in mixes was "flushed" or "fat" pavements. These were areas of predominantly asphalt and very fine aggregate that migrated to the surface, leaving smooth, soft areas. These areas rutted and deformed easily. Flushed pavements were caused when excess moisture filled the mixture voids and didn't leave enough room for the asphalt. As Oregon moved to coarser mixes, the number of flushed pavements decreased, yet the number of ravelled pavements continued to increase due to stripping. Moisture problems were significantly reduced by changes in the specifications at about this same time.

After several years of strip ravelling problems, the mix designs were changed back to finer gradations and less voids to overcome the ravelling and assist the contractors with the compaction and segregation problems. Now, because of finer gradations and lower void contents, rutting and flushing have again become problems.

1.1 PURPOSE OF THE STUDY

The OSHD is currently experiencing some rutting and flushing pavement problems. This study was undertaken to improve the OSHD's Class "B" asphalt concrete mix designs so that the Class "B" pavements will have good resistance to rutting while still resisting stripping and ravelling.

In Oregon, Class "B" asphalt concrete is the "workhorse" class of pavement. It is a coarsely graded mix with 3/4-inch top size crushed aggregate. It is used for all base layers under open graded friction courses. It is used for many other base layer applications, and is also used for many wearing courses in the State.

1.2 OBJECTIVES OF THE STUDY

1. Determine what effect gradation changes have on mix properties based on indicator test results.

2. Determine what effect asphalt modification has on mix properties based on indicator test results.

3. Determine the best gradation for Class "B" asphalt pavement based on mix property test results.

4. Determine if there is a significant relationship between mix property test results (see the list of tests in next section).

5. Develop principles for recommending modified asphalt use, based on minimum performance needs and cost effectiveness.

2.0 BACKGROUND

The study involved testing and evaluating results on OSHD Class "B" asphalt concrete. The study design focused on two types of potential mix design improvements. These were the effect of aggregate gradation and type of asphalt binder on mix properties. Several gradations were evaluated in four phases of testing. During each testing phase, several types of asphalts were used: rubber modified, polymer modified, and conventional AC graded asphalt. Asphalts used in Phase I were:

Chevron AC 20 -	A conventional viscosity graded asphalt.
Chevron AR 4000 -	A conventional aged viscosity graded asphalt.
Chevron AR 4000W -	Washington State viscosity graded asphalt.
Asphalt Supply 20R -	SBR modified, styrene-butadiene rubber.
Chevron CA(P)-1 -	EVA modified, ethylene-vinyl-acetate.
ELF PAC 20 -	SBS modified, styrene-butadiene-styrene block copolymer.
Husky 85/100 DN -	Dupont Neoprene modified.

The AC mixture tests performed and evaluated on each gradation (with several asphalt types) were:

1. Bulk Specific Gravity, AASHTO T-166 @ 1st and 2nd compaction*
2. Maximum Specific Gravity, AASHTO T-209
3. Percent Air Voids, OSHD TM-310 @ 1st and 2nd compaction*
4. Hveem stability, AASHTO T-246 @ 1st and 2nd compaction*
5. Index of Retained Strength, AASHTO T-165
6. The IRM_R -- Index of Retained Resilient Modulus, OSHD TM 315-89

Preliminary work done before testing began included an historic review of OSHD mix designs. The mix design criteria were reviewed from historic files dating back to the early 1960's. A literature review also pointed out that asphalts modified with additives, such as polymers,

* OSHD mix design procedure calls for two compactions on each sample. The first compaction is done according to AASHTO T-247. After first specific gravity and first stability, the sample is reheated to 250°F, inverted, and recompacted. The samples are then retested for gravity and stability.

latexes, and rubber are advertised as a means to control rutting, cracking, oxidation, moisture damage, and for rejuvenating aged asphalt.

After the preliminary investigation was completed the testing was set up in four phases.

3.0 TESTING

PHASE I of the testing program included fabricating and testing sets of samples with gradations at the fine and coarse extremes of the OSHD Class "B" broad band range, and also a set of samples directly on the 0.45 power maximum density gradation (Figure 1). A set of samples consisted of five specimens each fabricated at increasing asphalt contents in 0.5% increments (Table 3.1). The "optimum" asphalt content was intended to be near the center of each set.

TABLE 3.1: Asphalt Content

	FINE	MAXIMUM DENSITY	COARSE
Chevron AC 20	5.0 - 7.0	4.5 - 6.5	5.0 - 7.0
Chevron AR 4000	4.5 - 6.5	3.5 - 5.5	5.0 - 7.0
Chevron AR 4000W	4.5 - 6.5	3.5 - 5.5	4.5 - 6.5
Asphalt Supply & Service AC-20R	4.0 - 6.0	3.5 - 5.5	4.5 - 6.5
Chevron CA(P)-1	4.5 - 6.5	3.5 - 5.5	4.5 - 6.5
ELF PAC 20	4.5 - 6.5	3.5 - 5.5	4.5 - 6.5
Husky 85/100 DN	4.5 - 6.5	3.5 - 5.5	4.5 - 6.5

The purpose of this phase was to determine the effect on mix properties of large changes in gradation and to determine if asphalt modification changed mix properties at any place in the gradation range.

All three gradations in PHASE I were tested with the following paving grade asphalt cements: Chevron AC 20, Chevron AR 4000, Chevron AR 4000W, Chevron CA(P)-1 (EVA modified, Ethylene Vinyl Acetate), ELF PAC-20 (SBS modified styrene-butadiene-styrene block copolymer rubber), Asphalt Supply & Service AC-20R (SBR modified, styrene-butadiene rubber in latex form), and Husky 85/100 DN (DuPont Neoprene).

PHASE II testing consisted of a combined aggregate blend using the center of the separated size specification range for 3/4" - 1/4" and 1/4" - 0 aggregate. A target value of 58% passing the 1/4" sieve was the control sieve to determine separated size percentages (Table 3.2). The asphalt cement binders tested in this phase were: Chevron AC-20, ELF PAC-20, and Asphalt Supply & Service AC-20R. A set of five specimens, as described in PHASE I were fabricated and tested with each asphalt.

TABLE 3.2: Phase II Gradation

Sieve Size	Percent Passing
1"	100
3/4"	96
1/2"	83
3/8"	73
1/4"	58
#4	50
#10	31
#40	13
#200	4.4

The purpose of PHASE II was to compare mix properties at an "optimum" gradation (under the current OSHD approach) to the mix properties at gradation extremes. We theorized that mix properties at the extreme and densest gradations would be unsatisfactory based on design criteria. The number of asphalts used was reduced to three to limit the number of variables to examine. Each "type" of asphalt is represented (conventional, rubber modified, and polymer modified).

PHASE III was very similar to PHASE II, except that the chosen gradation included a higher percentage of aggregate retained on the 3/4", 1/2", and 3/8" sieves than the PHASE II samples. This was to determine if an increased coarse aggregate interlock would have an effect on mixture stability and stripping resistance due to the reduction in surface area. A target value of 58% passing the 1/4" sieve was retained from PHASE II (Table 3.3). The same asphalt cement binders used in PHASE II testing were used in PHASE III i.e., Chevron AC-20, ELF PAC-20, and Asphalt Supply & Service AC-20R.

TABLE 3.3: Phase II and Phase III Gradations

Sieve Size	PHASE II Gradation (Percent Passing)	PHASE III Gradation (Percent Passing)
1"	100	100
3/4"	96	94
1/2"	83	81
3/8"	73	70
1/4"	58	58
#4	50	50
#10	31	31
#40	13	13
#200	4.4	4.4

PHASE IV of this study investigated a change to a typical Class "B" asphalt concrete gradation. The variable is the percent passing the 1/4" sieve. The Class "B" aggregate passing 1/4" sieve was reduced from 62% to 59%, with the gradation at other sieves changing proportionally (Table 3.4). The Class "B" samples used in PHASE IV were fabricated with McCall AC-20 asphalt. The purpose for PHASE IV was to see what effect a small mid size gradation change would have on mix properties.

TABLE 3.4: Phase IV Gradations

Sieve Size	Percent Passing Gradation 1	Percent Passing Gradation 2
1"	100	100
3/4"	99	99
1/2"	84	83
3/8"	74	72
1/4"	62	59
#4	50	48
#10	29	30
#40	12	13
#200	4.1	4.3

The aggregate source used in PHASE I of the study was a crushed gravel that had angular and harsh particles, 100% fracture and no natural sand (Ontario Asphalt, OSHD No. 23-47-5). The PHASE II and III aggregate was also crushed gravel with about 15% natural sand included (Santosh Pit No. 5-4-1). PHASE IV employed mix designs from an actual construction project. The aggregate is also a crushed gravel with the same characteristics as the PHASE II aggregate, but with no natural sand (Scappoose Sand & Gravel No. 5-1-1).

TABLE 3.5: Design Criteria For Standard Duty Pavement

Criteria		Required Value
Asphalt Film Thickness		Sufficient - Sufficient/Thick
Air Voids	- 1st compaction	5% - 6% (1% lower for base)
	- 2nd compaction	2% minimum
Stability	- 1st compaction	35 minimum
	- 2nd compaction	35 minimum
IRS @ Design Asphalt %		75 minimum
IRMR @ Design Asphalt %		70 minimum
P#200/% Asphalt Ratio		0.6 - 1.2
VMA		14% minimum

3.1 PHASE I

3.1.1 Effect of Changes in Asphalt Binders

Maximum Specific Gravity (Rice Method) - Test results for all binders indicated that binder type had no effect on the maximum specific gravity. For all three gradations, the Maximum Specific Gravity varied by +/- 0.011 or less at any asphalt content (Figure 2). This is within normal testing tolerance for any pair of samples (See AASHTO T-~~206~~). 209

Compacted Bulk Specific Gravities (AASHTO T-165) - Bulk Specific Gravities for all asphalt binders at any one gradation were within 0.04 gravity units. This is twice the normal testing tolerance for a pair of samples (See AASHTO T-165). This level of variability held true at 1st and 2nd compaction. The ranking of gravities at a gradation was random and varied with asphalt contents (Figure 3). The rankings also vary from one gradation to another. Asphalt binder type does not appear to have any effect on specific gravity of compacted mix.

Hveem Stability Values (AASHTO T-246) - The Hveem stabilometer is a triaxial testing device consisting of a rubber sleeve within a metal cylinder containing a liquid which registers the horizontal pressure developed by a compacted test specimen as a vertical load is applied.

The OSHD varies from the standard Hveem Mix Design procedure in the following way. The compacted samples are tested by AASHTO T-246. This is called "1st stability" by OSHD. The samples are then reheated to 250°F and inverted. They are recompacted according to AASHTO T-247, and tested again for Hveem stability. This is called "2nd stability."

The asphalt binder type had very little effect on stability test results. At any of the three gradations, the stabilities varied by up to +/- 4 at a given asphalt content. This is within normal testing tolerance.[1] On many tests, the variation was less than +/- 4. With one exception the different asphalt types were ranked randomly (Figure 4).

At the finest gradation, and at the .45 power curve gradation the Chevron CA(P)-1 ranked last in average stability through a common asphalt content range. At the coarse gradation CA(P)-1 ranked next to last on 1st stability, and 5th out of 7 on 2nd stability. Chevron CA(P)-1 seems to inhibit stability.

Index of Retained Strength (IRS) (AASHTO T-165) - The AASHTO T-167 procedure to fabricate the IRS test specimens specifies that, "Not less than three specimens shall be prepared for each asphalt increment and the average of the three shall be reported as compressive strength." Duplicate samples must be fabricated and subjected to the prescribed water soaking to determine the Index of Retained Strength, sometimes called Immersion Compression Testing.

The AASHTO procedure was modified so that we fabricated duplicate samples at the low, middle, and high asphalt contents within a series of tests. This gives an indication of the effect the asphalt film thickness has on the water sensitivity of the mixture and allows a chance to relate this sensitivity to asphalt content. OSHD design criteria specifies that wet (conditioned) compressive strength is to be at least 75% of the dry (unconditioned) strength at the recommended asphalt content.

At each gradation, the asphalts were ranked in random order for Index of Retained Strength with one exception. ELF PAC 20 ranked best at stripping resistance at the coarse and fine gradation, and second best at the 0.45 power curve gradation. This ranking was determined by average IRS over a common range of asphalt contents (Figure 5).

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[1] Lund, J. and Boyle, G.E., Stabilometer "S" Value Study on Asphalt Concrete Samples, Oregon Department of Transportation Document No. 1733, 1985

Most of the asphalts achieved a minimum of 75 IRS with the addition of enough asphalt at all gradations. Resistance to stripping as determined by IRS is independent of asphalt type with the exception of ELF PAC 20.

Index of Retained Resilient Modulus (IRMR) - Ratio 2

(Ratio 2 = Freeze-Thaw M_R divided by unconditioned M_R X 100)

This procedure follows the Lottman method by testing the compacted bituminous mixture in the diametral mode using the Retsina Mark VI equipment. The compacted briquettes are tested unconditioned. They are then subjected to vacuum saturation, then frozen for 24 hours, warmed in a water bath at 140°F for 24 hours, then cooled to a test temperature of 77°F for 4 to 6 hours. Since the Resilient Modulus Ratio 2 is a more severe test than the Index of Retained Strength, the index percentage is reduced to 70 minimum.[2]

The test results indicated that a significant increase in binder content was required to protect the compacted mixture from moisture and freeze-thaw damage compared to the IRS results. As much as a 1% increase in asphalt content was required to meet design criteria at the coarse gradation over the IRS test results.

The same trends were noted for the IRMR as the IRS. Most rankings of asphalts were random as to IRS test results at a given asphalt content (Figure 6). This was true for each gradation. Again the ELF PAC 20 was ranked best for the coarse gradation, second for the 0.45 gradation, and third best for the fine gradation. Resistance to stripping as determined by IRMR is independent of asphalt type with the exception of ELF PAC 20.

With the exceptions noted for Chevron CA(P)-1 and ELF PAC 20 on specific tests, no consistent relationship can be established between asphalt binder types and the results of tests performed on mix using these asphalts. The conclusion is that for mixes with gradations at coarse and fine extremes, and at a gradation near maximum density the type of asphalt used in the mix has little or no effect on results of mix design tests.

3.1.2 Effects of Changes in Gradation

In this portion of the discussion, the test result used will be the averages of all asphalts evaluated (Figures 7 through 11).

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[2] Dickinson, L., et. al., Evaluation of Asphalt Stripping Test, Oregon Department of Transportation, 1989

Specific Gravity, Maximum Specific Gravity, and % Air Voids - The effects of changes in gradation on maximum specific gravity, compacted bulk specific gravities, and % air voids were examined (Figure 7). Through the range of asphalt contents examined, the reduction in maximum specific gravity per increase in asphalt content remained the same for all three gradations. The maximum specific gravity fell about 0.015 per 1% change in asphalt content. This indicates that maximum specific gravity is affected by asphalt content but not by gradation.

The compacted bulk specific gravity of the mix was effected strongly by the gradation. At the coarse gradation the bulk gravity of the mix drifted up for 1st and 2nd compaction at about .04 units per 1% asphalt increase. The gravities didn't peak, indicating that the voids were still not full of asphalt.

With the .45 power curve gradation the bulk gravities of the mix for 1st and 2nd compaction rose at about .06 units per 1% asphalt increase. At about 5.5% asphalt the gravities converged with the maximum gravity and decreased with it as the asphalt percentage got higher. The voids were filled with asphalt at 5.5%.

The reaction of the gravities with the fine gradation were similar to the .45 gradation in that they increased at .06 units per 1% asphalt increase until they converged with the maximum gravity. This occurred at about 6.5% asphalt content indicating an ability with this gradation to maintain voids slightly longer than the .45 gradation.

Two OSHD design criteria that are used for standard duty Class "B" asphalt concrete are minimum air voids of 5.0% at 1st compaction, and 2.0% at 2nd compaction (Figure 5). The coarse gradation met these void criteria from 4.5% to 5.7% asphalt content.

The fine gradation met the design voids criteria from 4.0% to 4.7% asphalt content. The .45 gradation met the design voids criteria from 3.5 to 4.2% asphalt content. Acceptable void contents were available over a very narrow range of asphalt contents for the fine and 0.45 gradations. Some of this range is shown to be impractical by other test results.

Stability - Hveem stability is an indicator of the ability of a mix to resist plastic deformation. It is generally considered a measure of "interlock" of the aggregate matrix. The higher the stability, the more a mix should resist rutting.

At the coarse gradation, we see little change in first stabilities throughout the range of asphalt contents (Figure 8). This is probably due to the fact that the aggregate particle arrangement is constant with changes in asphalt. There is enough asphalt coating to bind the particles together, but not enough asphalt to cause them to move apart. The percent air voids were above 3% at 1st compaction, and 1% at 2nd compaction throughout the range of asphalt contents, allowing room for more asphalt.

However, all the stabilities at 1st compaction, and the 2nd compaction stabilities above 6.0% asphalt are below the minimum design criteria of 35. This is because there are not enough particle to particle contacts within the coarse mix sacrificing internal friction needed to resist plastic flow.

At the fine gradation the mix remains at a higher stability (38+) until the asphalt content reaches 5.0% and then the stabilities drop rapidly to an average of 8 stability at 6.5% asphalt. At 5.0% asphalt the fine gradation voids drop below 4.5% and 2.0% for 1st & 2nd compaction. Additional asphalt causes the particles to be forced apart, reducing the particle contact and the internal friction.

The fine gradation second stability decreased more rapidly than first stability with increased asphalt due to a faster reduction in voids. At 6.5% asphalt the voids are 0% and the stability is near 0. The aggregate is "floating" in the asphalt, with no rock to rock contact.

The 0.45 gradation develops even lower stabilities. At both first and second stability the minimum of 35 occurs only below 4.5% asphalt. The stability decreases more sharply with additional asphalt than the fine gradation. This gradation is close to the densest possible, and is only stable with the minimum asphalt needed to bind the particles together. Additional asphalt reduces the voids even more quickly than the fine gradation. This additional asphalt pushes the particles apart, and "lubricates" the mix, rapidly reducing the internal friction. Stabilities reach a very low level above 5.5% asphalt. Void contents for this gradation are 0.8% at 5.5% asphalt for first compaction and 0 for second compaction.

At the 0.45 gradation, there is a lot of variation between second stability test results at the same asphalt content (Figure 9). We have already concluded that asphalt type does not affect stability test results. The value derived from performing second stability tests is seen here. This 0.45 gradation is very sensitive to small variations in asphalt content or gradation changes. The variability in performance was not discovered during first stabilometer testing. Second stability simulates additional compaction due to traffic. The second stability testing demonstrated just how variable the performance of the mix can be with small variations in mixture components. Second compaction is a good tool to evaluate mix performance sensitivity to small variations in components.

Void content seems to affect stability more than asphalt content does. Too many voids, as in the coarse gradation, causes the mix to be unstable. Too few voids, as in the fine mix, causes the stability to fall rapidly with decreasing voids. The critical void content where the stability falls below 35 is:

Coarse Mix	1st compaction - All	2nd compaction - 1.7%
Fine Mix	1st compaction - 3.5%	2nd compaction - 1.0%
0.45	1st compaction - 5.2%	2nd compaction - 1.3%

Index of Retained Strength - Resistance to asphalt stripping, as measured by the Index of Retained Strength test, is a function of the amount of asphalt coating the aggregate, the sealing of the mix from the environment, and the inability to generate pore water pressure in the mix.

The results of this test are quite variable. Disregarding the asphalt types mentioned earlier which enhance or reduce stripping resistance, the precision of any set of tests is about +/- 10%.

The general trend for all three gradations is that as asphalt content increases the Index of Retained Strength increases (Figure 10). This does increase the coating of the particles, and reduces voids which water and air can penetrate, as we have already seen.

Because of the increased voids, the coarse gradation does not rise above the minimum design criterion of 75 until it contains 5.3% asphalt. We can assume that the particles are well coated because of the low surface area of the coarser mix, and the pores are too large to generate any pressure, so the critical factor must be exposure to air and water in the voids. At 5.3% asphalt the voids are reduced to the point where aggregate-asphalt bonds are well protected. The void content at this point is 6.0%.

The mix with the fine gradation resisted stripping better than the coarse mix at all asphalt content. The average IRS never fell below the minimum 75. The IRS for the fine mix also increased with increased asphalt content. The mix has a sufficiently small percentage of voids, and the particles are well coated. The voids, in fact, are always below 6%.

At the densest gradation on the 0.45 power curve, we see a new phenomenon. The IRS is very low at low asphalt content. At 3.5% asphalt, it is below 65. As the voids reach 6% the IRS is still below 75. There is possibly a critical pore size created which generates pore water pressure. As the asphalt content increases to above 4.2% the IRS continues to increase above 75. It continues to rise to about an average of 93 at 6% asphalt. Again, the lower the voids the better the protection from weathering.

In summary the fine mixes offers the best stripping resistance, but even the coarsest mix can be protected by holding the void content below 6.0%. Fewer voids offer better stripping resistance if pore size is kept above some critical level.

Index of Retained Resilient Modulus - The Index of Retained Resilient Modulus (IRMR) is an indication of the ability of the mix to retain stiffness by resisting stripping.

Lottman has shown that as asphalt strips from the aggregate the resilient modulus decreases.[3]

The mix at the coarse gradation does not reach the minimum design criterion of 70 until it contains 6.5% asphalt (Figure 11). While this mix did achieve minimum IRS values at lower asphalt content, the vulnerability of this coarse mix is exposed by the more severe IRMR test. Because of fewer contact points in the coarse mix, each area that is stripped reduces the ability of the mix to maintain stiffness. An OSHD study[2] has shown that because IRMR is a more severe test, a value of 70 correlates with a 75 IRS value for stripping resistance.

The fine gradation follows the same pattern. The IRMR is below 30 at 4.5% asphalt and rises to the design minimum of 70 at 5.7% asphalt. It continues to improve to 90 at 6.0% asphalt. Above 6.0% asphalt, asphalt fills the voids and rock to rock contact is reduced. The stiffness of apshalt alone then begins to be measured.

At lower asphalt contents the fine gradation IRMR is lower than the coarse IRMR until the asphalt content gets above 5.5%. Particle to particle contact does not explain this because the contacts in the coarse mix are fewer. We theorized that there is pore water pressure generated in the conditioned samples of fine mix that is not generated in the coarse mix. This pore water pressure loosens the asphalt-aggregate bonds and reduces modulus in the conditioned samples. As the asphalt content increases, the size of the pores is reduced and they no longer contribute to stripping.

The 0.45 gradation mix acts similarly to the fine mix. At low asphalt content, the 0.45 gradation mix has lower IRMR than the coarse gradation mix. As asphalt content increases IRMR of the 0.45 gradation mix becomes larger than the IRMR of the coarse mix. Generally the 0.45 gradation mix has a higher IRMR than the fine mix. This is possibly because the voids in the 0.45 gradation mix are lower than the fine mix and better protect the mix. The amount of critically sized pores are also reduced.

No asphalt content met all design criteria with any gradation in PHASE I. No mix would perform satisfactorily based on these performance predictors (Figure 16). The coarse mix had low stabilities at first compaction and was not very resistant to stripping. The fine mix had good stabilities and satisfactory voids at low asphalt contents, but at low asphalt contents, the fine mix was not stripping resistant. The mix on the 0.45 power curve also had good stabilities and satisfactory voids at low asphalt contents, but could not resist stripping at low asphalt contents.

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[3] Lottman, R.P., "Predicting Moisture Induced Damage to Asphaltic Concrete", NCHRP Report 192, Transportation Research Board, 1978

[2] Dickinson, L., Evaluation of Asphalt Stripping Test, Oregon Department of Transportation, 1989

Because the gradations of each mix are so extreme, none provide a very broad range of acceptable voids or stabilities. The gradation extremes also eliminate effective stripping resistance by over exposure of the coatings to moisture or by allowing excess pore water pressure to build.

To develop a mix which has acceptable properties, it seems that a compromise between these extreme gradations should be chosen. Since the current philosophy is to coarsen mixes to resist heavier traffic loads, we should examine a gradation somewhere between the densest and the coarsest extremes. This gradation would fall somewhere between the 0.45 power curve and the coarse broadband limit.

By experience, OSHD had developed separated aggregate size gradation specifications. These have historically allowed good quality Class "B" mixes to be developed. They force the combined gradation to be in the range we want to examine. These separated size gradations will be the basis for the PHASE II testing.

3.2 PHASE II

PHASE II differed from PHASE I in that aggregate gradations were not at extreme specification limits or at maximum density. The gradation in PHASE II was set at midpoint in the separated size gradation specification for 3/4" - 1/4" and 1/4" - 0" separated sizes (Table 6). The 1/4" sieve was set at 58% to determine percentages of each separated size to use. By experience, this has been a successful % to use for the 1/4" sieve.

TABLE 3.6: Phase II Combined Gradation

Separated Size Specification			Phase II Combined Gradation
Sieve Size	**3/4 - 1/4**	**1/4 - 0**	
1"	99 - 100	100	100
3/4"	85 - 95	100	96
1/2"	52 - 68	100	84
3/8"	---	99 - 100	73
1/4"	0 - 16	86 - 100	58
#4	---	---	50
#10	0 - 10	41 - 56	31
#40	0 - 6	13 - 29	13
#200	0 - 2	4 - 11	4.4

The PHASE II gradation follows the maximum density line on the larger sieve sizes (Figure 12). This is generally at mid-range of the broadband specification for these sieves. From the 1/4" to the #200 sieves, the PHASE II gradation dips below (coarser) the mid point of the broadband specifications, and well below (3 - 6%) the maximum density line.

PHASE II testing used only three asphalt binders: Chevron AC-20, Elf PAC-20 and Asphalt Service & Supply AC-20R. The effects of asphalt were shown to be almost insignificant in PHASE I so the number of asphalts were limited in PHASE II to reduce the variables involved. Each "type" of asphalt was used to see if asphalt types influenced test results near an "optimum" gradation. If the differences are significant, this will be apparent. If the differences in test results for each asphalt are insignificant, this will provide multiple results on each test which will minimize testing variation.

Specific Gravity

Since aggregate from a different pit was used on PHASE II, no conclusions about gradation can be drawn directly from specific gravity tests. There was a consistent difference in bulk specific gravity for the three asphalt types though. The straight AC-20 was compacted to the densest mix, the AC-20R was compacted to the lightest mix, and the PAC-20 was consistently in between the other two asphalt mixes (Figure 13). This would seem to indicate that the modified asphalts are slightly more difficult to compact. The difference from highest to lowest is about 1% - 1.5% compaction This seems to be an actual difference because the maximum specific gravities of all asphalts are very tightly grouped and somewhat random.

In terms of voids, the PHASE II voids most closely resemble the fine blend of PHASE I. At a reasonable void level of 5% - 6% at first compaction and > 2% at second compaction, the asphalt content is near 4.5% for the PHASE II samples, as well as the fine blend of PHASE I. The PHASE I coarse blend developed these void levels at 5.5% asphalt, and the 0.45 power curve gradation for PHASE I developed these voids at about 4.2% asphalt. Recall that for the coarse blend and the 0.45 curve blend, these asphalt contents were well below the asphalt content to achieve minimum IRS and IRMR. This is also true of the PHASE I fine gradation. The range of asphalt contents over which the voids are acceptable is slightly wider in PHASE II than any PHASE I gradation.

Stability

Stability at first and second compaction were similar to stabilities for the PHASE I fine and .45 gradations. A satisfactory level of stability (35+) was maintained up to some critical asphalt content, then dropped rapidly with increased asphalt (Figure 14). The PHASE I coarse gradation was never at a satisfactory level.

The difference in response of the PHASE II gradation is that the stability at 1st compaction was at an acceptable level (35+) through as wide or wider range of asphalt contents before it dropped off. Generally, the PHASE II gradation held good stability up to about 5.5% asphalt, while the PHASE I gradations dropped rapidly above 4.5 to 5.0% asphalt. This gives this gradation a much better chance of maintaining stability in the mix.

Index of Retained Strength & Index of Retained Resilient Modulus

Discussion of these tests has been combined because of similar responses. This was the area with the most dramatic improvement because of gradation.

In PHASE I, the coarse gradation met a minimum of 75 for IRS and 70 for IRMR only above 6.3% asphalt. The fine blend only met these minimums above 5.6% asphalt. The 0.45 gradation met these minimums above 5.2% asphalt.

For the PHASE II samples the minimum test results were met at all asphalt percentages above 4.3% (Figure 15). This adjustment in gradation has provided adequate protection from stripping through exposure to water, while maintaining adequate voids to prevent excess pore pressures.

One of the goals of this study was to develop a gradation that would resist other failure criteria and inhibit strip ravelling. This gradation has been successful in resisting stripping, while improving the rutting resistance of the mix as measured by void maintenance and stability.

General Discussion of PHASE II

When examining PHASE I and PHASE II test results in the framework of OSHD design criteria, some significant improvements can be seen (Figure 16). At no point in all the PHASE I testing were all the OSHD design criteria met. Even by reducing the first stability criteria to 33 minimum there is no point in PHASE I where all criteria are met.

In PHASE II with the exception of first stability, all criteria are met from 4.3 to 4.8% asphalt content (Figure 19). First stabilities were all above 33. This is an acceptable stability for many mix applications. Still, the range of acceptable asphalt content is too narrow to be practical. Paving projects typically vary about +/- 0.4% in asphalt content in normal operations. If the mix testing does not indicate good performance over this wide a range of asphalt contents, it will probably not perform well in portions of the road being paved. PHASE II gradation is a marked improvement over PHASE I in terms of test results, but is not a serviceable gradation.

The coarse fraction (> 1/4") was not changed in PHASE II very much from the 0.45 gradation in PHASE I. Even the passing 1/4" in PHASE II is only 2% less than in PHASE I. The change that most influenced the test results was dropping the 1/4" - #200 fraction below the maximum density line 3% to 6%. While this change does not approach the broadband limit, it does open up the finer matrix of the mix enough to reduce pore pressure in the saturated mix without creating so many voids that the mix becomes unstable or overexposed to stripping. Void maintenance seems to be controlled much more by the finer half of the mix.

3.3 PHASE III

The most successful adjustment in the study was made in PHASE III. In PHASE III all the improvements gained in PHASE II were maintained and several mix responses were improved over PHASE II. Specifically, the range of asphalt contents where desired voids were achieved was broadened, stabilities at 1st compaction were above 35 for many asphalt contents, and IRS and IRMR values were above minimum acceptable limits over all asphalt contents.

The aggregate and asphalt used in the test samples for PHASE III were identical to those in PHASE II. The adjustment made for PHASE III was to move the gradation of the 3/4" - 1/4" material nearer to the coarse limit of that separated size specification (Table 6). The percent passing 1/4" was maintained at 58%. The gradation below the 1/4" sieve was identical to PHASE II gradation. The effective change was to increase the amount of coarse material in the upper part of the gradation (Figure 17).

<u>Specific Gravity</u>

The effect on specific gravity is negligible. The noticeable difference from PHASE II in specific gravities is that the asphalt types are not ranked by specific gravity of the mix, but are now random throughout the asphalt range (Figure 18). This is probably due to an increase in voids at any asphalt content. This might reduce the physical effects of an asphalt on the compaction characteristics of the mix. The compaction seems to be dependent completely on aggregate characteristics at this gradation.

Another benefit derived from this gradation change is the increased latitude of the voids. In PHASE II at 6% asphalt the voids at first compaction were 1.1% and at second compaction were 0%. At the PHASE III gradation the voids at first compaction were 2.0% and at second compaction were 0.4%. At a high asphalt content this would result in more ability to resist rutting. At any void level this gradation would require less asphalt producing a cost saving (Figure 19).

<u>Stability</u>

The first noticeable difference in stability of this gradation is the result at first compaction (Figure 20). The lowest value is 34 compared to 27 in PHASE II. The more dramatic difference is that the stabilities do not drop off at higher asphalt contents like they did in PHASE II. Apparently the extra amount of large aggregate, coupled with a slight increase in voids, allows sufficient aggregate interlock and load transfer to maintain good stability even when higher asphalt contents begin to force the aggregate apart.

Apparently voids are not the critical difference in this maintenance of stability. At a point where the voids are 2% for the PHASE II and PHASE III mixes the stabilities are 30 and 35 respectively. Stability seems to be a function of the gradation independent of the void content.

The range at which stability is above 35 is even wider than the range at which voids are at a desirable level.

When the mix is compacted denser as in second compaction, the mixes in PHASE II and PHASE III react very much alike. The test results are almost identical. The range of acceptable stabilities (35+) is wide (4.0% to 5.6%).

Index of Retained Strength & Index of Retained Resilient Modulus

The improvement made in the response of these tests in PHASE II continues in PHASE III. Where almost all average test results were above minimum design criteria in PHASE II, all average test results were above minimums in PHASE III. The slight increase in voids has not affected the stripping resistance of the mix negatively, but has improved it slightly.

General Discussion of PHASE III

The changes made from PHASE II to PHASE III have moved the mix closer to achieving the objectives of this study. The resistance to stripping has been improved greatly, while the ability of the mix to resist rutting through void maintenance and consistent stability has been achieved.

In PHASE I no range of asphalt contents would provide compliance with OSHD design criteria. In PHASE II a range from 4.3% to 4.7% asphalt content would provide a mix that met design criteria if stability at first compaction was lowered to 33. PHASE III changes in gradation results in a mix that meets all design criteria from 4.0% asphalt to 4.9 asphalt (Figure 19). This is an acceptable range of asphalt contents to expect a supplier to provide.

3.4 PHASE IV

PHASE IV testing was incidental to the "B" MIX IMPROVEMENT STUDY. The testing was done on an actual construction project #10670, the S. City Limits Scappoose - Multnomah Co. Line. The adjustment to the gradation tested and the comparison of test results can shed some light on the subjects examined in this study.

The mix design testing was done at the gradation shown in the left column with the adjusted gradation for this report shown on the right:

Sieve	Original Gradation	Adjusted Gradation
1"	100	100
3/4"	99	99
1/2"	84	83
3/8"	74	72
1/4"	62	59
#10	28	29
#40	11	12
#200	4.1	4.3

It is apparent that the only significant gradation change was in the range of 3/8" to 1/4" material. This gradation was developed using two stockpiles 3/4 - 1/4 and 1/4 - 0. The change was made by adjusting the 3/4 - 1/4 percentage from 43% to 47%, and adjusting the 1/4 - 0 percentage from 57% to 53%. The resulting change in test results are:

Test	Original Test	Adjusted Result
Bulk Specific Gravity	2.35	2.35
Second Specific Gravity	2.43	2.43
Voids @ 1st compaction	5.1%	5.6%
Voids @ 2nd compaction	1.9%	2.4%
Maximum Specific Gravity	2.476	2.490
1st Stability	33	37
2nd Stability	39	42
IRS	108%	Not tested
IRMR	99%	Not tested

There was a clear improvement in the stabilities and voids with the adjustment to the gradation. IRS and IRMR were not examined because the results were considered high enough that a small adjustment would not affect them significantly.

The point of the inclusion of this small set of tests is to substantiate the selection of the 58% passing 1/4" sieve used in PHASE II and III. It can be seen from this PHASE IV testing that an adjustment of just 3% at the 1/4" sieve can have a significant effect on the response of the mix. This mix could probably be improved even further by increasing the amount of coarse material in the upper sieve sizes as was done in PHASE III.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 CONCLUSIONS

1. Gradations using a 1/4" sieve as the percentage control should not be set above 58% passing the 1/4" sieve.

2. Increasing the amount of coarse material in the 1" - 1/4" fraction can increase voids, stability, and/or lower the target asphalt content without jeopardizing stripping resistance.

3. The 1/4" to #200 material gradation must be kept coarser than the maximum density gradation to allow for enough voids, ensure satisfactory stability, and combat stripping.

4. The broadband gradation extremes are not suitable for asphalt concrete mixture.

5. A gradation which falls on the maximum density line throughout its range will not produce a suitable Class "B" asphalt concrete mixture.

6. At gradations in which void content is very sensitive to asphalt content, modified asphalts may reduce the ability of the mix to be compacted by 1% - 2% of theoretical maximum density.

7. This study did not address a gradation that is finer than maximum density, but below the broadband limit. The industry seems to be indicating that larger aggregate is needed to resist rutting due to heavier stress on pavements, therefore only the coarser side of maximum density was examined.

8. Fine gradations resist stripping better than coarse gradations. An excessively dense gradation does not ensure stripping resistance. Stripping can occur from voids which are small enough to generate excess pore water pressure.

9. An "optimum" gradation can protect the mix against wide swings in asphalt content. The PHASE III gradation had a wide range of asphalt contents which had satisfactory test results. The other gradations showed very narrow or no range where all test results were satisfactory.

10. Except as noted for asphalt stripping, modified asphalts do not produce a mix which performs any better statistically in lab tests.

11. IRMR stripping test is more severe than the IRS test. Both are valuable at evaluating stripping resistance, but the IRMR may be a better predictor for coarse mixes.

12. Hveem stability was shown in PHASE II and III to be a function of gradation primarily, and asphalt content only secondarily.

4.2 RECOMMENDATIONS

1. For pavements where heavy duty performance is required, gradation and asphalt tolerances should be tightened to ensure more consistent mix, even if a greater cost is incurred.

2. Use conventional asphalts for all Class "B" asphalt concrete unless evironmental or construction conditions dictate a modified asphalt.

3. Continue to test mixes for stripping with both the IRS test and the IRMR test.

4. The current optimum grading for Class "B" asphalt concrete is:

Sieve Size	Percent Passing
1"	100
3/4"	94
1/2"	81
3/8"	70
1/4"	58
#4	50
#10	30
#40	13
#200	4.5

5. Broad band specifications should be adjusted to place this gradation in the center of each grading band. Separated size gradations should be adjusted if necessary to allow for this grading when combined.

5.0 IMPLEMENTATION

At the time of publication of this study, the Oregon Highway Division has already implemented use of the gradation shown above. Further work is being done to learn the effects of adjusting the passing 1/4" fraction. Some aggregate needs this fraction adjusted to provide a quality mix. The variables which seem to influence the need to adjust fine gradation are aggregate shape and surface texture.

Heavy duty pavement specifications in Oregon have been written and implemented at this time. These specifications call for tighter tolerances on gradation, asphalt content, and moisture in the mix. These specifications also call for minimum specific gravity and maximum percent absorption limits on the aggregate. These limits aid in reducing the variability of the aggregate.

Future work may include an investigation into a mix that is graded on the fine side of the maximum density curve.

APPENDIX

FIGURES

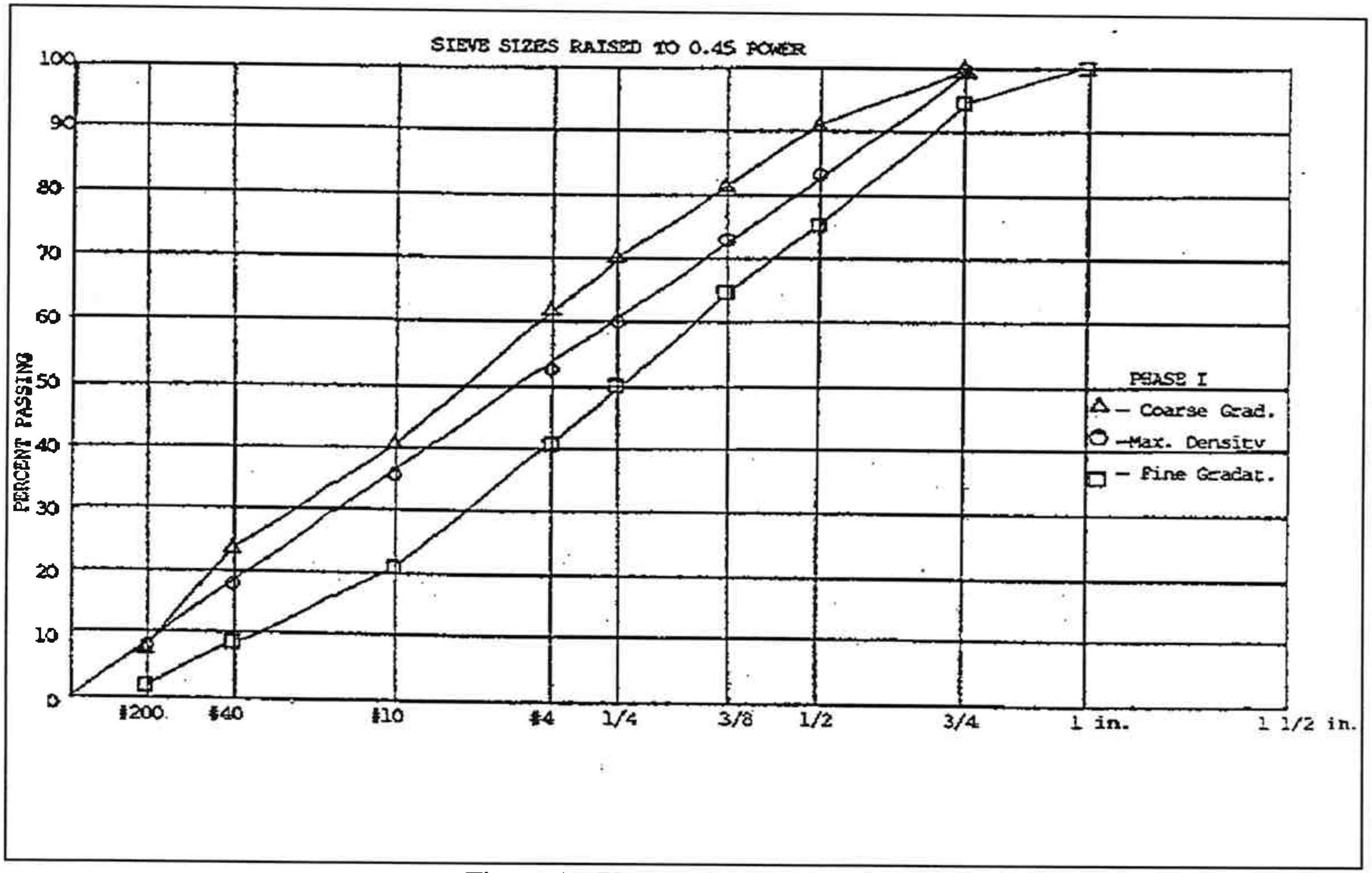


Figure 1: Phase I Gradation Chart

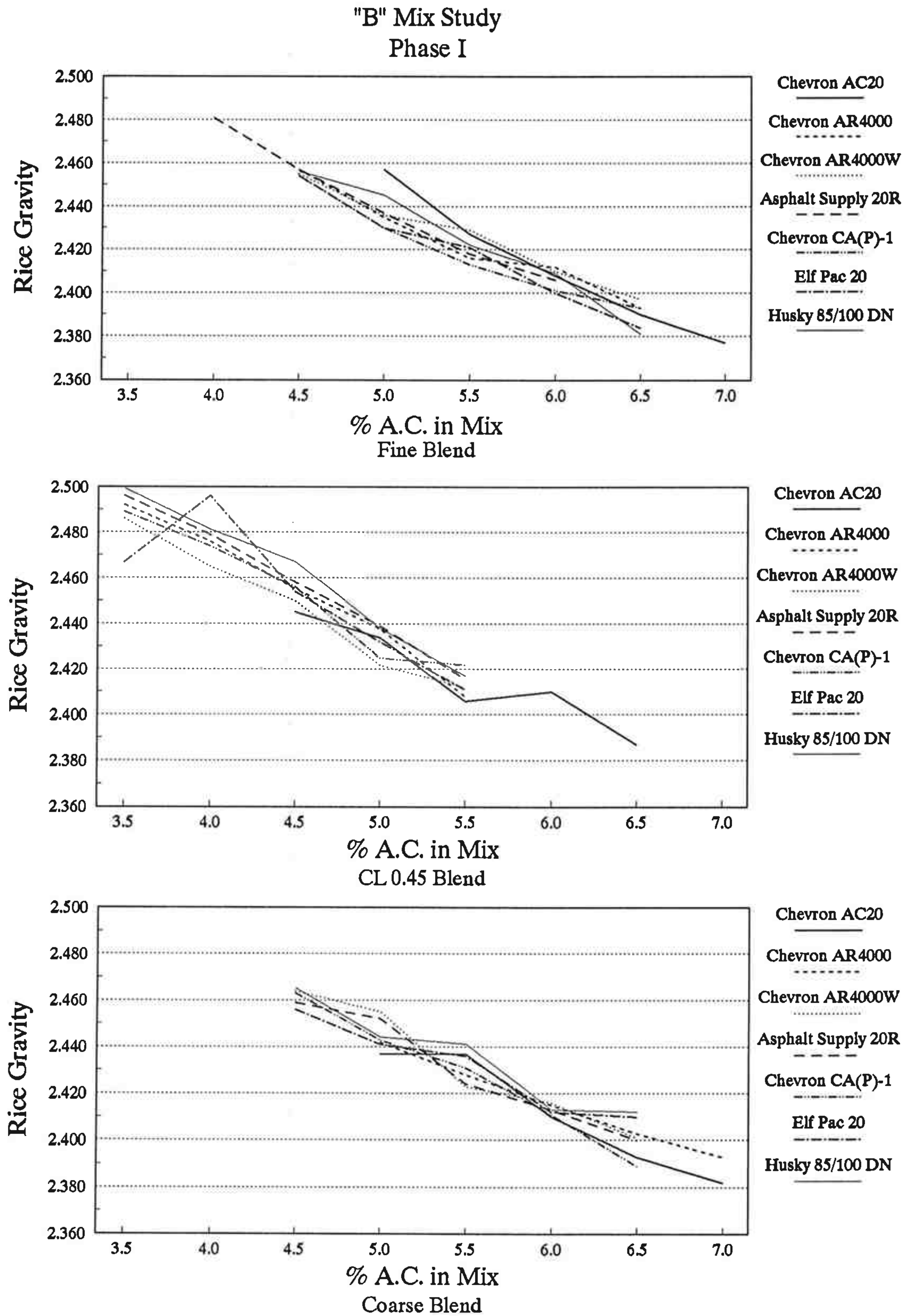


FIGURE 2: Rice Gravity Test Results

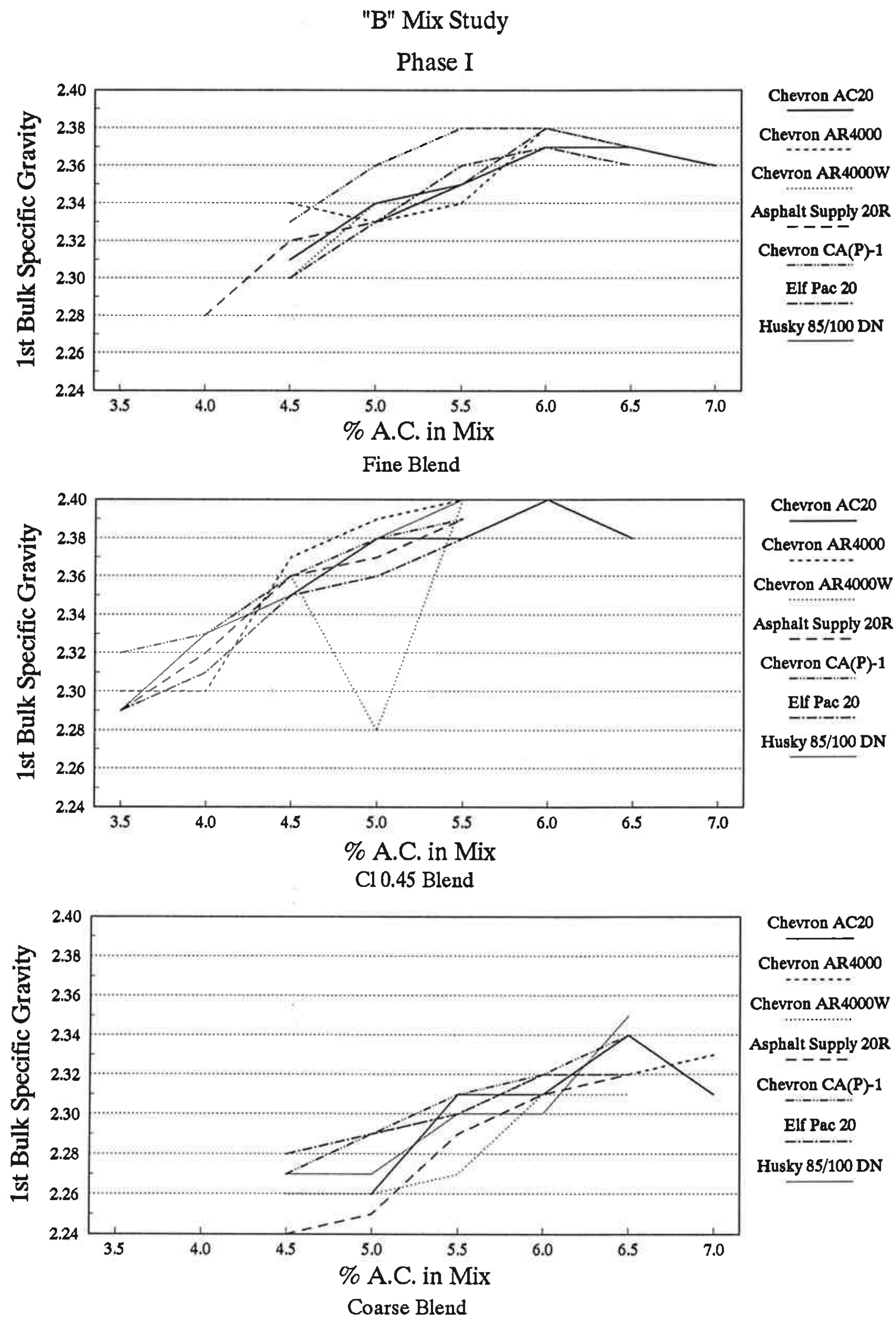


FIGURE 3: Individual Bulk Specific Gravity Test Results 1st Compaction

"B" Mix Study

Phase I

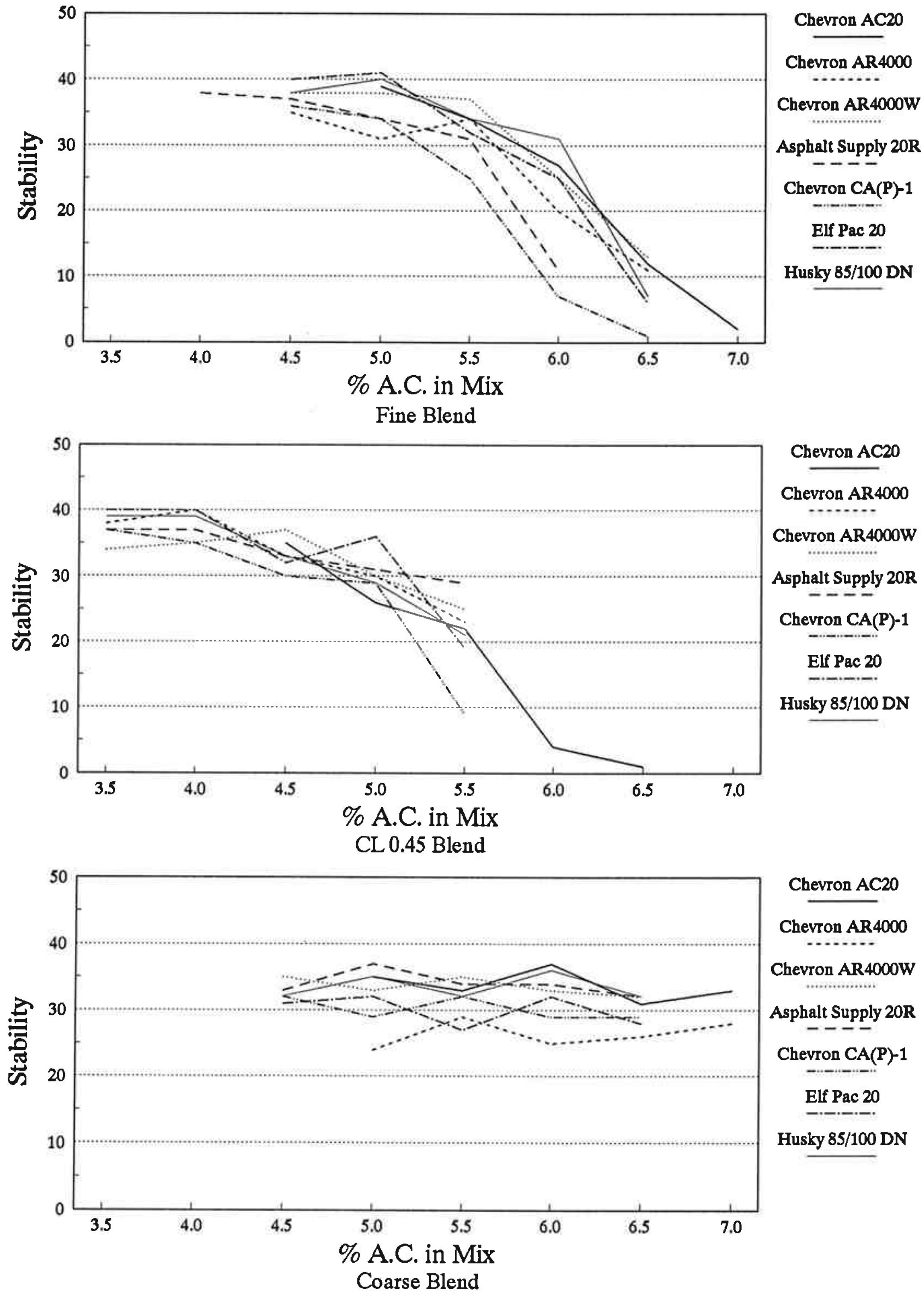


FIGURE 4A: Individual Stability Test Results

"B" Mix Study

Phase I

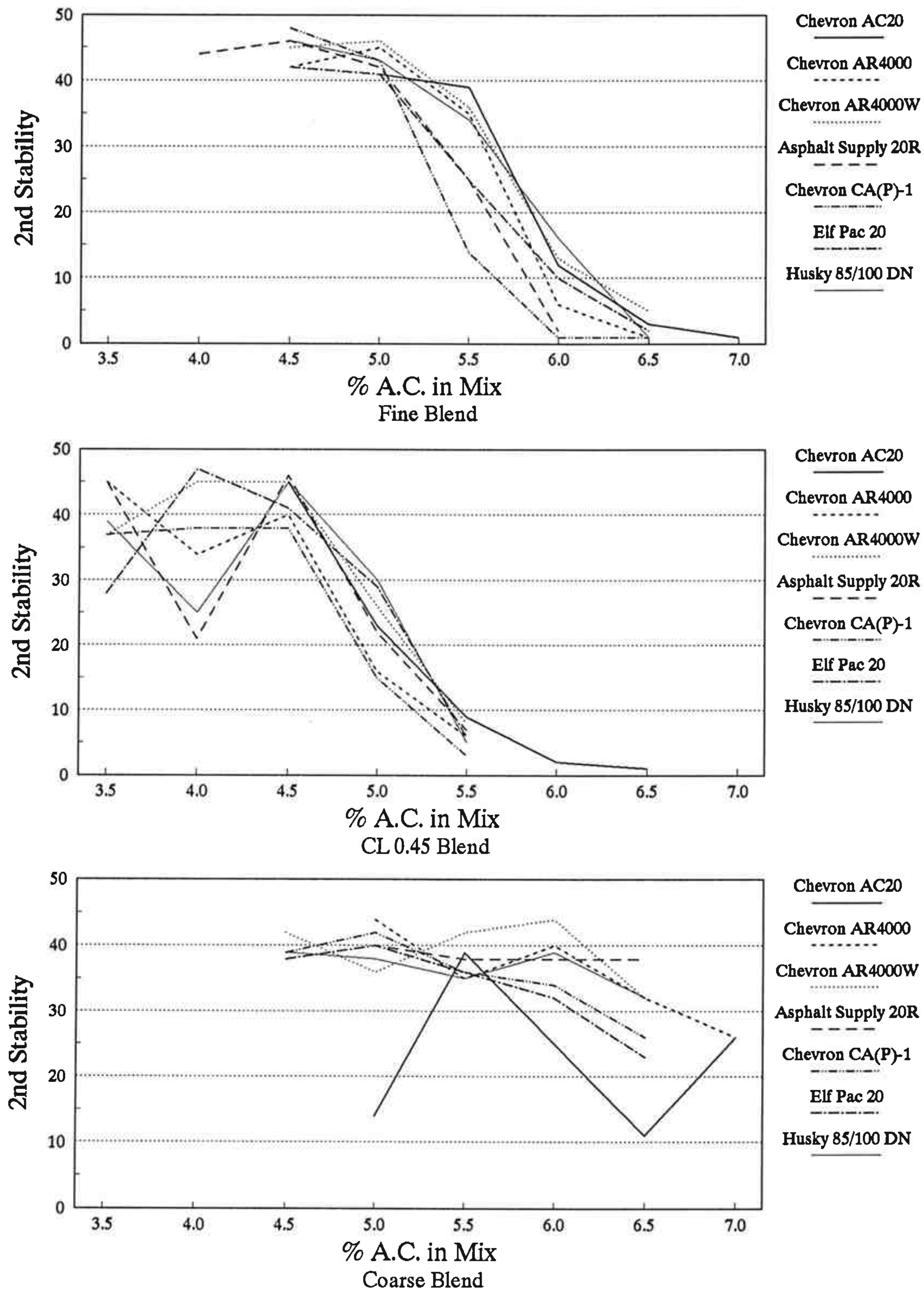


FIGURE 4B: Individual Stability Test Results 2nd Compaction

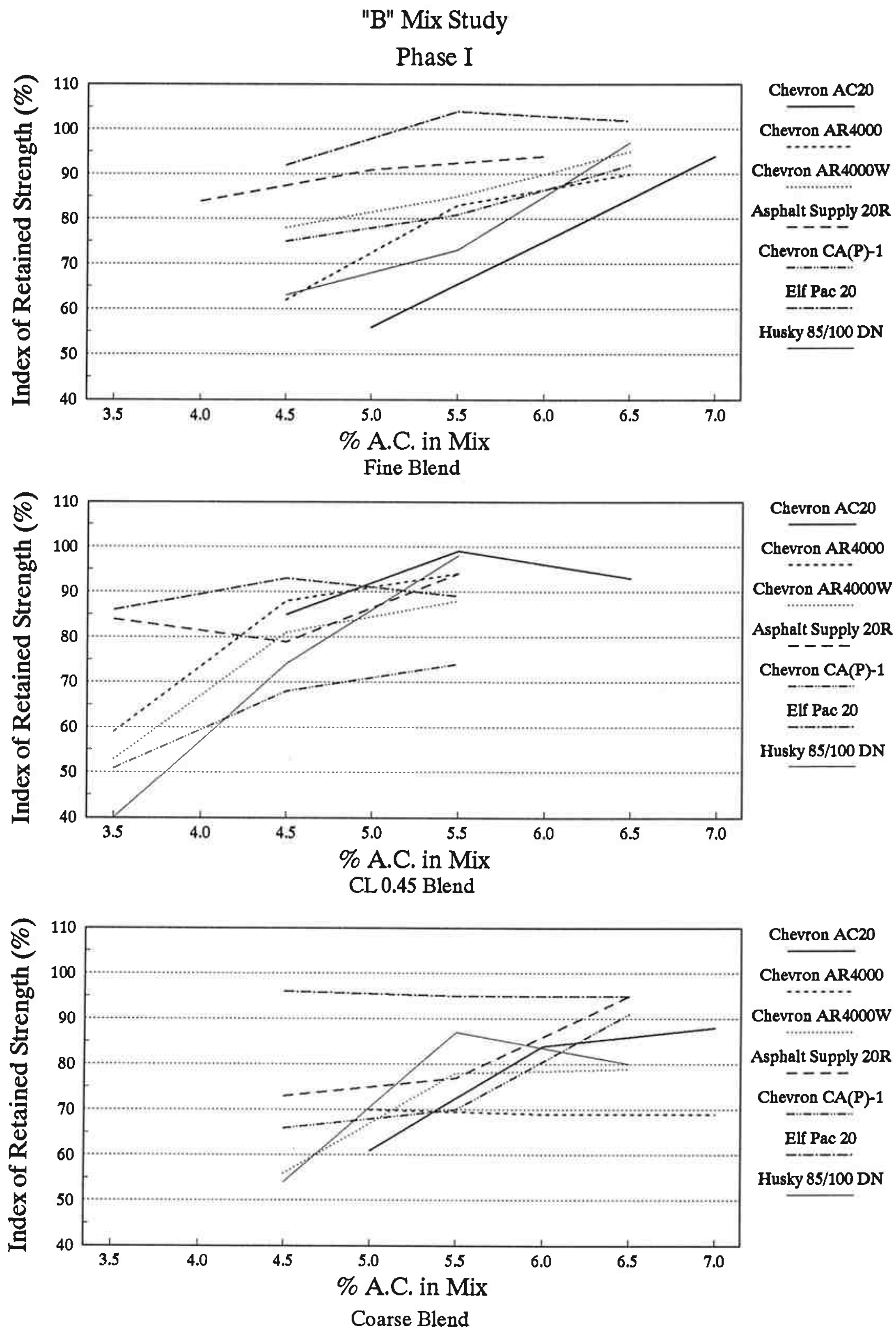
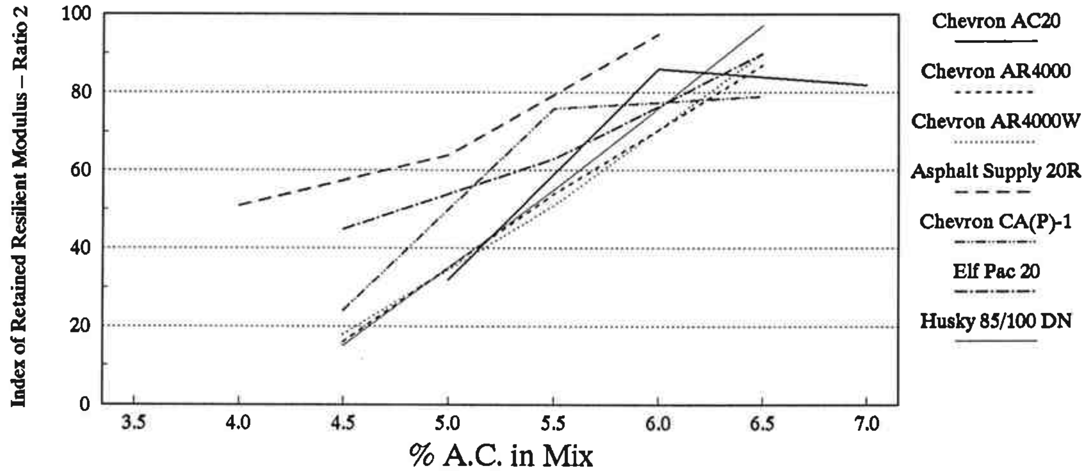


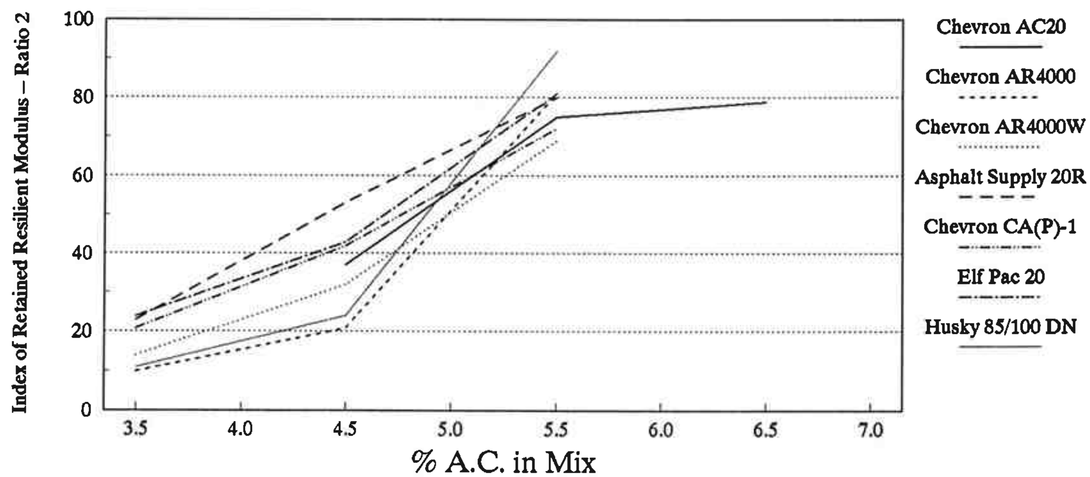
FIGURE 5: Individual Index of Retained Strength Test Results

"B" Mix Study

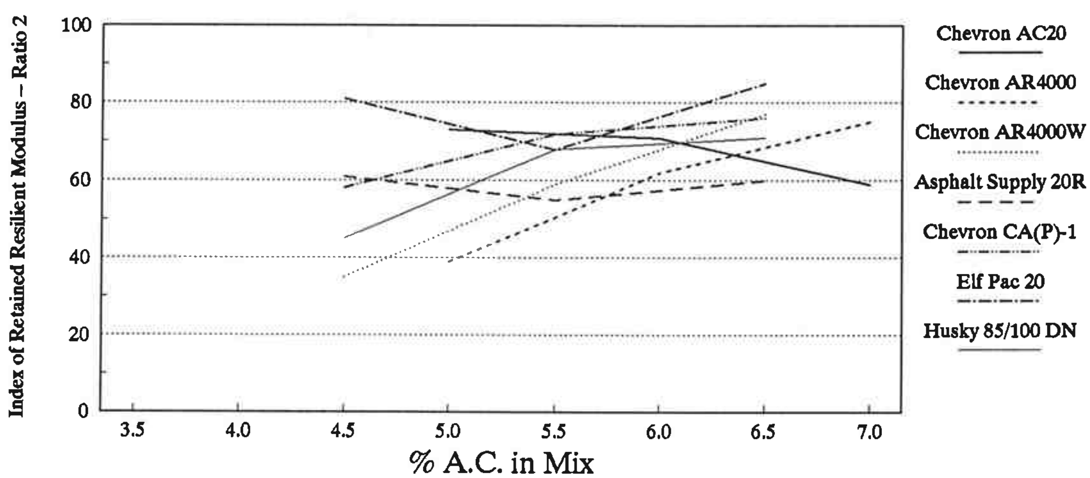
Phase I



Fine Blend



CL 0.45 Blend



Coarse Blend

FIGURE 6: Individual Index of Retained Resilient Modulus Test Results

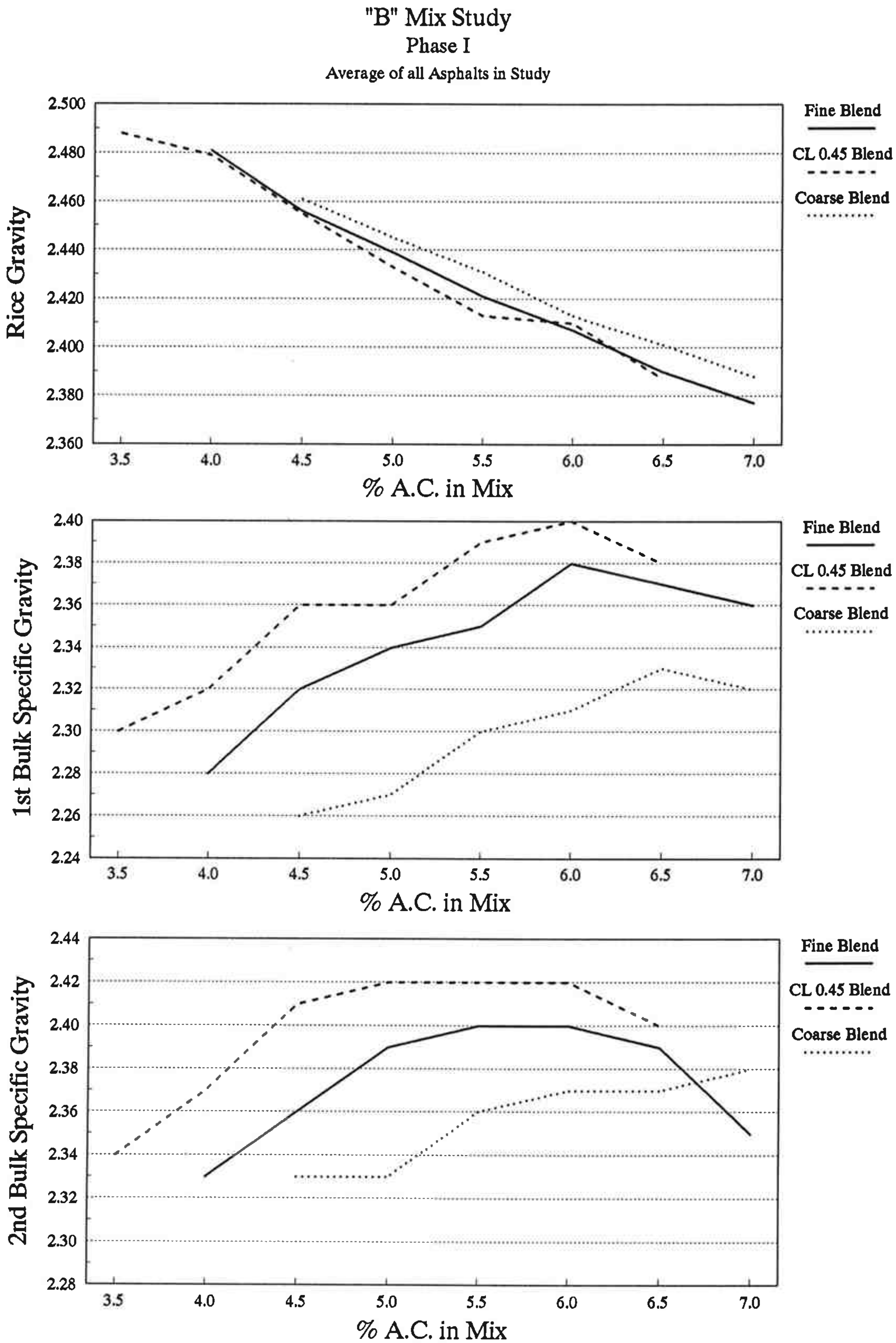


FIGURE 7: Average Test Results for Rice Gravity & Specific Gravity

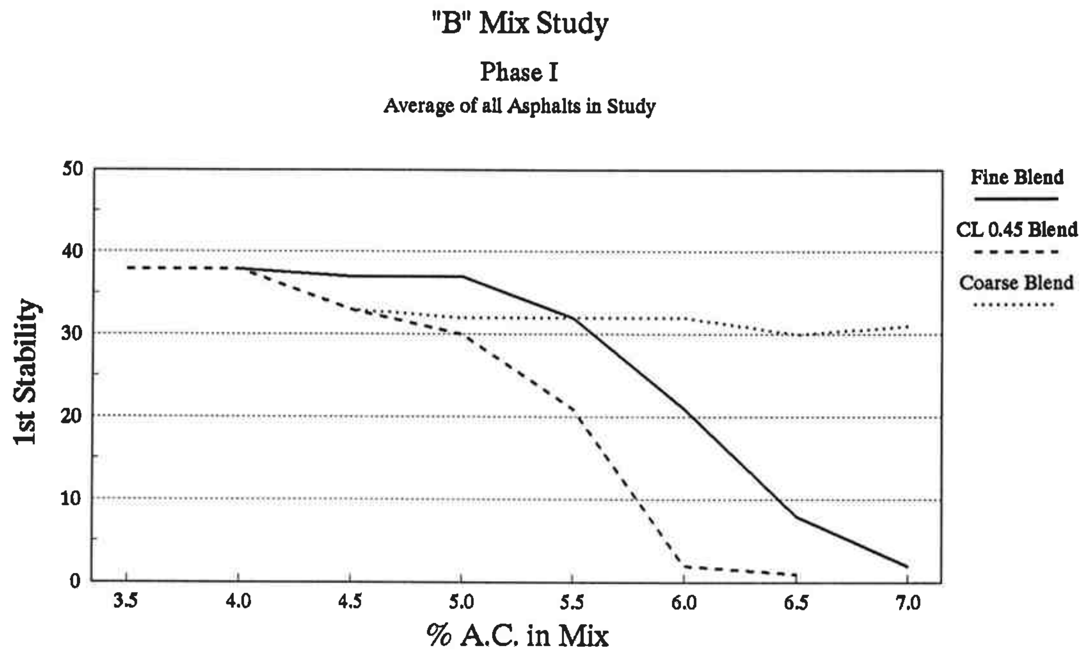


FIGURE 8: Average 1st Stability Test Results

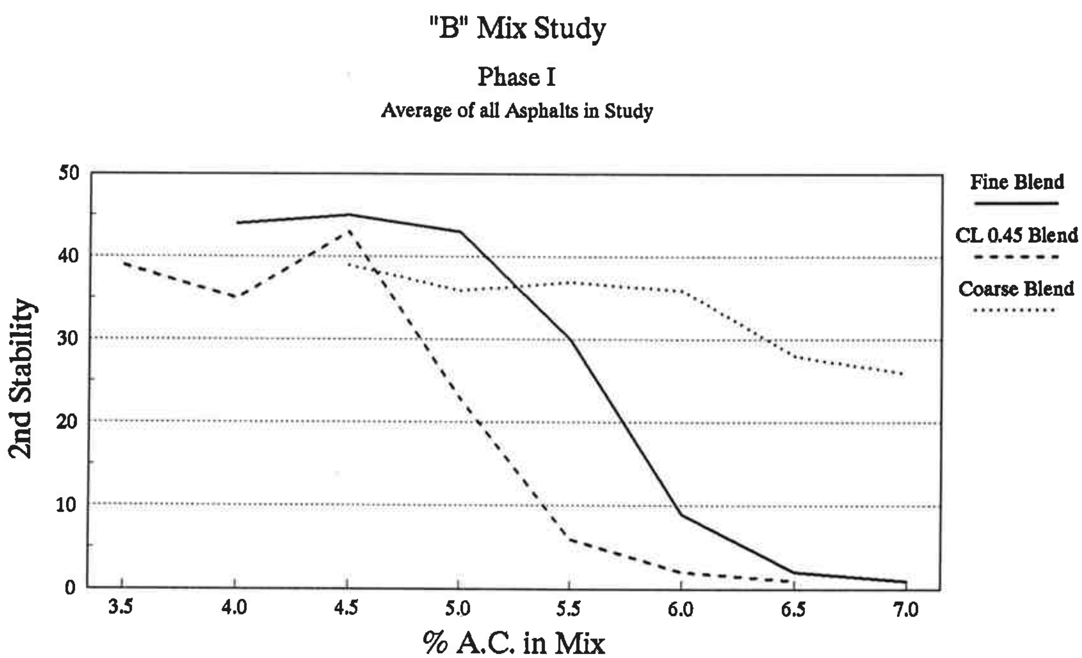


FIGURE 9: Average 2nd Stability Test Results

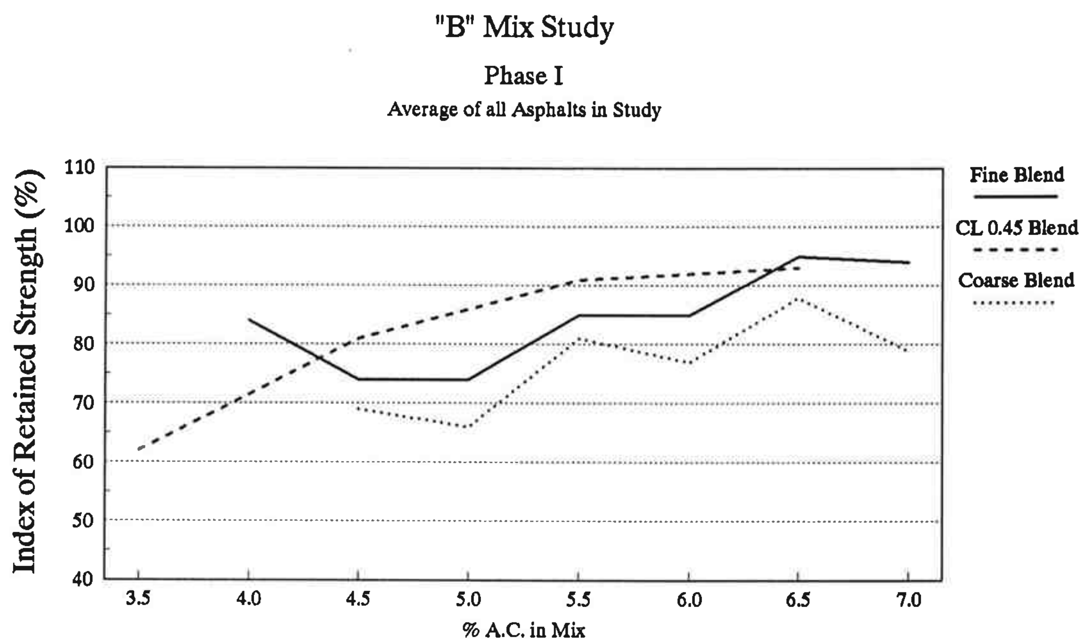


FIGURE 10: Average Index of Retained Strength Test Results

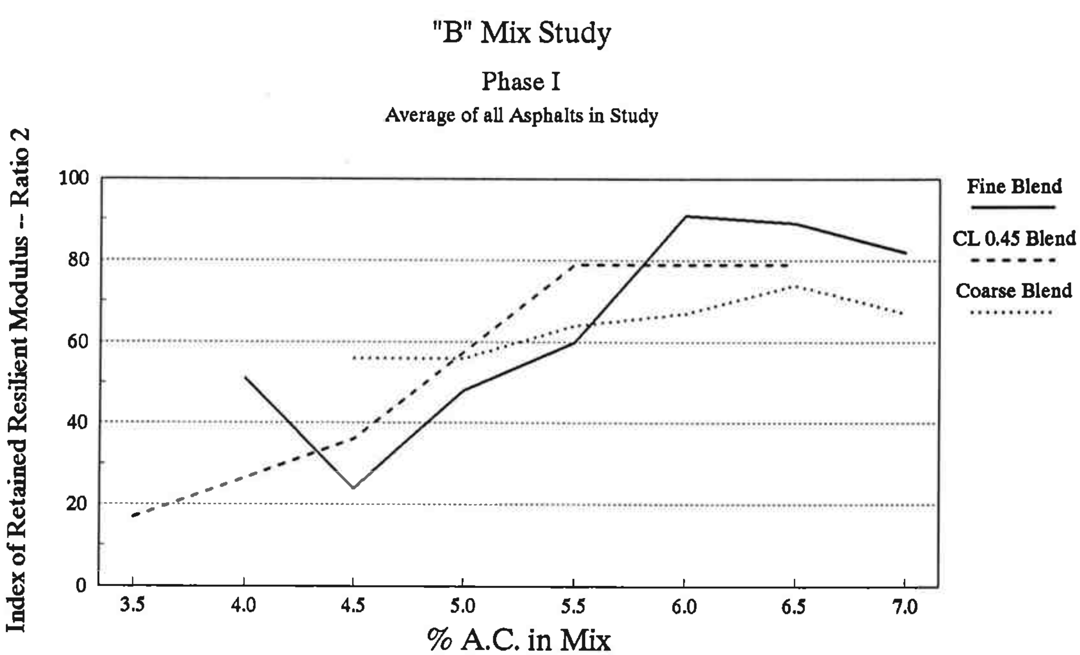


FIGURE 11: Average Index of Retained Resilient Modulus Test Results

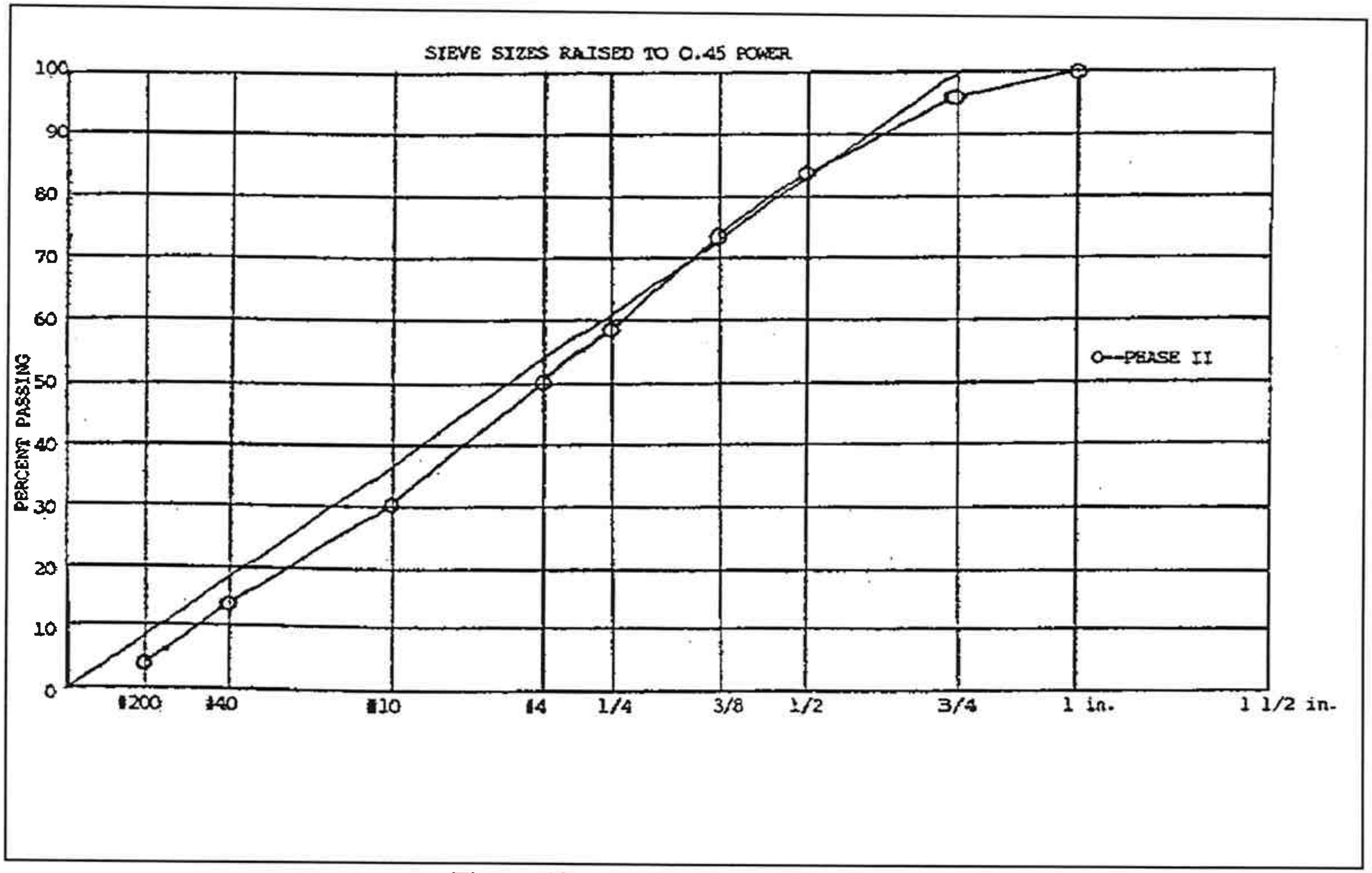
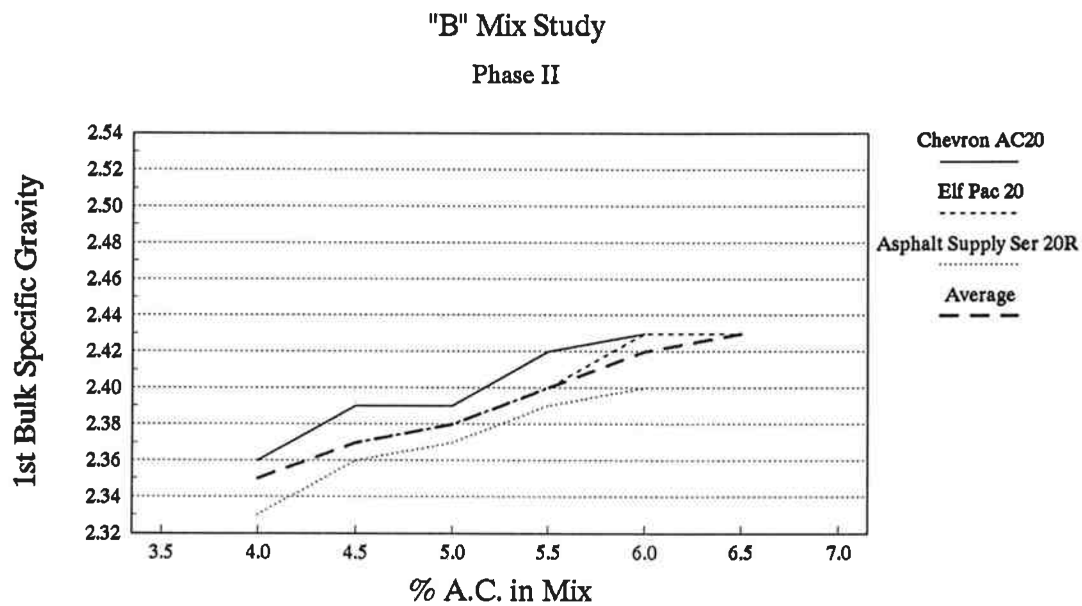


Figure 12: Phase II Gradation Chart



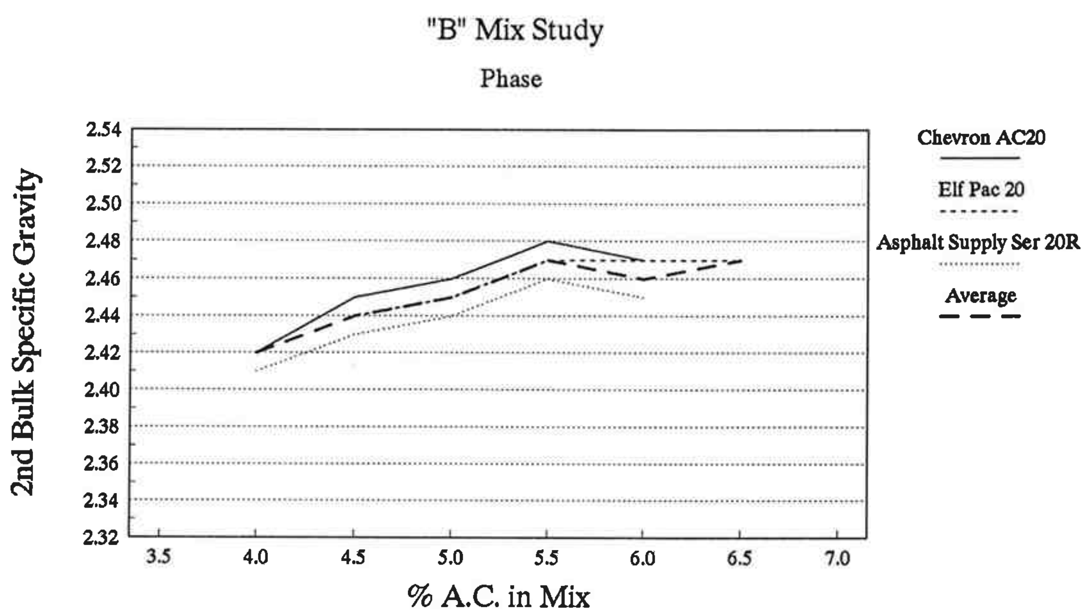
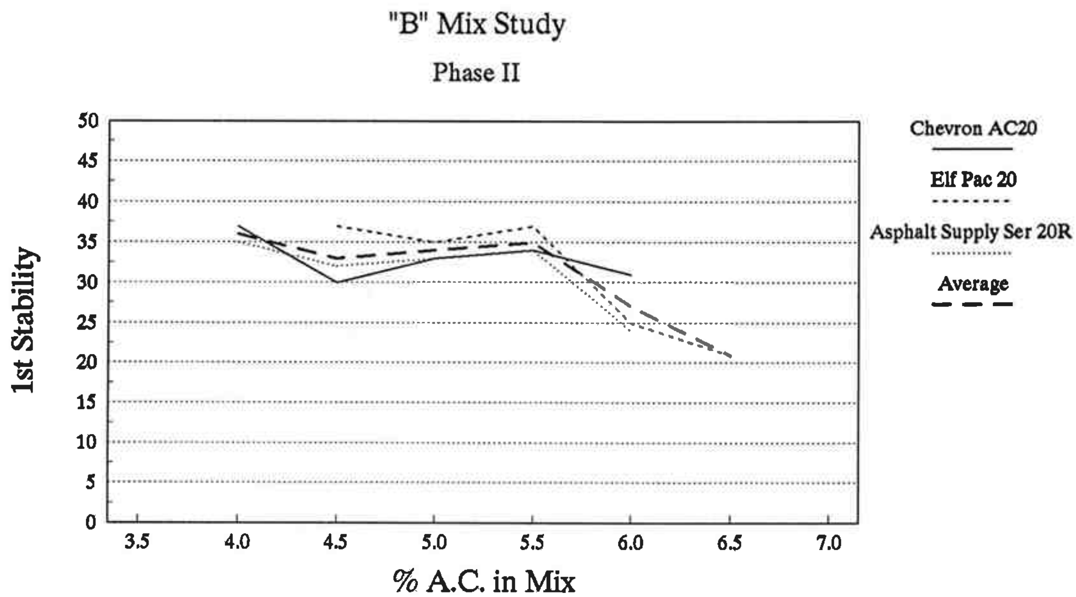


**FIGURE 13: Bulk Specific Gravity Test Results
Phase II**



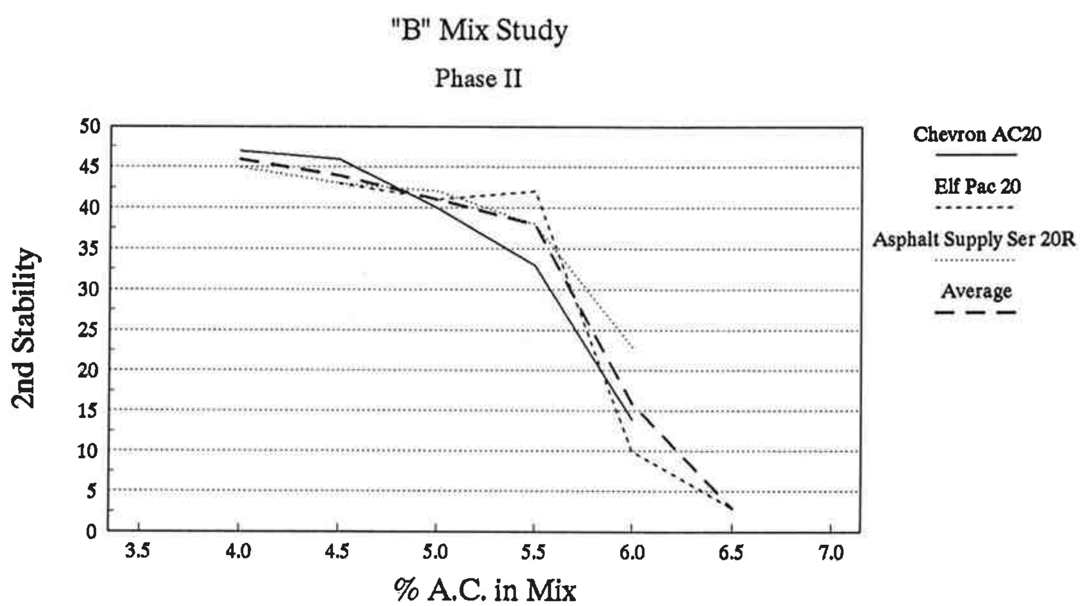
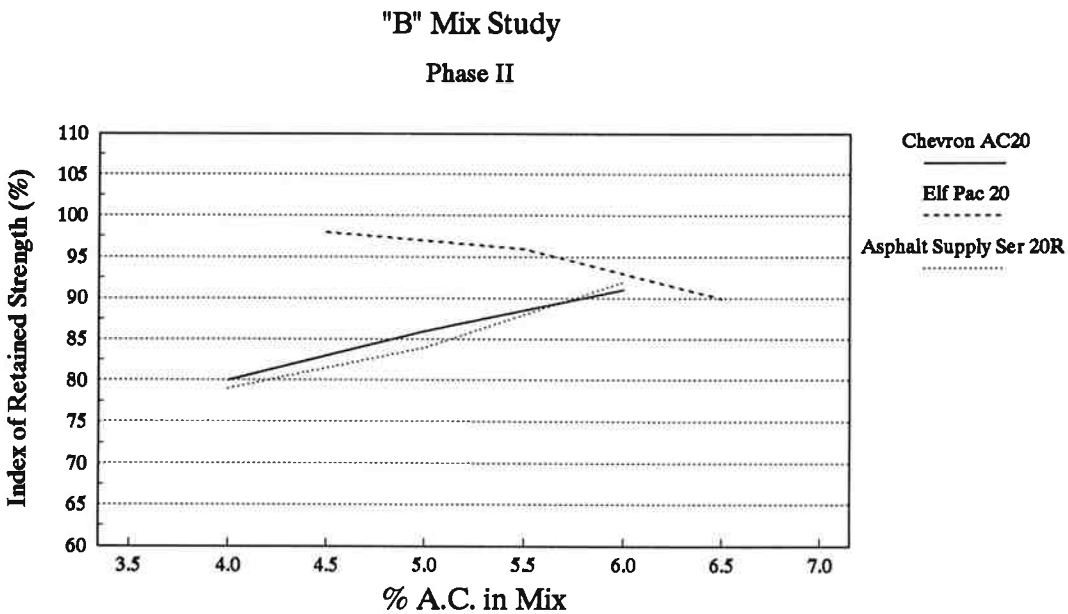


FIGURE 14: Stability Test Results
Phase II



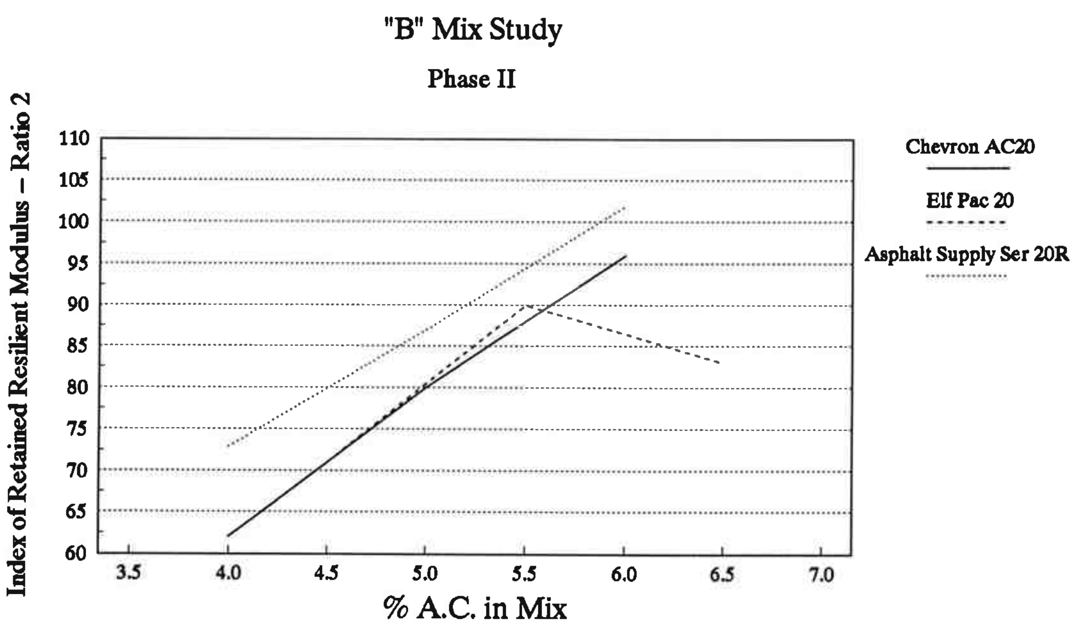


FIGURE 15: IRS and IRMR Test Results Phase II

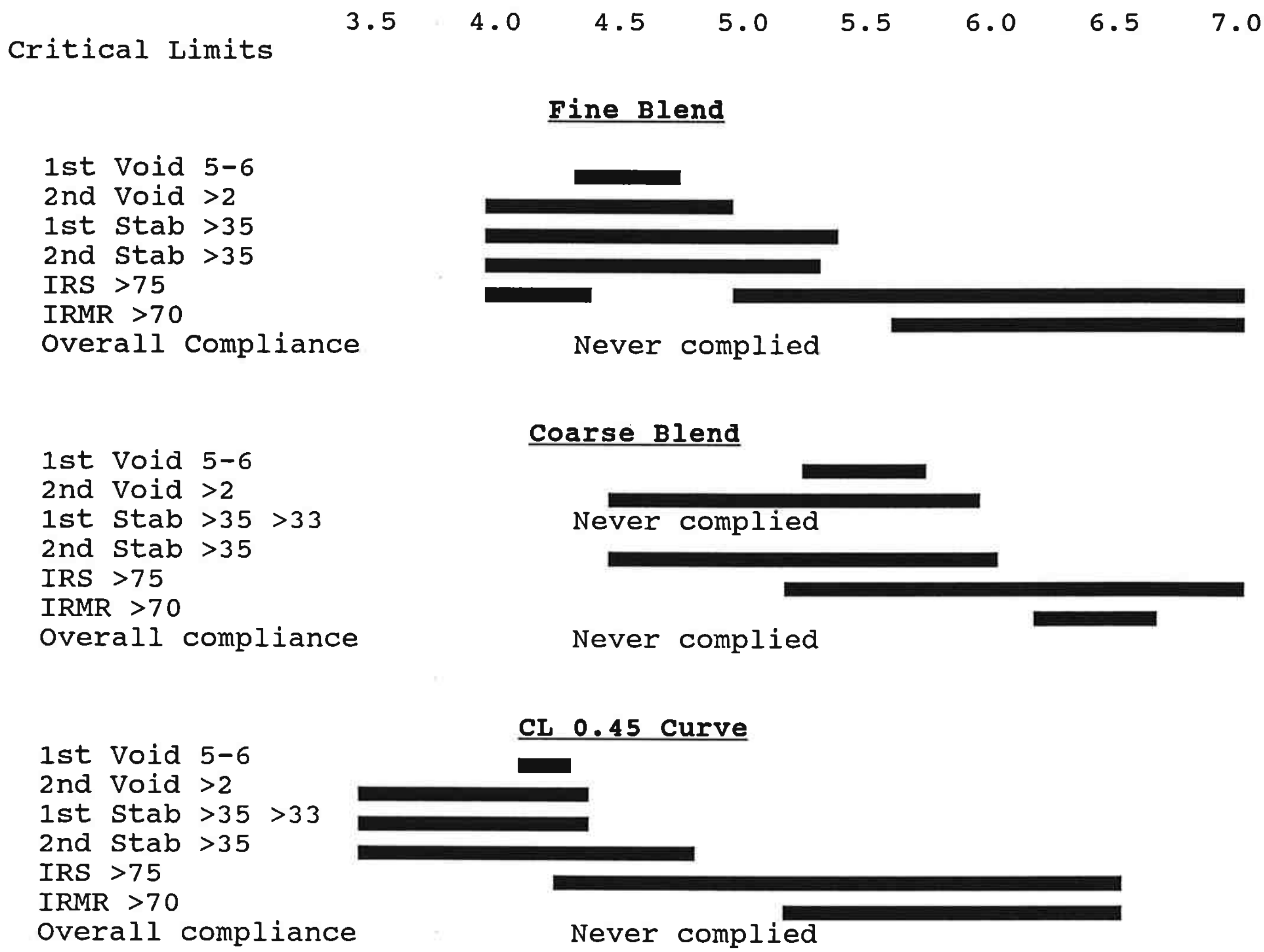


FIGURE 16: Phase I Design Criteria Compliance

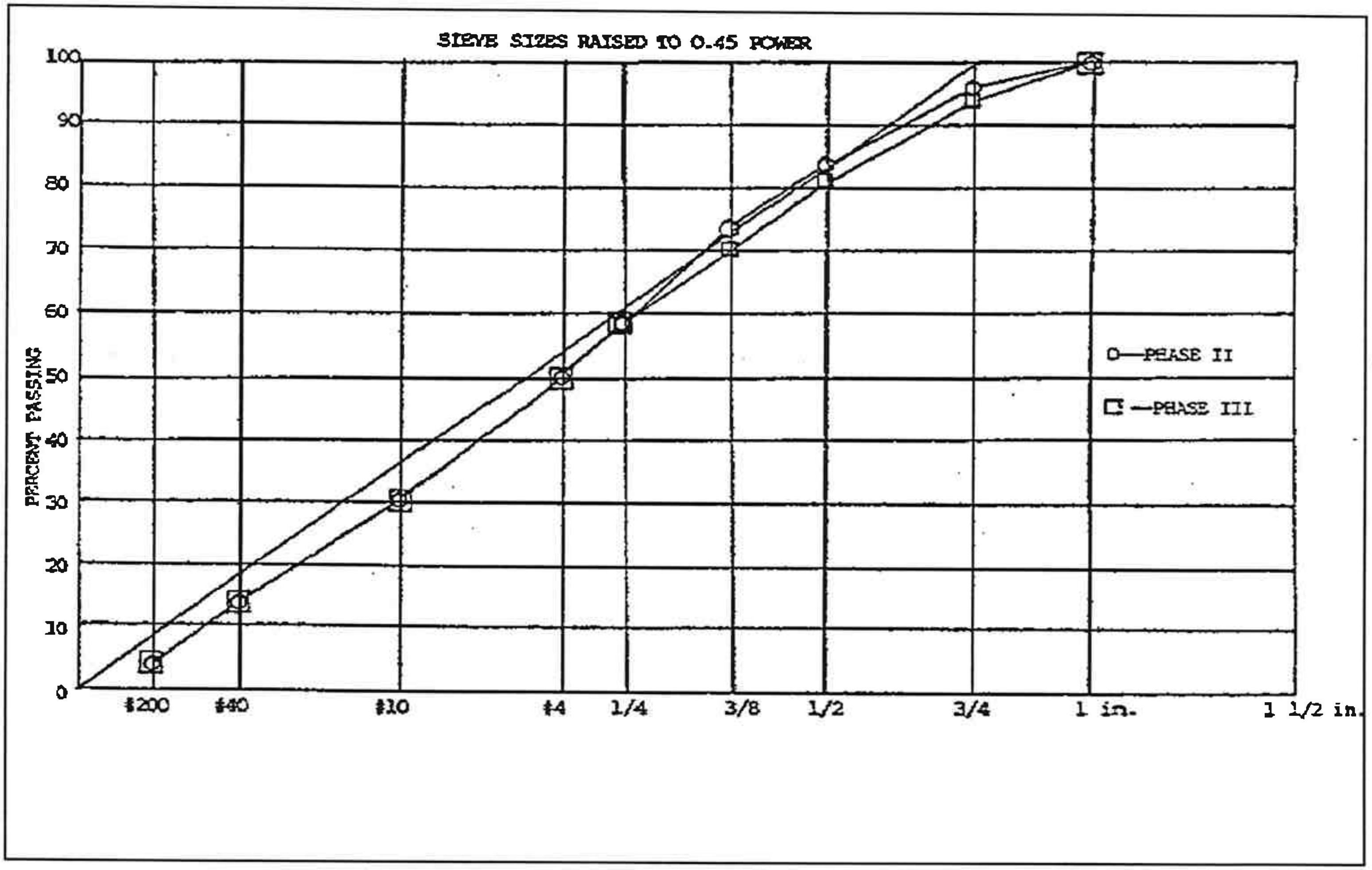
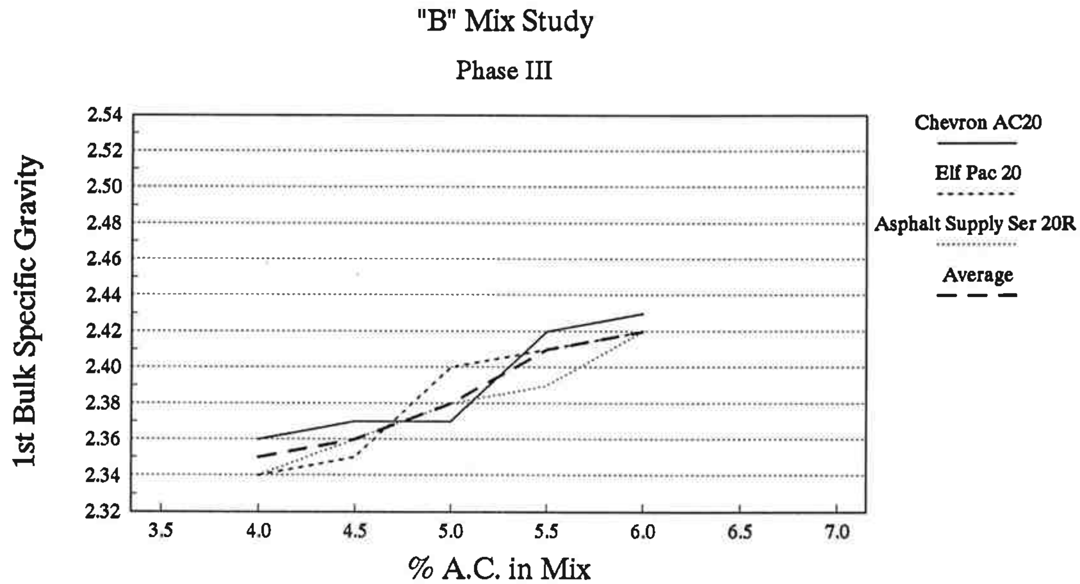


FIGURE 17: Gradation Chart for Phase II and III



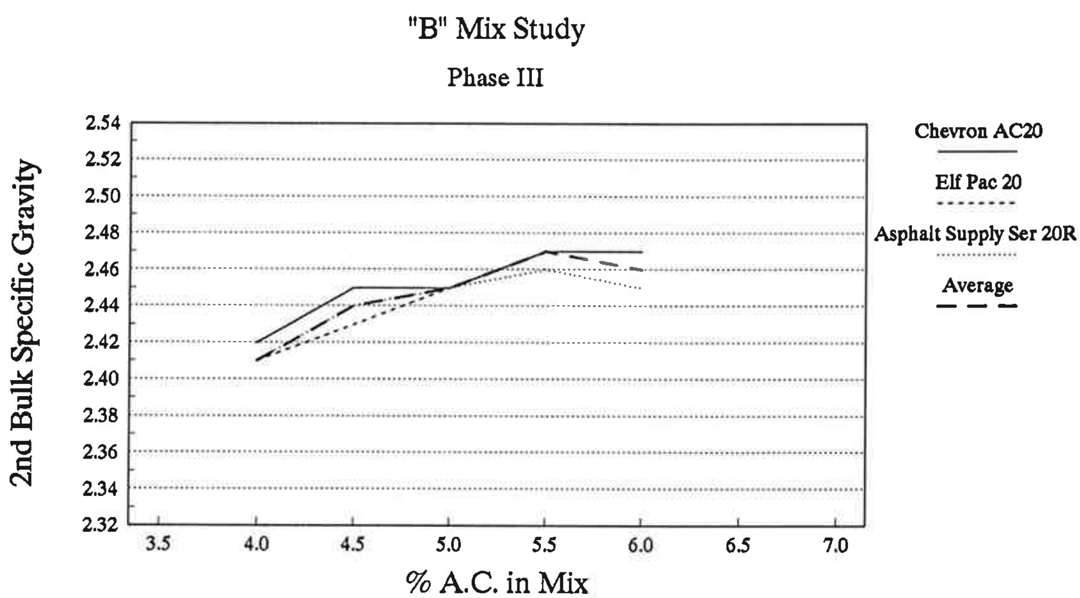


FIGURE 18: Bulk Specific Gravity Test Results
Phase III

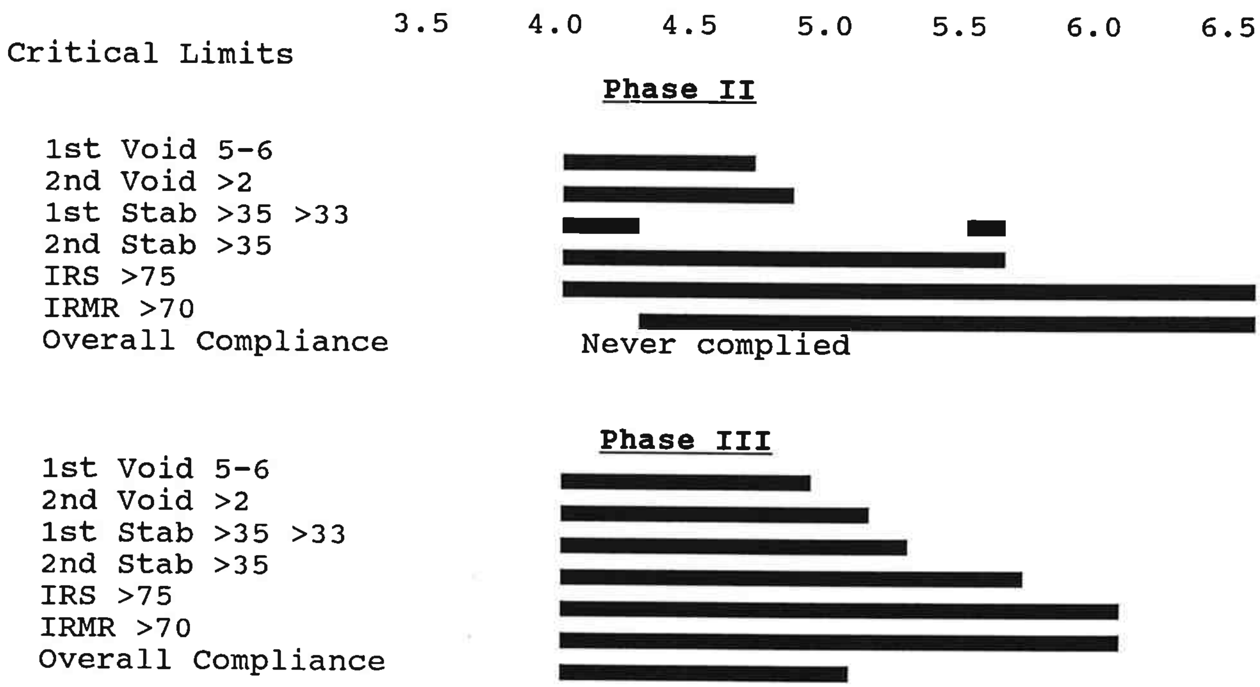
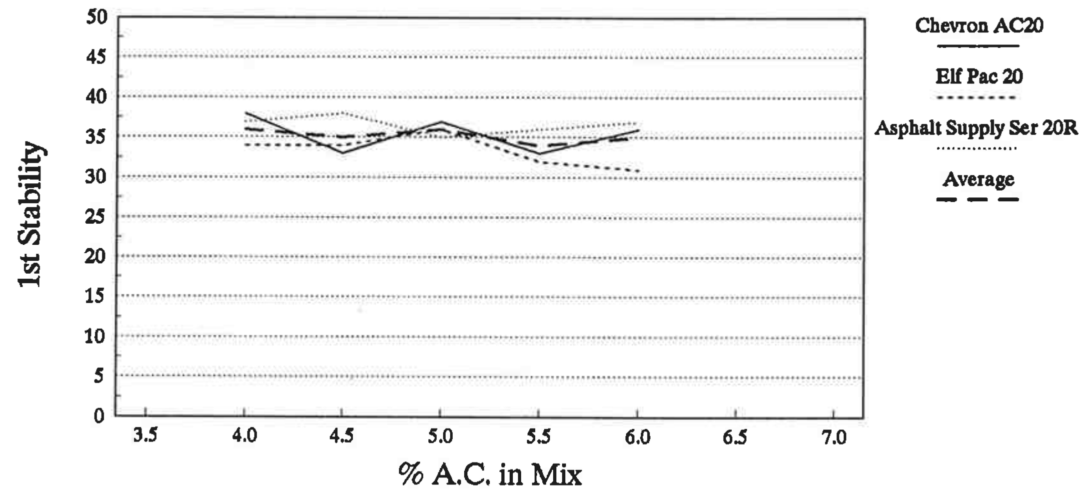


FIGURE 19: Design Criteria Compliance for Phase II & Phase III

"B" Mix Study

Phase III



"B" Mix Study

Phase III

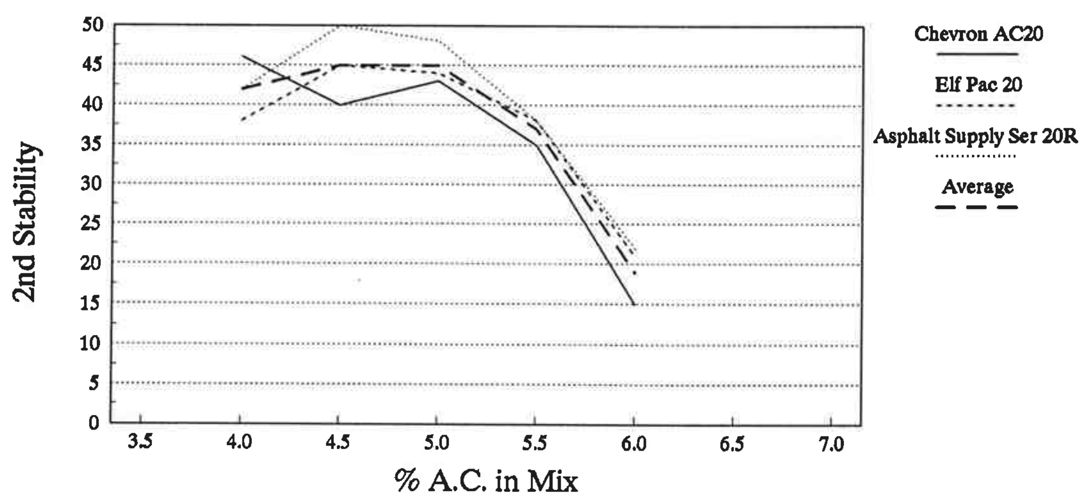
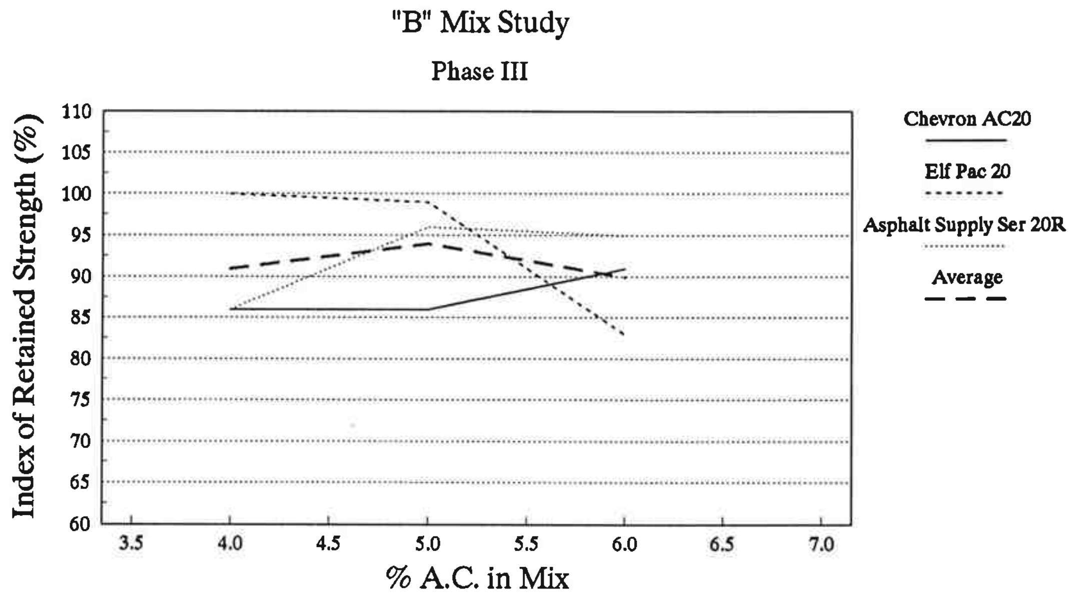


FIGURE 20: Stability Test Results
Phase III



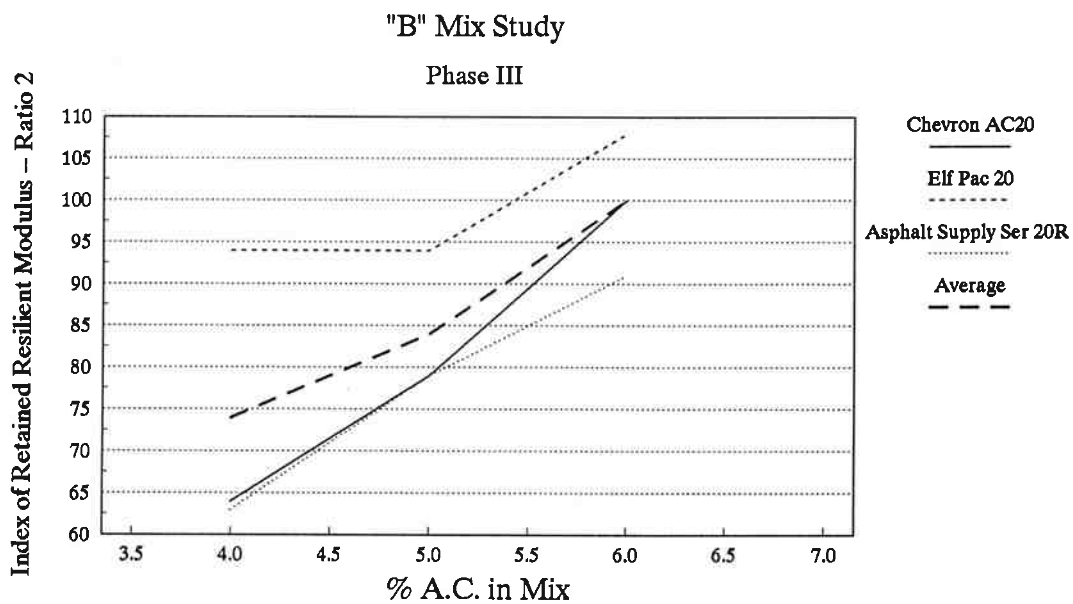


FIGURE 21: IRS and IRMR Test Results
Phase III